NCHRP
REPORT 455

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Recommended Performance-Related Specification for Hot-Mix Asphalt Construction: Results of the WesTrack Project

In Cooperation with

U.S. Department of Transportation
Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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Recommended Performance-Related Specification for Hot-Mix Asphalt Construction: Results of the Westrack Project

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SUBJECT AREAS
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Edward T. Harrigan
Staff Officer
Transportation Research Board

This report presents an overview of the WesTrack accelerated pavement testing experiment; a detailed description of its principal product, a performance-related specification for hot-mix asphalt (HMA); and a summary of observations made and lessons learned during the course of this major program. The report will be of particular interest to personnel of state highway agencies, materials suppliers, and paving contractors with responsibility for specification and production of HMA and construction of asphalt pavements, and to others with an interest in accelerated pavement testing and development and implementation of performance-related specifications.

WesTrack refers to an experimental test road facility constructed at the Nevada Automotive Test Center (NATC) near Fallon, Nevada, under the Federal Highway Administration (FHWA) project "Accelerated Field Test of Performance-Related Specifications for Hot-Mix Asphalt Construction" (Contract No. DTFH61-94-C-00004). The project was conducted by the WesTrack team, a consortium of seven public- and private-sector organizations lead by the NATC and including Granite Construction Co., Harding Lawson and Associates, Nichols Consulting Engineers, Chtd., Oregon State University, the University of California, Berkeley, and the University of Nevada, Reno.

The WesTrack experiment had two primary objectives. The first was to continue development of performance-related specifications (PRS) for HMA construction by evaluating the impact of deviations in materials and construction properties from design values on pavement performance in a full-scale, accelerated field test. The second was to provide some early field verification of the Superpave® mix design procedures. Because the WesTrack site typically experiences less than 100 mm of precipitation per year and no frost penetration, it was well suited for evaluating the direct effects of deviations of materials and construction properties on performance.

WesTrack was constructed as a 2.9-km oval loop incorporating twenty-six 70-m-long experimental sections on the two tangents. Construction was completed in October 1995; trafficking was carried out between March 1996 and February 1999. During this period, four triple-trailer combinations, composed of a tandem axle, Class 8 tractor and a lead semi-trailer followed by two single-axle trailers, operated on the track at a speed of 64 kph, providing 10.3 equivalent single-axle load (ESAL) applications per vehicle pass. The use of autonomous (driver-less) vehicle technology provided an exceptional level of operational safety and permitted loading to occur up to 22 hours per day, 7 days per week.

The experimental variables were asphalt content, in-place (i.e., field-mixed, field-compacted) air void content, and aggregate gradation; the main response variables were rut depth and percentage of the wheelpath area with fatigue cracking. Approximately 5 million ESALs were applied during the trafficking period. Several original sections failed early in the experiment; they were replaced with a mix design that duplicated the coarse-graded mix experiment in the original construction, but changed from the crushed gravel used in the original sections to a more angular, quarried andesite aggregate. The total experiment yielded clearly differentiated levels of permanent deformation and fatigue cracking among the experimental sections.

The experimental results were analyzed to develop the performance models for permanent deformation and fatigue cracking that drive the PRS for HMA construction implemented in the alpha version of the software program *HMA Spec*. This specification statistically compares the predicted life-cycle cost of the "as-designed" HMA pavement with that of "as-built" HMA pavement calculated from measured quality control and acceptance data to determine pay factors and pay adjustments for the paving project.

Construction, trafficking, and all related testing were accomplished under the FHWA contract. Data analysis, PRS development, and reporting were completed under NCHRP Project 9-20, "Performance-Related Specifications for Hot-Mix Asphalt Construction," as a cooperative effort with FHWA.

This final report is organized as four separate parts in one volume. Part I is a detailed overview of the planning, construction, operation, and data collection of the WesTrack experiment. Part II describes the development, components, and features of the HMA PRS and the *HMA Spec* software. Part III is a description of the WesTrack Database, a Microsoft® Windows-based relational database that, for easy access and use, contains the key pavement performance results of the experiment, including pavement distress data, materials properties, and weather and seasonal data. Finally, Part IV presents key observations and lessons learned by the WesTrack team during all phases of the WesTrack project; these will be of value to the engineering community in general, but more specifically to those involved in future, full-scale accelerated pavement testing operations, the development of models for pavement thickness design or HMA design purposes, or both.

Much of this final report is derived from 44 comprehensive technical reports prepared by the NATC, Nichols Consulting Engineers, Oregon State University, the University of California, Berkeley, and the University of Nevada, Reno. A companion CD-ROM (FHWA-RD-02-094 / CRP-CD-9) will contain the full text of these technical reports, the final report itself in portable document file (.pdf) format, and the WesTrack Database. The *HMA Spec* software is not available for public distribution; further development and validation is underway in NCHRP Project 9-22, "Beta Testing and Validation of HMA PRS."

CONTENTS

Note: Each part has its own Contents page.

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under contract to the U.S. DOT's FHWA and the NCHRP by the Nevada Automotive Test Center (NATC). Nichols Consulting Engineers (NCE) was the prime subcontractor and Granite Construction Company was a subcontractor. The University of California at Berkeley (UCB), University of Nevada at Reno (UNR), Oregon State University (OSU), and Harding Lawson and Associates (HLA) were subcontractors to Nichols Consulting Engineers.

Colin Ashmore at the NATC was the program manager. Jon Epps, UNR, and Steve Seeds and Sirous Alavi, NCE, were principal investigators on the project. Terry Mitchell was the contracting officer's technical representative for the FHWA and Edward Harrigan was the senior program officer for the NCHRP.

The WesTrack project received technical support from a number of groups during the conduct of the project. The FHWA technical panel and the NCHRP Project 9-20 panel provided overall technical input. Members of the FHWA technical panel included Ron Collins, John D'Angelo, Dale Decker, John Hallin, Joe Massucco, Jack Montrose, Dave Newcomb, Charles Potts, Tom Peterson, Dean Weitzel, and Gary Whited. Ray Bonaquist and Chris Williams provided additional technical assistance in their role as alternate contracting officer's technical representatives. Members of the NCHRP Project 9-20 panel included John D'Angelo, Vince Aurilio, Wade Betenson, Dale Decker, Thomas Hoover, M.W. "Mike" Lackey, Larry Michael, Terry Mitchell, David Newcomb, Charles Potts, Ronald Sines, Haleem Tahir, James Warren, Dean Weitzel, and Gary Whited.

Additional project technical support was received from a forensic team used to investigate the early distress in the replacement sections and from a group of consultants to the WesTrack project staff. The forensic team included Ray Brown, Erv Dukatz, Gerald Huber, Larry Michael, Jim Scherocman, and Ronald Sines. Consultants to the WesTrack team included Fred Finn, Garry Hicks, Paul Irick, Dave Newcomb, Jim Shook, and Dick Weed.

During construction and reconstruction of WesTrack, a large workforce was used. Key personnel for Granite Construction Company included Mike Robinson, Kevin Robertson, Kevin Atkins, Les Platt, Jim Lind, James Munson, and Dino Smernis. The FHWA supplied its mobile asphalt binder and mixture laboratories for mixture design and quality control/quality assurance (QC/QA) testing. Key personnel associated with the FHWA activities included Carl Gordon, Chuck Paugh, Julie Nodes, Larry Ilg, John D'Angelo, Chris Williams, and Tom Harman. Raj Basavaraju, Andrew Brigg, Mark Potter, and Haiping Zhou were among those at NCE who were active with QC testing and data reduction during the field construction stage. HLA performed QC testing at the field laydown site. Mike Hobbs, Dan Ridolfi, and Don Shervold were among those involved in these activities for HLA. BRE, Inc., of Austin, Texas, represented by Brian Killingsworth, also performed some of the QC testing. FHWA personnel were also involved in plant and field QC/QA activities. Joe Massucco organized this group.

A large research team was needed to plan, construct, operate, test, summarize data, and prepare the reports for this project. Personnel are identified in the introductory section of the report. Key personnel for the various contractors not previously identified include the following: Henry Hodges, Jr., at NATC; Weston Ott, Joseph Mactutis, Jim Nichols, Tony Lorenzi, Magdy Mikhail, and Carol Chiappetta at NCE; Maria Ardila-Coulson, Moetaz Ashkour, Lisa Cody, Shameem Dewan, and Steve Healow at UNR; Charles Shin, Maggie Paul, David Hung, Irwin Guada, Clark Scheffy, Lorina Popescu, Bor-wen Tsai, and Samar Madanat at UCB; Tom Walker, Chris Waters, and Derryl James at OSU; and Debbie Davis, Stuart Dykins, and John Welsh at HLA.

A team of truck and truck component manufacturers was assembled to support the development of the advanced transportation technologies. The vehicle team members included the following companies: Navistar (vehicles), Detroit Diesel (engine control), Twin Disc (automatic transmissions), Goodyear (tires), Haldex Brake Systems (brake-by-wire and ABS technology), Alcoa (aluminum wheels), Disc-Lock (wheel fasteners), Shell Oil (lubricants), and East Pennsylvania Manufacturing (batteries).

Astec Industries, Huntway Refining, Idaho Asphalt, and Chemical Lime Company contributed to various materials-and-construction-related activities during the construction and reconstruction operations.

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PART I:
PROJECT OVERVIEW

CONTENTS

CHAPTER 1

INTRODUCTION AND BACKGROUND

1.1 INTRODUCTION

WesTrack is a multimillion dollar accelerated pavement test facility located in the State of Nevada approximately 100 km (60 mi) southeast of Reno (Figure 1 [figures and tables are provided separately at the end of each chapter]). The pavement test facility was designed, constructed, and operated by a team of private companies and universities (the WesTrack team) under contract to the U.S. Department of Transportation's Federal Highway Administration (FHWA) and the National Cooperative Highway Research Program (NCHRP). The project was awarded to the WesTrack team by the FHWA in September 1994. The test track, which includes 26 hot-mix asphalt (HMA) test sections, was designed and constructed between October 1994 and October 1995. Traffic was initiated in March 1996 and was completed in February 1999. Five million equivalent single-axle loads (ESALs) were placed on the track during the trafficking period.

The initial sponsorship by the FHWA provided for the design of the track, construction of the track, design of the driverless vehicles, trafficking, performance measurements, sampling and testing of materials, preliminary analysis of materials data, and development of the WesTrack database. The NCHRP provided funding for continued analysis of materials data, development of performance models, development of the performance-related specification (PRS), and reporting.

1.1.1 Background

HMA is used extensively throughout the United States and the world as a cost-effective pavement surfacing material for highways, streets, air fields, and parking lots. More than 500 million Mg (550 million tons) of HMA are placed annually in the United States at a cost of nearly 18 billion dollars. Small improvements in the life of HMA can result in large economic savings to those public agencies and private groups that are responsible for funding, constructing, rehabilitating, and maintaining pavements.

A Strategic Transportation Research Study (STRS) conducted in the early and mid 1980s recognized the potential savings associated with life extension of HMA pavements and defined a research program to develop an asphalt mixture analysis system. The resulting 5-year research effort was conducted as part of the Strategic Highway Research Program (SHRP) and was completed in 1992. The asphalt portion of the SHRP provided an asphalt binder specification and an HMA mixture design method based on the use of the gyratory compactor and performance test for rutting, fatigue, low temperature or thermal cracking, aging, and water sensitivity. Because of the relatively short duration of the SHRP research program, only a limited amount of field performance information was used to calibrate and correlate the newly developed tests and acceptance criteria for the asphalt binders and the HMA mixture design method.

Prior to SHRP, the technology used to design and construct HMA materials was based largely on research conducted in the 1930s and 1940s associated with the Marshall and Hveem mixture design methods. Increased truck traffic volumes and truck tire pressures and perhaps changes in materials (aggregates and asphalt binders) created an increased number of projects with premature distress of the HMA starting in the late 1970s and continuing into the 1980s. Many public agencies changed their specifications to reduce the premature pavement distress that was occurring during this period. Some of the specification changes resulted in more widespread use of modified asphalt binders, crushed aggregates, "cleaner" aggregates, volumetric mixture design principles, in-place air void requirements, and quality control/quality assurance (QC/QA) types of specifications. Many of these changes have resulted in better performing pavements.

The combination of changes in specification and construction practices by the public agencies and the implementation of the SHRP research findings in the 1990s have produced higher quality HMA pavements. In the early 1990s, however, additional gaps in information resulted in several major research projects associated with HMA. Three of the more visible research efforts initiated in the mid 1990s include projects to (1) improve the structural design practice of HMA pavements, (2) develop improved performance tests for HMA, and (3) define relationships among material properties and pavement performance with the use of accelerated pavement testing on a full-scale test track.

This report, *NCHRP Report 455*, "Recommended Performance-Related Specification for Hot-Mix Asphalt Construction: Results of the WesTrack Project," is the final report for the last of these three projects.

1.1.2 Performance-Related Specifications

PRS research has been ongoing in the United States under the primary sponsorship of the FHWA and NCHRP. Reports on the second phase of the PRS development for HMA by Shook et al. (*1*) and the second phase of PRS development for portland cement concrete (PCC) pavements by Darter et al. (*2*) provide a chronology of the research that has been accomplished over the years in PRS system development.

A PRS system for pavements is a method or model that allows pavement engineers to prepare practical specifications for pavement construction that focus on the actual material properties and construction practices that have the most effect on the long-term performance of the pavement. By considering the multitude of costs associated with the design, construction, and future performance of a pavement, the system provides not only a means for identifying or specifying a cost-effective "target" pavement to build initially, but also a means for equitably rewarding or penalizing the contractor for the "as-constructed" pavement delivered. Under such a system, the contractor on a given job could be penalized for not meeting the specification for subbase compaction and be rewarded for exceeding the target specification for initial pavement surface smoothness. Assuming that the predicted long-term performance of the pavement is dependent more on its initial smoothness than on the compacted density of a subbase material, the net effect would be reward for the contractor and some assurance for the client (road agency) that its funds were well spent.

This project is one of several that will be necessary to develop a comprehensive PRS for HMA pavements. The long-term developmental effort must involve three key points:

- The PRS developed for asphalt binder and HMA mixtures in the SHRP was the result of a highly focused, relatively short-term research program. Because of the time and financial limitations of the SHRP study, performance relationships among material property measurements and pavement performance are considered incomplete by a segment of the pavement engineering community. In addition, the focus of the research was not directed toward defining the cost consequences of noncompliant materials, but to developing test methods and acceptance criteria that were related to pavement performance.
- Although the properties and characteristics of the asphalt binder, aggregate, and HMA mixture are key factors in the PRS system, they are not the only factors. A PRS system is designed to consider variables that affect the performance of an asphalt pavement, including the properties of the other layers.
- A PRS is a true "system" as defined by "systems methodology." Consequently, the various components that constitute the system should be upgradeable or replaceable. Thus, improved methods developed under ongoing or future research efforts can ultimately be incorporated into the PRS.

1.1.3 WesTrack Project

Research conducted by Shook et al. (*1*) under the second phase of PRS systems development of HMA focused on (1) identifying the most significant materials and construction factors that affect asphalt pavement performance and (2) developing secondary prediction relationships among these factors and other factors or variables found in the available primary performance prediction relationship. The findings of this Phase 2 effort were based, in large part, on the results of a laboratory study of HMA mixtures. Although there was some overlap with the work conducted as part of the SHRP asphalt research program in terms of the materials being tested, the laboratory tests used in the Phase 2 study were state-of-the-practice tests, not the new SHRP developed tests.

The Phase 2 PRS system development effort for HMA provided recommendations for a full-scale accelerated field test to investigate the actual pavement performance impacts of contractor nonconformance to an HMA specification. In addition, the research provided some guidance on how the results from the SHRP asphalt program could be adapted within the PRS system framework.

The Phase 2 study finding, together with the long-term research goals of the FHWA relative to PRS, formed the basis for soliciting a research project titled "Accelerated Field Test of Performance-Related Specifications for Hot-Mix Asphalt Construction." The project was awarded at the end of September 1994 to a group of private companies, academic institutions, and a construction company that are collectively referred to as the WesTrack team. The project has commonly been called the "WesTrack" project.

1.1.4 Objectives

The objectives of the WesTrack project are as follows:

- Continue the development of PRS for HMA pavements by evaluating the effect of variations in materials and construction quality (asphalt binder content, aggregate gradation, in-place air void content, and so forth) on pavement performance as evaluated by a full-scale accelerated field test track.
- Provide early field verification of the SHRP Superpave® volumetric mixture design procedure.

The primary product of this research effort is a PRS for HMA based on performance models derived from the accelerated pavement testing performed at the WesTrack facility. Valuable field verification information for Superpave mixtures was also obtained and has resulted in changes in some of the original Superpave specifications and methods for HMA mixture design.

This research effort considers two primary types of HMA pavement distress: (1) permanent deformation or rutting and (2) fatigue cracking. Thickness design and HMA mixture design considerations concentrated on developing a facility and mixture that would rut and fatigue crack during the experiment.

1.2 WESTRACK TEAM

1.2.1 Organization

The WesTrack team consists of seven organizations, each with specific roles in the project as defined in Table 1 and Figure 2. Nevada Automotive Test Center (NATC) was the prime contractor and was responsible for project management, driverless vehicle development, and trafficking, as well as the collection of some performance information. The test track facility is located on NATC property.

Nichols Consulting Engineers, Chtd., (NCE) was the prime subcontractor and was responsible for project management, construction management of the subgrade and base course placement, sampling, performance monitoring, WesTrack database, and PRS.

The University of Nevada at Reno (UNR); University of California at Berkeley (UCB); Oregon State University (OSU); and Harding Lawson and Associates (HLA) were subcontractors to NCE. The UNR was responsible for project management, construction management of the HMA, construction materials sampling, some of the QA testing and conventional asphalt binder and HMA testing. UCB was responsible for advanced HMA testing for rutting and fatigue cracking as well as performance modeling. OSU was responsible for advanced HMA testing for thermal cracking and water sensitivity as well as performance modeling.

HLA assumed responsibility for geometric design of the track, preparation of the plans and specifications, construction inspection, and some of the QC/QA testing.

Granite Construction Company was a subcontractor to NATC and was responsible for construction, rehabilitation, and maintenance of the track. Granite Construction was a member of the WesTrack team and was involved in decisionmaking throughout the project.

The FHWA and the NCHRP were considered team members and participated in the decisionmaking and were actively involved in the HMA mixture design as well as QC testing and performance testing.

Five members of the WesTrack team are located within 100 km (60 mi) of Reno, Nevada. The UCB is 350 km (220 mi) from Reno and OSU is 650 km (400 mi) from Reno. Figure 1 shows the location of each team member in northern Nevada.

1.2.2 Personnel

Colin Ashmore of NATC was the project manager. Sirous Alavi, Steve Seeds, and Jon Epps were the co-principal investigators. Principal personnel from each of the WesTrack team members, including the FHWA and the NCHRP are shown in Table 1. Colin Ashmore at NATC; Sirous Alavi, Weston Ott, Joseph Mactutis, Steve Seeds, and Todd Scholz at NCE; Jon Epps, Adam Hand, and Peter Sebaaly at UNR; Carl Monismith at UCB; Rita Leahy at OSU; Stuart Dykins at HLA; Mike Robinson and Kevin Robertson at Granite Construction; Terry Mitchell, Chris Williams, John D'Angelo, and Ray Bonaquist at FHWA; and Edward Harrigan at NCHRP were the principal personnel on the project.

During construction of the project, the WesTrack team was significantly expanded. For example, the workforce and the organizational structure involved in the placement of the HMA is shown on Figure 3. Personnel from NATC, NCE, UNR, HLA, FHWA, and Granite Construction were involved. BRE Engineering also contributed to the effort as part of NCHRP Project 9-7 with funding from the WesTrack project. Several FHWA employees were temporarily assigned to the WesTrack team during the construction of the hot-mix. These individuals were responsible for sampling, hot-mix plant monitoring, and laydown and compaction monitoring.

A large group of engineers, technicians, and crafts-persons were involved in the design, construction, sampling, testing, performance monitoring, analysis, and report preparation effort for each of the team members. Table 2 lists more than 75 individuals involved in the WesTrack project.

1.2.3 Advisory Groups

Three advisory groups and an investigative team have been active with the WesTrack project as shown in Figure 4. As stated previously, the FHWA was the original and major financial sponsor of the project. An FHWA technical panel was formed to provide input from industry and state highway agencies as well as the federal government. This technical panel was active primarily during the formation of the experimental plan, construction, and early trafficking. The FHWA formed a "forensic team" to investigate the premature distress experienced on the replacement sections placed in summer 1996.

The NCHRP provided funding to complete the project. An NCHRP panel was formed to guide the analysis and report preparation portion of the project. As noted in Figure 4, the NCHRP panel and the original FHWA advisory group had common membership to provide continuity to the project. Table 3 lists FHWA and NCHRP advisory group meeting dates and topics.

The WesTrack team used a small group of consultants to provide an external review for the project. This group consisted of individuals who were part of the AASHO Road Test research team in the 1959–1962 period as well as state highway department personnel familiar with statistical specifications.

The members of the various advisory groups are shown in Figure 4.

1.3 REPORT ORGANIZATION

This report is organized into four parts:

- Part I: Project Overview.
- Part II: Performance-Related Specification.
- Part III: WesTrack Database.
- Part IV: Observations and Lessons.

Each part of this report has been further divided into chapters and subsections. The overall report format and the individual chapters and subsections in Part I are shown in Figure 5. The chapters in this part are as follows:

- Chapter 1: Introduction and Background.
- Chapter 2: Preconstruction Activities.
- Chapter 3: Construction.
- Chapter 4: Operations.
- Chapter 5: Materials Characterization and Performance Models.
- Chapter 6: Reports and Public Information Activities.

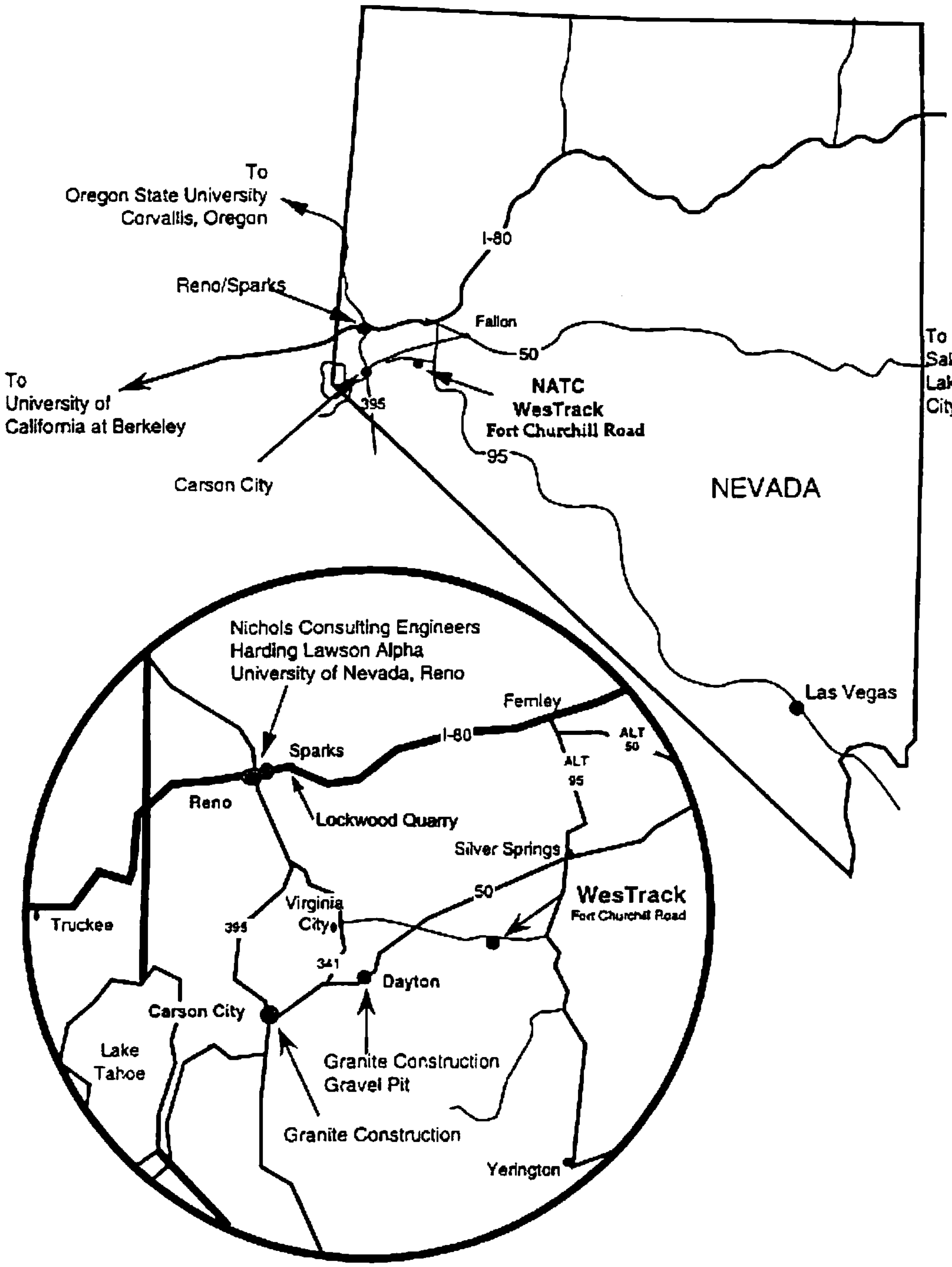


Figure 1. Location of team members.

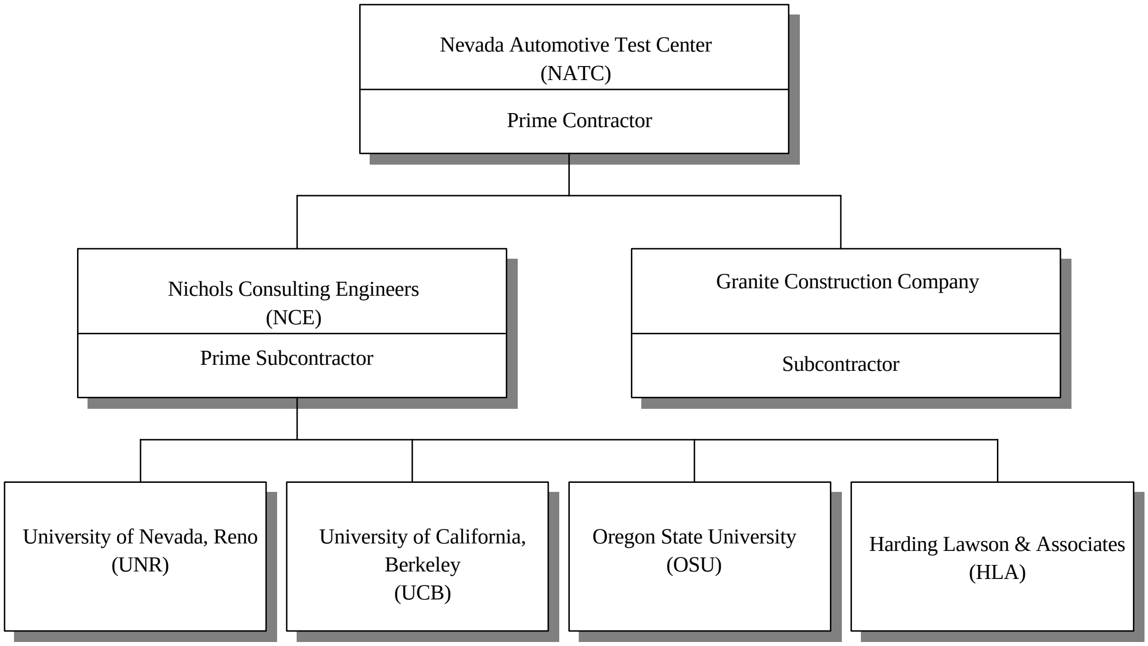


Figure 2. *WesTrack team organizational structure.*

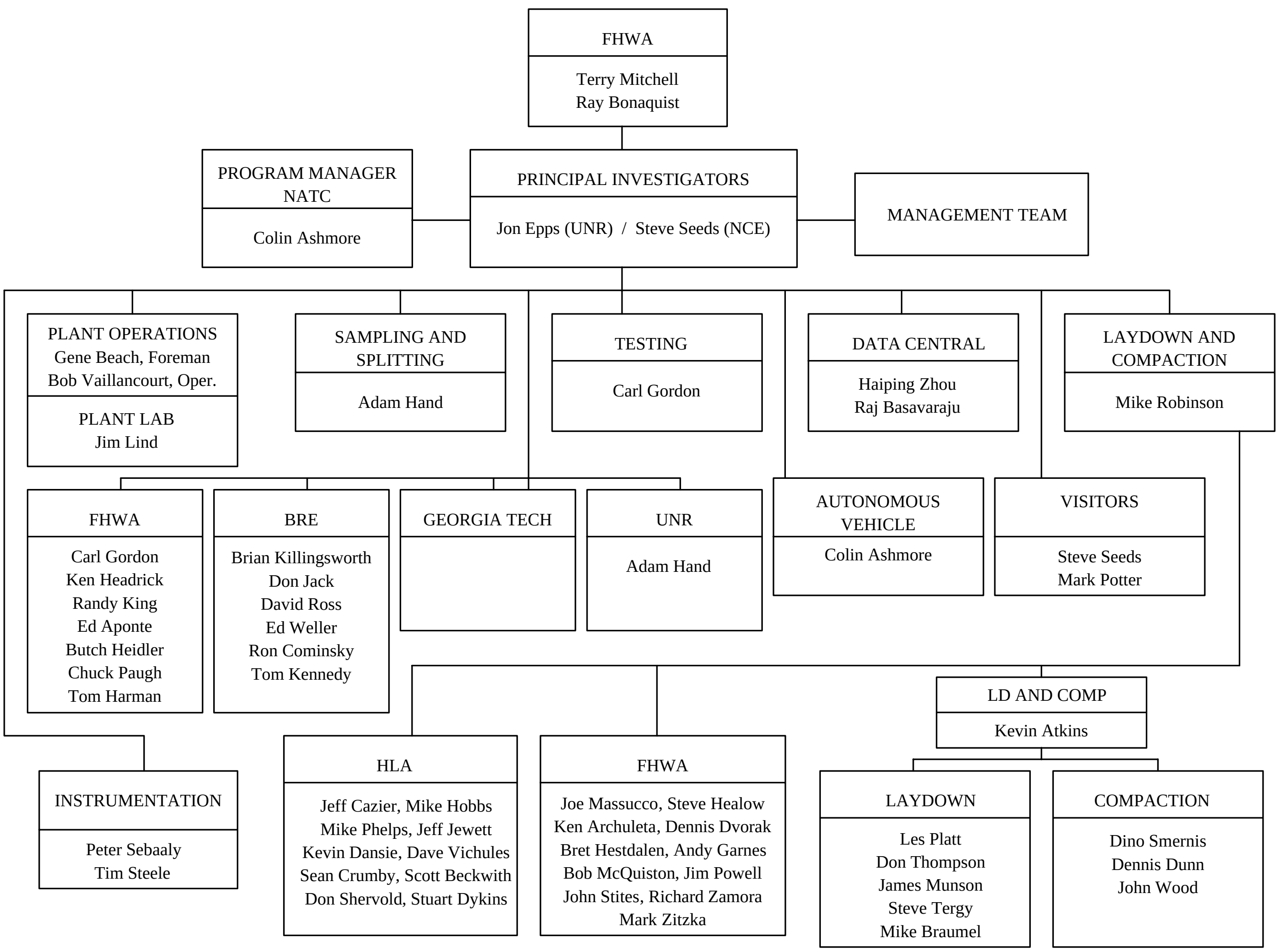


Figure 3. *HMA construction organization and personnel.*

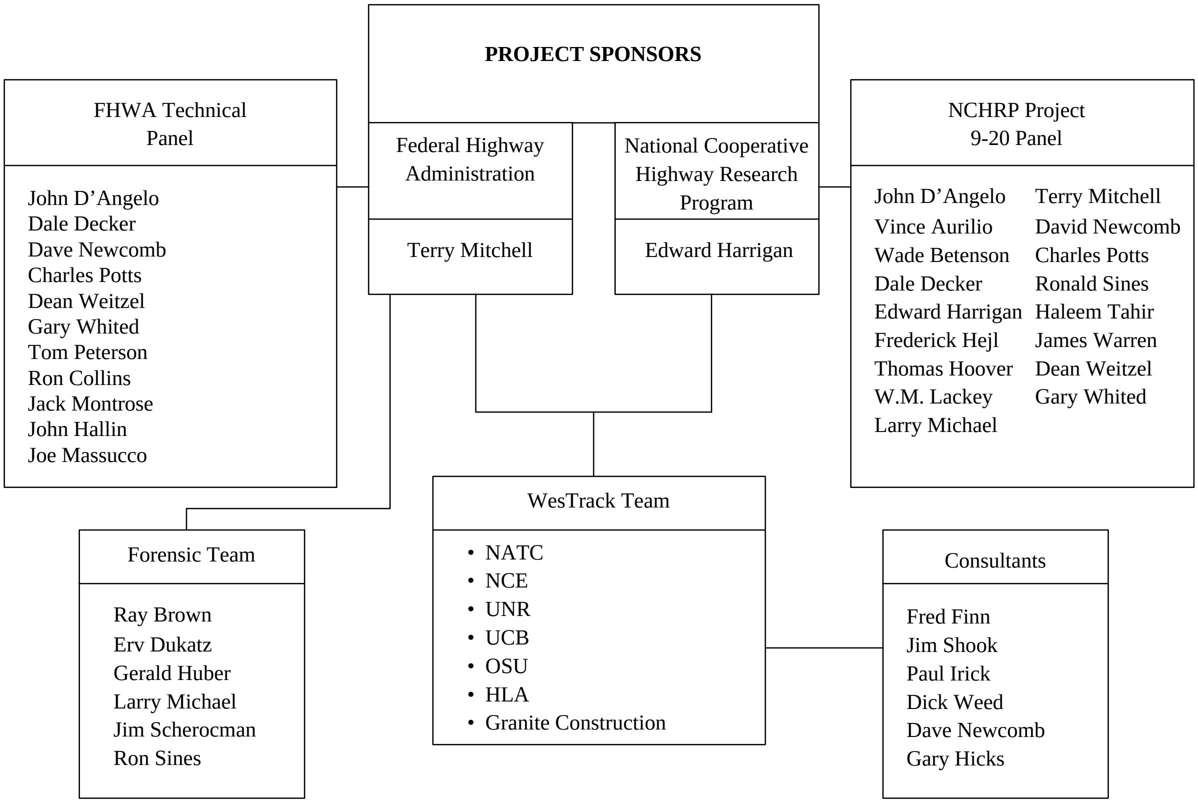


Figure 4. Advisory groups.

WESTRACK REPORT

PART I	**PART II**	**PART III**	**PART IV**
Project Overview	Performance-Related Specification	WesTrack Database	Observations and Lessons
1. Introduction and Background 2. WesTrack Team 3. Preconstruction Activities 4. Construction 5. Operations 6. Materials Characterization and Performance Models 7. Reports and Implementation			

Figure 5. Report organization.

TABLE 1 WesTrack team

Organization/Location	Primary Role(s)	Principal Personnel
Nevada Automotive Test Center (Silver Springs, Nevada)	• Prime contractor • Project management • Driverless vehicle development • Trafficking • Performance monitoring	• Henry Hodges, Jr. • Colin Ashmore • Paul Pugsley • Dave Heinz • Rick Capps • Randy Carlson • Muluneh Sime
Nichols Consulting Engineers, Chtd. (Reno, Nevada)	• Prime subcontractor • Project management • Construction management • Sampling • Performance monitoring • Relational database • Performance-related specification	• Sirous Alavi • Weston Ott • Joseph Mactutis • Larry Musa • Steve Seeds • Todd Scholz
University of Nevada, Reno (Reno, Nevada)	• Project management • Construction management • QA testing • Conventional testing	• Jon Epps • Adam Hand • Peter Sebaaly
University of California at Berkeley (Berkeley, California)	• Permanent deformation testing • Fatigue testing • Performance prediction models	• Carl Monismith • J.A. Deacon • J.T. Harvey
Oregon State University (Corvallis, Oregon)	• Thermal cracking testing • Water sensitivity testing • Performance prediction models	• Rita Leahy
Harding Lawson and Associates (Reno, Nevada)	• Geometric design • Plans and specifications • Construction inspection • QC/QA testing	• Stuart Dykins • John Welsh
Granite Construction Company (Reno, Nevada)	• Construction • Rehabilitation • Maintenance	• Mike Robinson • Kevin Robertson
Federal Highway Administration (Washington, D.C.)	• Project oversight • Mix design • QC testing • Performance testing of hot-mix asphalt	• Terry Mitchell • Chris Williams • John D'Angelo • Ray Bonaquist
National Cooperative Highway Research Program (Washington, D.C.)	• Project oversight • PRS and report review	• Edward Harrigan

TABLE 2 WesTrack personnel

Organization	Personnel
Nevada Automotive Test Center	• Colin Ashmore
Nichols Consulting Engineers	• David Adams • Sirous Alavi • Raj Basavaraju • Carol Chiappetta • Allan Coldani • Kevin Kawalkowski • Tony Lorenzi • Joseph Mactutis • Magdy Mikhail • Larry Musa • Jim Nichols • Weston Ott • Mark Potter • Steve Seeds • Todd Scholz • Haiping Zhou
University of Nevada	• Maria Ardila-Coulson • Moetaz Ashour • Lisa Cody • Shameem Dewan • Jon Epps • Adam Hand • Steve Healow • Andrew Lake • Marty McNamara • Abdel Osama • Peter Sebaaly • Siva Sivasubramaniam • Tim Steele
University of California	• Adrian Archilla • S. Madanat • John Harvey • Carl Monismith • Charles Shin • J.A. Deacon • Maggie Paul • David Hung • Irwin Guada • Clark Scheffy • Lorina Popescu • Bor-wen Tsai • David Kim
Oregon State University	• Rita Leahy • Tom Walker • Chris Waters • Derryl James
Harding Lawson and Associates	• Scott Beckwith • Jeff Cazier • Sean Crumby • Kevin Dansie • Debbie Davis • Stuart Dykins • Mike Hobbs • Jeff Jewett • Mike Phelps • Dan Ridolfi • Don Shervold • John Welsh
Granite Construction	• Kevin Atkins • Gene Beach • Mike Braumel • Dennis Dunn • Jim Lind • James Munson • Les Platt • Kevin Robertson • Mike Robinson • Dino Smernis • Steve Tergy • Don Thompson • Bob Vaillancourt
Federal Highway Administration	• Ed Aponte • Ken Archuleta • Ray Bonaquist • John D'Angelo • Dennis Dvorak • Andy Garnes • Carl Gordon • Tom Harman • Ken Headrick • Steve Healow • Butch Heidler • Bret Hestdalen • Randy King • Joe Massucco • Bob McQuiston • Terry Mitchell • Chuck Paugh • Jim Powell • John Stites • Kevin Stuart • Chris Williams • Richard Zamora • Mark Zitzka
National Cooperative Highway Research Program	• Edward Harrigan

TABLE 3 FHWA technical panel and NCHRP panel meetings

Date	Location	Major Topics
February 2-3, 1995*	Reno, NV	• Geometric design of track • Experiment design
May 18-19, 1995*	Reno, NV	• Experiment design • Sampling and test plan (QC/QA) • Mixture design • Pavement structural design
October 31, 1995**	Washington, D.C.	• Construction summary • Budget review
April 3-4, 1997*	Reno, NV	• Review performance • Rehabilitation section designs
August 18-20, 1997**	Reno, NV	• Forensic team • Performance of reconstruction section • QC/QA reconstruction mix
February 1-2, 1999***	Reno, NV	• Review work plan • Project schedule
July 14-15, 1999***	Reno, NV	• Performance-related specifications • QC/QA test results • Performance models
October 21, 1999***	Reno, NV	• Performance-related specifications • Performance models • Relational database
November 29-30, 1999***	Reno, NV	• Performance models • Performance-related specifications

* FHWA Technical Panel
** Special Review Groups
***NCHRP Panel

CHAPTER 2

PRECONSTRUCTION ACTIVITIES

2.1 INTRODUCTION

A number of activities were performed by the WesTrack team prior to the construction of the test track. These preconstruction activities include the following:

- Literature Review.
- Experiment Design.
- Site Evaluation.
- Geometric Design.
- Driverless Vehicle Development.
- Pavement Instrumentation.
- Geotechnical Investigation.
- Pavement Thickness Design.
- Quality Control/Quality Assurance Test Plan.
- Plans and Specifications.
- Hot-Mix Asphalt Mixture Design.

Each of these activities is briefly described and discussed below.

2.2 LITERATURE REVIEW

A limited literature review was conducted during the first year of the project and reported in the Task G Interim Report (*3*). Six topic areas were targeted for review: HMA technology, HMA pavement construction variability, pavement test track/road test experiments, PRS, pavement instrumentation, and driverless vehicle technology.

2.2.1 Hot-Mix Asphalt Technology

The HMA technology review was directed toward identifying models and techniques that could be used to predict the performance of an asphalt pavement in terms of fatigue, permanent deformation, thermal cracking, roughness, friction, and raveling. Several fatigue and permanent deformation prediction models are available in the literature. Most of these models are based on tensile strain in the bottom of the HMA layer for fatigue prediction and compressive strain at the top of the subgrade for prediction of permanent deformation. Only a limited amount of information is available to predict permanent deformation in pavements that result from the permanent deformation of the HMA layer.

Several thermal cracking or cold temperature cracking models are contained in the literature review.

Two models in the literature review predict friction values of pavements. These models are based on data collected in the northeast and the southeast United States.

2.2.2 Construction Variability

The literature review conducted to define construction variability is contained in Chapter 12 of the Task G Interim Report (*3*) and in WesTrack Technical Report UNR-29 (*4*). Construction variability information is provided for density and water content of subgrade and fill materials; gradation, density and water content of subbase and base course materials; and gradation, asphalt binder content, Marshall properties, Superpave volumetric properties, temperature, in-place air voids, thickness, and smoothness of the HMA. Typical construction variabilities for HMA construction taken from field data from around the United States and expressed as standard deviations are shown in Table 4. This variability information was used to develop the PRS defined in Part II of this report.

2.2.3 Test Track and Road Test Experiments

Several facilities have been built around the world to study the response of pavement structures under simulated or actual traffic loading. The majority of these facilities were designed for accelerated loading and for studying the response of the road structure under controlled conditions (i.e., load, tire pressure, vehicle type, and pavement material properties). The various test facilities may be classified into three groups:

- Linear test tracks.
- Circular test tracks.
- Test tracks or road test experiments.

NCHRP Synthesis of Highway Practice 235, "Application of Full-Scale Accelerated Pavement Testing,"(*5*) provides details on these three types of facilities.

The literature review for this project focused on test tracks and road test experiments because WesTrack was to be designed as a facility with a fixed pavement structural section, with nearly identical subgrade and base course conditions and with traffic of a single load, tire pressure, and vehicle configuration. The major experimental variable on WesTrack was intended to be the HMA mixture.

Information from eleven test tracks and road tests was reviewed and summarized. The eleven are identified below:

- AASHO Road Test.
- Pennsylvania State University Test Track.
- Two Mn/Roads.
- San Diego.
- Long-Term Pavement Performance.
- WesTrack.
- Washington State.
- Three U.S. Forest Service test tracks.

Test track and road test geometrics, test section lengths, and the vehicles types used to load the facilities are summarized in Tables 5, 6, and 7, respectively.

2.2.4 Performance-Related Specifications

NCHRP Synthesis of Highway Practice 212, "Performance-Related Specification for Highway Construction and Rehabilitation," (*6*) and the PRS research conducted on HMA pavements (*1,7*) and PCC (*8*) form the background information for this portion of the literature review. Terminology used for the PRS work conducted in this project was mostly obtained from the NCHRP Synthesis authored by Chamberlain (*6*). A more detailed literature review on PRS can be found in Part II of this report and in the Task G Interim Report for this project (*3*).

2.2.5 Pavement Instrumentation

Three different categories of pavement instrumentation were envisioned for the WesTrack project:

- Environmental.
- Subsurface permanent deformation (rutting).
- Pavement strain under axle load.

Early in the project, the SHRP Long-Term Pavement Performance (LTPP) seasonal monitoring instrumentation was selected for recording environmental information and the liquid level gauge developed by the U.S. Forest Service was selected to monitor subsurface permanent deformation. The pavement instrumentation literature review, therefore, focused on strain gage instrumentation for use in measuring pavement surface layer response to wheel load.

The in situ measurement of strains in the HMA layer of a flexible pavement provides information for pavement evaluation and design. The measured strains can be used to investigate the effects of material properties, various types of tires, and tire pressures and load levels on the performance of flexible pavements.

Considerable progress has been made in recent years toward the development of accurate and reliable in situ pavement instrumentation. Strain gages have been used in various pavement field trials in the United States and Europe (*9,10,11*). The following four methods have been used by various investigators to measure the strain in HMA layers in pavements:

- H-gages and strip gages.
- Foil strain gages cemented to or embedded in carrier blocks prepared in the laboratory.
- Foil strain gages cemented to a core extracted from and then returned to a pavement.
- Strain coils.

On the basis of the literature presented in reference 3, the H-gage was selected for use on WesTrack. It should be noted neither the scope nor the budget for WesTrack allowed for the inclusion of a substantial amount of instrumentation on the test track.

2.2.6 Driverless Vehicle Technology

The proposal for the project anticipated that the driverless vehicle technology was to be developed by a subcontractor to the NATC. During the first year of the project, it became apparent the subcontractor would not be able to deliver the system; NATC then undertook the development of the driverless vehicle system used at WesTrack.

The research team investigated the driverless vehicle technologies developed by Cyplex, a private company, and by the UCB in detail. On the basis of this literature review and the internal knowledge base at NATC, a driverless vehicle system was developed with five major components. The literature search was focused on these five areas, as follows:

- Wire-in-road for the vehicle guidance.
- Truck-mounted antennas to guide the vehicle.
- Base station for traffic control and programmed startup and shutdown.
- Steering actuator and control computer in vehicle.
- Data acquisition computer for safety controls and data logging.

The design and integration of these systems is discussed in Section 2.6.

2.3 EXPERIMENT DESIGN

The scope and the objectives of the WesTrack project limited the variables to be studied to those associated with the materials selection, mixture design, and placement of the HMA. The location of the track, the geometric design of the track, the thickness design, and the construction operations all attempted to produce subgrade, fill, base course, and pavement layers as uniform as possible in material properties and thickness. The objective of the project was to evaluate the effect of variations in materials and construction quality of HMA on pavement performance, so all other variables were to be as uniform as possible.

Therefore, the experimental design in this context refers to the design of the partial factorial used for the HMA placed during the original construction of WesTrack. The experimental design for the replacement sections will be discussed in Section 2.3.11. Seven experimental factors related to the HMA surface layer were initially considered in the development of the experimental design:

- Asphalt binder type.
- Aggregate type.
- Aggregate shape and surface texture.
- Aggregate gradation.
- Asphalt binder content.
- In-place air void content.
- HMA thickness.

A discussion follows that defines how these seven factors were considered in the experiment design.

2.3.1 Asphalt Binder Type

The AASHTO Specification MP1 titled Performance Graded (PG) asphalt binder specification system was used to select the asphalt binder(s) to be used on this project. The Superpave (AASHTO MP1) specification for binders was selected to satisfy, in part, the second objective of the project (provide early performance information on the Superpave volumetric design system).

The use of several grades of asphalt binders and both neat and modified binders was considered in the experiment design. When considering different grades of PG binders, the concept was to hold the low temperature designation constant (say –22 or –28) and vary the high temperature grade from say 70 to 64 and 58. The relative performance of neat asphalt and modified asphalt binders of the same PG or perhaps different PG grades was also of interest to the WesTrack team.

Because of the limited size of the project and considering that asphalt binder type is selected in the mixture design process and is not a construction variable, a single asphalt binder was selected. The asphalt binder grade selection process is described later in this report under mixture design and more specifically in WesTrack Technical Report UNR-1 (*12*). The binder selected was a PG 64-22 at the WesTrack location. This asphalt binder meets the high temperature requirement at the 98th percentile level for the Superpave specification. The asphalt binder meets the low temperature Superpave requirement at the 50th percentile level. Since the life of the test track was not expected to exceed 3 years, the use of a somewhat higher than desired “low temperature grade” for this climate was considered adequate. In addition, obtaining a nonmodified PG 64-28 from domestic crude sources was not possible at a reasonable cost for use on this project. The use of a modified asphalt binder for the WesTrack was not desirable because the SHRP research program focus was nonmodified or neat asphalt binders.

2.3.2 Aggregate Type

The original concepts developed for the HMA to be used on WesTrack were based on developing mixture designs from two aggregate sources that had relatively different degrees of sensitivity (as measured by mixture mechanical properties) to asphalt binder content, gradation, and in-place air void content. The sensitivity of the mixtures was to be evaluated during the mixture design process by use of the Hveem stabilometer and volumetric properties. Historically, sensitive mixtures or critical mixtures were defined as HMAs whose stability decreased rapidly with an increase in asphalt binder content.

The two aggregates preliminarily selected for use were a 100 percent crushed, quarried aggregate, and a partially crushed, water deposited aggregate. The 100 percent crushed aggregate was to be used to produce a coarse-graded Superpave mixture that was relatively insensitive (noncritical) to changes in asphalt binder content and other construction variables. The partially crushed aggregate was to be used to produce coarse- and fine-graded Superpave mixtures. The fine-graded mixture was expected to be sensitive to changes in asphalt binder content and other construction variables (critical mixture).

The 100 percent crushed, quarried aggregate was from the central California coast, had been used extensively in central California, and had a good performance history. In addition, this crushed granite aggregate had been used extensively for research purposes at several universities in the United States and on the SHRP project. Unfortunately, a suitable coarse- or fine-graded Superpave volumetric designed mixture could not be obtained with the production from this source. Three laboratories attempted to develop a suitable Superpave mixture design with this aggregate.

The second aggregate selected was a partially crushed, water-deposited gravel from near Dayton, Nevada. The aggregate met the coarse aggregate and fine aggregate angularity requirements of Superpave. This aggregate was selected to produce a mixture that was relatively sensitive to changes in asphalt binder content (sensitive mixture or critical mixture). Suitable Superpave coarse- and fine-graded mixtures were developed with this aggregate. An additional sharp, natural sand (from Wadsworth, Nevada) was used to develop the fine-graded mixture.

Based on the mixture design results, a single aggregate source was used for the test sections placed during the original construction of the test track. Even though only a single aggregate source was used, the coarse- and fine-graded mixtures were respectively expected to satisfy the experiment targets of a sensitive and nonsensitive mixture.

2.3.3 Aggregate Shape and Surface Texture

Aggregate shape and surface texture are essentially defined when a specific aggregate source is selected. Physical properties of the partially crushed, water deposited aggregate and the sharp, natural sand are contained in Sections 2.12.4 and 2.12.5.

2.3.4 Aggregate Gradation

As described above, the original hot-mix design concepts included a coarse-graded 100 percent crushed granite aggregate mixture and a coarse-graded and fine-graded, partially crushed gravel aggregate mixture. Since the 100 percent crushed aggregate could not be used because of mixture design considerations, a third gradation of the partially crushed, gravel aggregate was developed. This third gradation consisted of the same fine gradation as previously described with the addition of 2 percent baghouse fines (minus 0.075-mm [No. 200] material); this mixture was termed “fine plus.”

The three gradations used on the project are shown in Figure 6. The three gradations meet the Superpave gradation requirements and are identified as the “fine-” graded mixture, “fine-plus-” graded mixture, and the “coarse-” graded mixture. The fine-graded mixture has a gradation that plots above the restricted zone and has a relatively large amount of minus 4.75-mm (No. 4) material. The fine plus mixture has a gradation that plots above the restricted zone and has an additional 2 percent minus 0.075-mm (No. 200) material as compared with the fine-graded mixture. The coarse-graded mixture has a gradation that plots below the restricted zone and has a relatively large amount of material retained on the 4.75-mm (No. 4) sieve. Additional details for the mixture designs of these mixtures are contained in Sections 2.12.6 and 2.12.7.

Selection of the fine and coarse gradations allowed for a comparison to be made between the performance of the two extremes of the Superpave gradation band. Selection of the fine plus gradation allowed for the comparison of the effects of additional minus 0.075-mm (No. 200) material (fines control systems including baghouse return systems) on pavement performance.

2.3.5 Asphalt Binder Content

Optimum asphalt binder contents were determined by use of the Superpave volumetric mixture design process for the fine- and coarse-graded mixtures. The “target” asphalt binder content for the fine plus gradation was set at the target value for the fine gradation. The asphalt binder content was varied ±0.7 percent from this optimum asphalt binder content for each of the gradations selected for evaluation. The target asphalt binder content was designated as the “optimum” or medium level, while the asphalt binder contents 0.7 percent below and 0.7 percent above the target values were designated as “low” or “high.”

The range of ± 0.7 percent was selected based on statistical as well as practical considerations. Typical variability for asphalt binder content expressed as standard deviation is 0.3 percent. In order to be reasonably sure that the asphalt contents were statistically and practically different among the three levels (low, medium, and high), a separation of 0.7 percent was considered appropriate. In addition, the range of asphalt binder contents was likely to ensure that differences in rutting and fatigue performance would be obtained in the mixtures.

2.3.6 In-Place Air Void Content

Like asphalt binder content, in-place air void content has a nonlinear effect on HMA performance. In addition, few performance prediction models are available that directly relate asphalt binder content and in-place air void content to rutting and fatigue performance of in-service pavements. Consequently, air void content was assigned a high priority for this experiment.

Three levels of in-place air voids were selected; low, medium, and high. The medium level was selected at 8 percent to represent a typical in-place air void content in pavements in the United States. A low value of 4 percent and a high value of 12 percent were selected to represent expected extremes in in-place air voids as currently experienced in HMA construction. The separation of 4 percent air voids from the target value of 8 percent was considered sufficient (provided these values were obtained) to ensure that statistical differences will exist between the low and medium value sections and the medium and high value sections. Typical standard deviations of in-place air voids are of the order of 1.5 percent.

2.3.7 Hot-Mix Asphalt Thickness

HMA layer thickness is a controllable construction factor that has a major impact on pavement performance. Several prediction models exist that relate HMA thickness to fatigue performance. Consequently, thickness was not to be included as a factor in this experiment.

A single thickness of HMA was selected. The structural section of the pavement was designed to provide fatigue failure for a typical hot-mix at about 3.3 million ESALs. This would theoretically ensure probable fatigue failures within the application of the 10 million ESALs planned for the project. Structural design considerations will be addressed in Section 2.9.

2.3.8 Factorial Design

Based on the selection process described, a full factorial of the combinations would suggest 27 pavement sections as follows ($1 \times 1 \times 1 \times 3 \times 3 \times 3 \times 1$):

- Asphalt binder type, one level.
- Aggregate type, one level.
- Aggregate shape and surface texture, one level.
- Aggregate gradation, three levels.
- Asphalt binder content, three levels.
- In-place air void content, three levels.
- HMA thickness, one level.

This experimental design does not include replicates.

Cost considerations, including track geometrics and trafficking costs, limited the number of available sections to 26. To reduce the 27 sections needed for the full factorial and to provide for replicate sections, two revisions in the experimental plan were considered. Typical construction operation would usually not result in pavements with the following:

- Low asphalt binder contents and low in-place air void contents.
- High asphalt binder contents and high in-place air void contents.

With these practical considerations included in the development of the experimental plan, 6 of the 27 experimental sections in the full factorial plan were eliminated.

Based on input from the project statistician, a minimum of five replicate sections should be considered for inclusion in the experimental plan. The replicate sections were selected such that each mixture contained a replicate section at the optimum asphalt binder content and the medium in-place air void content (8 percent). Two additional replicate sections were included in the fine and fine plus sections. The selected experimental plan is shown in Table 8.

By definition, two test sections are replicates if both have been constructed and tested at the same nominal levels of all design factors. Thus, performance differences between two replicate sections can be attributed solely to the effects of uncontrolled variables. Therefore, replicate sections were included in the experiment plan to provide an indication of section variability and to provide some statistical basis for establishing whether the difference in performance between two experimental sections is significant. The replicate sections for this experiment are the optimum asphalt binder content/medium air void content level cells for each of the three gradations, the low asphalt binder content/high air void content level cell for the fine-graded mixture and the high asphalt binder content/low air void content level cell for the fine plus gradation (Table 8).

2.3.9 Randomization

Despite efforts to provide controls over all one-level factors, there are perhaps hundreds of uncontrolled variables that operate over the time-space environment of any pavement test study. Some of these variables are associated with construction materials and procedures, while others reflect vagaries of the test vehicles and traffic period. Some uncontrolled variations may occur randomly over time and space, while others may be systematic over the course of time (e.g., climatic conditions), and still others may be systematic over the test track site (e.g., subsurface moisture, support conditions and traffic wander).

Randomization of test section construction and testing provides a means for separating design factor effects from the effects of systematic uncontrolled variation. If, for example, the mixing and paving order of hot mixes containing different gradations correspond to a systematic increase in ambient temperature during construction, the performance effect of gradation could perhaps be correlated (confounded) with the performance effects of ambient air temperature during construction. A second possible example of confounded effects could be associated with some uncontrolled variable that produces systematically better performance on one test tangent than on another. For example, all test sections placed on one tangent meet the target construction requirements, while the test sections on the second section do not meet the target construction requirements or, as a second example, the pavement support conditions on one tangent are different than on the second tangent. Thus, when comparing results of test sections from one tangent to the other, construction variability and pavement support conditions dominate the performance rather than the desired material and construction variability dominating it.

For this project, randomization was used to minimize the effects of site location, material variation, and construction variability. Ideally, this randomization would be accomplished by developing a list of all possible combination sequences of numbers 1 through 26 (for each test section) and randomly selecting one combination that would represent the sequence of mixing and placement of the test sections. Unfortunately, there are some practical construction considerations that dictate a more "controlled" randomization process. Thus, the following randomization restrictions were considered:

1. All mixes (for a given pavement layer) using a specific gradation (fine, fine plus, and coarse) should be mixed and placed prior to the use of a second specific gradation.
2. Replicate sections should have at least one intervening section whose asphalt binder content is different from that of the replicate section.

3. For each gradation, the first four mixes should be paved as a block of successive sections in one tangent, while the remaining mixes for the same gradation should be paved as another block of successive sections on the opposite tangent.
4. The two blocks of test sections for a given gradation should occupy different east-west locations in the respective tangents.
5. Paving should be in the direction of traffic (counter-clockwise).
6. The first section of a block of a specific mixture should represent the optimum asphalt binder content and medium in-place air voids.
7. Replicate sections within a given gradation should be placed in separate blocks.

The first restriction allows for construction to proceed at a reasonable rate. Each gradation requires different cold feed settings, aggregates, or both. Changing cold feed settings, or aggregates, or both in the cold feed is somewhat time-consuming and the time for the HMA plant to produce a uniform mixture after these types of changes is considerable and results in the loss of substantial quantities of HMA. Based on these considerations, the paving schedule was set to produce a given mixture gradation on a single day. For example, the fine-graded mixture (all nine sections) was placed on day 1, the fine-plus-graded mixture (all nine sections) was placed on day 2, and the coarse-graded mixture (all eight sections) was placed on day 3.

The second and third restrictions forced the HMA plant to change asphalt binder contents between replicate sections and to place the replicate sections in different paving blocks and, hence different tangents. The replicate sections were produced from HMA obtained from two distinct plant settings instead of the two replicate pavement sections being produced from a single plant production setting. All other test sections placed at the test facility required either a change in HMA plant setting, or changes in roller patterns (to achieve the desired in-place air void contents), or both between each of the sections.

The third restriction was also based on practical construction considerations and the desire to place approximately one-half of the test sections for a given gradation on each of the two tangents. To provide for better uniformity and to reduce the time of construction, four or five test sections (blocks) were placed with a given aggregate gradation successively before changing locations from one tangent to the other.

Restriction four provided assurance that the paving blocks for the three gradations were randomly located along the tangents. The fifth restriction restricted the paving direction to the direction of traffic. Typically, paving proceeds in the direction of travel on conventional construction operations. Traffic was placed on the track in a counter-clockwise direction.

2.3.10 Construction Sequence

Table 9 shows the placement sequence and mixture designation for the 26 test sections at WesTrack. The construction block assignment is shown together with the aggregate gradation, asphalt binder content, in-place air void content, mixture placement sequence, and section number.

2.3.11 Replacement Section Experimental Design

The experimental design for the replacement sections was identical to a portion of the original construction sections and is shown in Table 10. The details of the replacement section materials and construction are discussed in Sections 2.12 and 3.2, respectively. The replacement sections were coarse-graded mixtures and duplicated (on a relative basis) the asphalt binder content and in-place air void content of the coarse-graded mixtures placed during the original construction. The section numbers used to describe the coarse-graded mixtures assumed that section 1 of the originally constructed track would become section 31 of the replacement section track. Thus, section 5 (an original construction coarse-graded mixture) became section 35 of the replacement sections.

2.4 SITE EVALUATION

Three issues were addressed during the site evaluation portion of this study:

- Effects of prevailing climate on the performance of the pavement sections.
- Impact of the test track on the environment.
- Potential for flooding.

Specific information on the evaluation of the soil support at the site is covered in Section 2.8.

2.4.1 Climate

The AASHO Road Test conducted between 1959 and 1962 was the most comprehensive closed-loop, traffic controlled road test experiment. The AASHO Road Test was targeted at evaluating the performance of pavement sections built with different materials and at different thicknesses. Unfortunately, one of the problems associated with analyzing data from the AASHO Road Test was the impact that seasonal changes in subgrade soil strength had on the deterioration of the pavement sections. For the HMA pavement sections, most of the pavement sections “failed” during one of the two spring-thaw periods. This behavior made analysis of the data from a traffic loading effects standpoint much more complicated and the findings uncertain.

The climate at the WesTrack test site is relatively mild and is suitable for conducting year-round trafficking. The test site has an annual precipitation of less than 100 mm (4 in.) per year and no annual subgrade soil freeze-thaw conditions are expected. The yearly average daytime temperature is 21°C (69°F) and humidity is typically below 20 to 30 percent. Extreme daytime high temperatures of 40°C (104°F) and low temperatures of –20°C (–4°F) could reasonably be expected during the conduct of the WesTrack project.

The dry, hot summer months at the test site ensured nearly ideal construction conditions for the test track. Moisture conditions for the subgrade, engineering fill, and base course compaction were relatively easy to control in this climate. The most difficult problem was the relatively rapid drying during the hot, somewhat windy afternoons during the summer. The hot daytime temperatures at the test site during the summer provided ample time for compaction of the HMA.

2.4.2 Environmental Impact

The NATC, the site of the test track, was established in 1957 in Carson City, Nevada. In 1969, a proving ground site was established 45 km (28 mi) east of Carson City. NATC is one of the largest independent proving grounds in the world. NATC has 1,400 hectares (3,500 acres) deeded proving ground along 12 km (7.5 mi) of the Carson River. The main proving ground area is intersected by the Carson River and Nevada State Route 2B (Fort Churchill Road). In addition, NATC has 392,000 leased hectares (967,000 acres).

The application for the construction and environmental permits to construct the WesTrack project was largely handled by NATC. The information presented to the governing jurisdiction, Lyon County, included the following summary of the plans for the track and environmental impact issues:

- The site has little vegetation other than varied sagebrush. The proposed test track location qualifies as a "categorical exclusion" as defined by 40 CFR 1508.4; Department of Transportation guidelines as outlined in 23 CFR 771.117 were used for that determination. Specifically, locating the test track on this site will not induce an impact to planned growth or land use for the area; require the relocation of people; have an impact on any natural, cultural, recreational, historic or other resource; involve significant air, noise, or water quality impacts; have an impact on travel patterns; or otherwise have any significant environmental impact. No "unusual environmental circumstances," such as a significant environmental impact, a substantial controversy on environmental grounds, significant impact on historical properties, or any inconsistency with federal, state or local law, requirement, or administrative determination relating to the environmental aspects of the proposed test track exists.
- A chain link fence installed around the test track will keep out wildlife and trespassers. Also, the available large land mass is an added safety factor to ensure the overall safety of the program. NATC's location guarantees that security and safety will not be compromised and that vehicles can be run 24 hours per day, 365 days per year. Because of the Department of Defense work NATC performs, a security procedure is already in place that further limits access to the site 24 hours per day.

Overall, considering the existing use of the site as a proving ground, construction of the track did not pose any additional environmental impact. Furthermore, the site is located in a remote, sparsely populated area, so the track's operation would not affect neighbors.

2.4.3 Flood Potential and Risk

A portion of the test track lies within zone A of the Carson River floodplain as shown on the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Panel 320029 0155C, dated September 30, 1992. This zone on the FIRM panel shows an approximate 100-year floodplain for this portion of the Carson River; however, the water surface elevations have not been established and a regulatory floodway has not been defined. The regulatory floodway would define the maximum amount of encroachment that can take place in the floodplain and define the portion of the floodplain that must be left open for the conveyance of the 100-year flow.

The test track lies in and falls under the jurisdiction of Lyon County, Nevada. The County Public Works Department did not require an engineering analysis to examine the impacts of the project on the floodplain and surrounding properties.

The test track was sited as far to the north and west of the river as possible. The nearest the track centerline lies to the river is 85 m (280 ft). The track elevation is approximately 0.6 m (2 ft) above the original groundline. Review of the peak annual discharges for the Carson River near Fort Churchill, Nevada, (which is a short distance downstream from the project site) indicates that a flow of 430 m^3/s (15,300 ft^3/s) was recorded in 1963 and a flow of 470 m^3/s (16,600 ft^3/s) in 1986. Those two discharges are the largest of record dating back to 1911. The regulatory peak 100-year discharge for the Carson River at Dayton (approximately 26 km (16 mi) upstream from the project site) is 1,020 m^3/s (36,000 ft^3/s). The 50-year and 10-year regulatory peak discharges are 665 m^3/s (23,500 ft^3/s) and 215 m^3/s (7,600 ft^3/s), respectively. Table 11 summarizes the peak discharges at the Fort Churchill gauge.

Detailed information was not available, and more specifics were required regarding the extent of the floodplain, accuracy of the floodplain currently mapped, depth of flow, velocities, and recommendations on the type of protection needed to protect the track from inundation. Based on current information, some potential did exist for flooding of the track. Discussions with Mr. Henry C. Hodges, Sr., resident of the immediate area since 1969, indicated that the high flow of 1986 would have inundated a portion of the track's original ground, but the track ground would not have been in the river's "regulatory floodway" (i.e., fast moving water). The 1986 flow was somewhat exaggerated because NATC had a suspension bridge across the river just upstream of the test track area that collected a large amount of debris before it failed under the pressure of the flood waters.

Figure 7 provides a summary of historic peak discharges with their approximate recurrence interval and a discharge frequency curve. Based on a review of the flow records for the Carson River near Fort Churchill, a summary was prepared to compare the highest recorded peak discharges with the discharge frequency curve published by FEMA. The historic flow records were obtained from U.S. Geologic Survey (USGS) records for the USGS gauge near Fort Churchill, just downstream of the project site. The discharge frequency curve was developed by FEMA for the Lyon County Flood Insurance Study. FEMA discharges are the regulatory discharges which must be used by Lyon County for floodplain management purposes until revised by FEMA on the basis of better technical information.

Although the circumstances surrounding each flood are different, the data indicated there was a 3 in 100 chance that the peak discharge rates recorded for the 1986 flood would recur. Given this probability of recurrence, there was a chance that water would encroach the south side of the test track and trafficking would have to be halted until the subgrade dried out.

Winter 1994–1995 and spring 1995 were wet by northern Nevada standards and did create some construction delays at the track during the compaction of the subgrade and placement of the engineering fill. Water levels in the Carson River did not reach flood stage at the site; however, a low level berm was placed between the river and the test track site. Water did not overflow the berm during that period.

A record flood occurred during winter 1995–1996 along the Carson River. This flood was in excess of a 100-year flood. During the flood, slow moving water encroached upon the track site and flowed from the outside to the inside of the track and from the inside to the outside of the track. Although a portion of the pavement shoulder along the southeast corner washed away during the flood, none of the HMA pavement or supporting layers themselves were damaged. The traffic was removed from the track during this period and for a period after the flood. Falling weight deflections were used to determine when traffic could be resumed on the track.

The winter 1995–1996 flood was typical of the major floods in the Sierra Nevada mountain watershed when early winter, heavy snows at both low and high elevations were followed by 6 to 12 days of relatively steady warm rains; the snow pack melts significantly. The results have been major floods in this watershed in the 1930s, 1950s, 1960s, 1980s, and now the 1990s.

2.5 GEOMETRIC DESIGN

This section of the report relates the considerations associated with the geometric design of WesTrack. The geometric design refers to the three-dimensional features of the track. These features include the cross section (lanes, shoulders, roadside slopes, and clear area) intersections (off and on ramps for the vehicle maintenance area) and the horizontal and vertical alignment of the test track.

2.5.1 Design Assumptions

A number of basic design controls and criteria governed the manner in which the test track was designed. Design speed was established as 64 km/h (40 mi/h) for the test vehicles. The selection of this speed was a compromise among the desire to operate traffic as near as possible to highway speed, the sensitivity of the vehicle control systems, construction cost, and traffic operating costs. Operation of the trucks at higher speeds would require larger radius turnarounds (assuming a superelevation of 18 percent, which was considered the maximum for conventional construction equipment use) at the end of the test sections tangents. The larger radius curves would increase the overall length of the track and thereby increase the construction and vehicle operating costs. Higher operating speeds for the loading vehicles would require more sensitive (quicker responding) vehicle control systems and would likely increase the costs for the driverless vehicles.

Spiral curves were used to allow for a smooth transition for the driverless vehicles into the curves from the tangent sections. Spiral curves provide a less abrupt transition from tangents to circular curves and are widely used for horizontal alignment of rails. This less abrupt transition afforded by the spiral curves also provided less stringent requirements on the automatic vehicle control systems.

Since the terrain at the site is very flat, the complications that can result from the integration of vertical alignment and horizontal alignment was not an issue. The natural ground at the site has less than 1 m (3.0 ft) fall from south to north (slightly greater than 0.1 percent) and less than 0.5 m (1.5 ft) fall from west to east (slightly greater than 0.2 percent). The design relied on the normal cross-slope for drainage.

2.5.2 Test Section Lengths

Several factors were considered in identifying an optimum length for each experimental test section on the WesTrack project:

1. Vehicle dynamics.
2. Construction uniformity.
3. Performance monitoring.
4. Destructive sampling.
5. Costs.

Based on a preliminary analysis of the first four items, the desirable length for each section should be about 100 m (328 ft). However, the high cost associated with construction and operation of a test track consisting of 26 100-m (328-ft) sections was considered excessive. Thus, it was necessary for the design team to determine the technical feasibility of the use of shorter test sections.

2.5.2.1 Vehicle Dynamics

As truck loading progresses and the pavement sections begin to deteriorate, there is a strong possibility that some test sections will deteriorate faster than others. There is also the potential for discrete bumps developing at the cold construction joints at the beginning and end of every section. As the test sections start to develop different levels of roughness and as the cold joint bump amplitudes increase, it is likely that this roughness will excite dynamics within the vehicle. If the effects of the vehicle dynamics are not properly considered in the track design, roughness from one section could be carried to an adjacent section, resulting in "sympathetic failures." This phenomenon can be addressed by allowing a transition zone of significant length between the test sections so that the vehicle dynamics have time to dampen or stabilize before loading the actual performance monitoring section.

To define the length of transition required to dampen vehicle energetics excited by a cold joint bump or section of high amplitude roughness, a representative "triple" vehicle combination was leased and instrumented with accelerometers. The time (or distance based on 64 km/h (40 mi/hr)) required to dampen unsprung and sprung mass oscillations was measured.

In summary, with the axles loaded to 89 kN (20 kips), the suspension effects are significantly attenuated and dampen quickly. Figure 8 shows the worst case measured event, which was a 45-mm (1¾-in.) high bump impacted at 64 km/h (40 mi/hr). The time histories from the top of Figure 8 are vehicle speed (mph), bumper or sprung mass vertical acceleration (g's), front axle vertical acceleration (leaf spring g's) and intermediate axle vertical acceleration (air ride g's). At 64 km/h (40 mi/hr), it required approximately 0.45 seconds for the suspension energy to dampen to 95 percent of the maximum acceleration variation. This translates to 8 m (25 ft) of vehicle travel. Based on these results, a 25-m (82-ft) transition zone between test sections was determined to be sufficient for the vehicle dynamics (arising from either a bump at the cold joint or localized roughness in the "upstream" section) to dampen.

In addition to localized roughness and cold joint bumps, longer wavelength amplitudes (i.e., greater than 15-m [50-ft] wavelengths) were potentially present in the track at the time of construction or as pavement loading progressed. These longer wavelengths could also affect vehicle sprung mass oscillations. Because these long wavelength, low frequency vehicle oscillations could also result in sympathetic failures in the test sections, these vehicle frequencies were monitored. The vehicle instrumentation, shown in Table 12, was installed on one tractor to allow correlation of the pavement performance measurements with vehicle performance data.

These data were recorded initially and at 4-week intervals to correspond to the pavement performance and pavement strain gauge measurements. Approximately ten vehicle passes at 64 km/h (40 mi/h) per test section were recorded and processed (i.e., 30 min of data).

2.5.2.2 Construction Uniformity

A transition length is not only required to address the technical issue associated with vehicle dynamics, but also necessary so that the contractor can have adequate distance after a cold start to establish mix uniformity. The Performance-Related Specification Phase II study (*1*) recommended 12 m (40 ft), which was probably based on the 12-m transition length used at the AASHO Road Test. Unlike the AASHO Road Test (which had only one HMA design), the WesTrack Project studied the performance of a number of different mix combinations, each of which required special attention to laydown and compaction procedures. Although a transition length greater than 30 m (98.4 ft) was desirable to help achieve mix uniformity, the selection of a 25-m (82-ft) length to account for vehicle dynamics was judged to be satisfactory for developing a uniform mix. Three independent construction operations were planned to achieve better mix uniformity and to avoid roughness that might be induced at a cold joint.

- A mat reference was used with the paver to provide the smoothest joint possible between adjacent sections.
- The mix was laid and compacted approximately 3 m (10 ft) beyond the end of a given test section. Most of this length was then broken and removed prior to placement of the next test section. Removal was carried out in a manner that least disturbed the underlying base material.
- After construction, a profilograph was used to measure the smoothness, and diamond grinding was used to remove any waves or bumps.

2.5.2.3 Performance Monitoring

The Performance-Related Specification Phase II study (*1*) recommended a length of 46 m (150 ft) for monitoring the performance of the HMA in each individual section. This length was based on the general opinion that the 30.5-m (100-ft) length used for AASHO Road Test sections was not adequate. The 152.5-m (500-ft) length used for Global Positioning

System (GPS) sections in the LTPP program was based primarily on the ride quality requirement to measure roughness for wavelengths up to 76 m (250 ft). If ride quality were not a requirement, the LTPP sections might have been shorter to control longitudinal variability of soil, structure and performance within a section. Forty meters (131.2 ft) was selected for performance monitoring in the WesTrack project because of cost and the team's concern over the treatment of longitudinal variability. The 40-m (131.2-ft) length is longer than the 30.5-m (100-ft) length used at the AASHO Road Test; unlike the SHRP study, accounting for long wavelength roughness was not considered essential in this study.

2.5.2.4 Destructive Sampling

According to the plan for field sampling and laboratory testing, numerous core and slab specimens were to be collected during the loading period. These specimens were to be obtained from both wheelpaths and between the wheelpaths, but relatively close together. Five meters (16.4 ft) of pavement length were judged to be satisfactory for destructive sampling. To keep subsequent distress downstream of the sampling area from affecting future sampling locations, all sampling commenced near the end of the section and progressed upstream. Quality repairs in the sampled area were performed to reduce the potential for sympathetic failure in a downstream section.

2.5.2.5 Summary

The test section geometrics are shown in Figure 11. The transition is 25 m (82 ft), the test area 40 m (132 ft), the destructive sampling 5 m (16 ft), and the total length 70 m (230 ft).

2.5.3 Cross Section

The test track construction plan involved the use of the existing subgrade material, a base course, and a variety of HMA mixtures. Maximum uniformity of the subgrade soil and base course in terms of structural load-carrying capacity and thickness were critical to the success of the project. Consequently, a cross section that provided a high-level, uniform foundation was a key consideration in the structural and geometric design.

The HMA was constructed with different aggregate, gradations, asphalt binder contents and in-place air voids as part of the experimental design; however, the thickness of the HMA remained constant for all sections.

Three separate cross sections were considered for the test track (Figure 9). The first section (cross section A) was the one initially proposed and served as the basis for the original cost estimate. It consisted of two separate travel lanes, one that served as the actual experimental test lane and the other that served as both a trial construction lane and a traffic bypass lane. These two lanes were separated by a median barrier. One advantage of this horizontal configuration was that it permits a "practice" placement of the mix before it is constructed on the actual test lane. A second was that its lane separation permits the track to continue to be trafficked while one side is closed for repair or testing. The disadvantage of this section was cost.

The second section (cross section B) considered was the simplest. It involved the elimination of the trial construction/bypass lane. The advantage was the major reduction in cost. The disadvantage was the fact that the contractor could no longer have a "practice run" at achieving the desired mix characteristics before placing the final experimental section. Given that 21 different asphalt mixes were being evaluated, this was considered a major disadvantage.

The third section (cross section C) also represented a step down from the original section in that the median and median barrier between the trial and test lanes were eliminated. The main impact of this is that the trial lane can no longer be used as a bypass lane while one side of the track is being sampled, surveyed, or rehabilitated. Thus, track loading would have to be discontinued during any of these activities. Because projected track downtimes related to these activities were minimal (roughly 10 percent of the time) and because unbalanced loading (loading one side of the track while not loading the other) is generally undesirable from a performance evaluation standpoint, sacrificing the bypass lane was not considered critical. The reduction in construction cost, however, was significant.

After a review on February 2–3, 1995, the consensus of both the WesTrack team and FHWA technical panel was to pursue cross section C and realize the cost savings associated with the overall reduced width while still retaining the trial construction benefit of the trial lane. The final design and selection of the various layer thicknesses are presented in Section 2.9 of this report.

2.5.4 Turnarounds

The final geometrics were based on analysis of the pavement loading, comments from the FHWA technical panel, driverless vehicle requirements, and suspension and tire wear considerations. It was determined that a superelevation rate (e) of 0.18 and side-friction factor (f) of 0.05 were optimal for developing the final curve design. These values are based on suspension dynamics, trailer off-tracking, and irregular tire treadwear issues. Secondly, the lateral control system for the driverless trucks has improved control capabilities at low lateral acceleration.

Based on these considerations and previous limitations, the curve design was set in accordance with the following design parameters:

- The spiral curve design placed 50 percent of the superelevation runout in the curve. (This compares with 33 percent for a conventional superelevation design.)
- The superelevation rate was set at e = 0.18 because the track is not in a location where snow and ice conditions prevail. (Also, asphalt pavers have been used in the past at this superelevation rate without any need for ballast or equipment modifications.)
- The design included a vehicle transition length in the tangent after exiting each curve of 15 m (50 ft) to allow vehicle speed and dynamics to stabilize.

Although the spiral curve design is slightly more aggressive than AASHTO highway design guidelines, the track is a dedicated facility and has a dedicated speed; the spiral design will provide a smooth change from the tangent section to the circular curve, and vice versa.

2.5.5 Summary

The selected track length is summarized as follows:

tangents	2 tangents × 13 sections × 70 m (230 ft) per section = 1,820 m (5,980 ft).
spiral transitions	4 spirals × 46 m (150 ft) per transition = 184 m (600 ft).
alignment transitions	2 transitions × 15 m (50 ft) per transition = 30 m (100 ft).
horizontal curves	2 curves × 398.5 m (1,307 ft) = 797 m (2,614 ft).

The total length of the track is 2,831 m or 2.8 km (9,288 ft or 1.76 mi).

The layout and plan view of the test track are shown in Figure 10. The individual test section dimensions are shown in Figure 11.

The profile of the track was set above existing ground with the inside edge of the shoulder measured at the top of the subgrade set near existing ground elevation after the stripping of vegetation. With a 2 percent cross-slope, the track section results in an outside edge of pavement (measured at the outside edge of the shoulder on top of the subgrade) that ranges between 0.5 m (1.5 ft) and 1 m (3.0 ft) higher than adjacent natural ground. Most of the fill for construction of the track subgrade and engineering fill was borrowed from the inside of the track. This borrow area provided for a vehicle safety area (run-off) as well as for drainage collection and conveyance.

Because of the flat profile of the natural ground, vertical curves were not needed. The 11-m (36-ft) track cross section consists of the following elements as described from the outside of the track toward the inside of the track and as shown in Figures 9 (cross section C) and 11:

• Outside shoulder	1.8 m (6 ft) gravel and 1.2 m (4 ft) HMA.
• Test lane	3.7 m (12 ft).
• Trial lane	3.7 m (12 ft).
• Inside shoulder	0.6 m gravel (2 ft).

Roadside safety slopes beyond the 11-m (36-ft) cross section were provided. Beyond the outside aggregate base shoulder, a 6 : 1 graded slope was used to original ground level. On the inside of the track, a 6 : 1 to downslope hinging off the aggregate base shoulder (for an approximate distance of 16 m [52.5 ft]) was used to “catch” the existing ground. WesTrack Technical Report NCE-2 contains finalized geometric design information including a plan view and pavement cross sections (*13*).

2.6 DRIVERLESS VEHICLE DEVELOPMENT

2.6.1 Introduction

The pavement loading was accomplished using four triple-trailer vehicle combinations (Figure 12). Four conventional tandem axle class 8 tractors were used to pull the trailers. This configuration provided a total of 10.48 ESALs per truck/trailers pass (Figure 13).

NATC developed autonomous (driverless) vehicle technology to allow near-continuous vehicle operation in an otherwise monotonous driving environment. Four triple-trailer combinations were designed, developed, and certified to operate on the track up to 22 hours per day, 7 days per week to meet the loading goals.

The triple-trailer vehicle combinations were operated an average of 15 hours per day over the 2½-year period.

2.6.2 Driverless Vehicle Features

The major features of the driverless vehicles are briefly discussed below. The block diagram shown in Figure 14 shows the multiple computers and redundant control systems integrated into the truck-trailer combination for fail-safe operation and continuous safety monitoring.

To aid in the electronic control of the trucks through the driverless vehicle system, the trucks were equipped with a Detroit Diesel Series 60, turbo-charged, electronically-controlled engine. Twin Disc automatic transmissions in each truck allowed electronic control of the transmission. The trucks and trailers were equipped with a Haldex Brake Systems’ Anti-Lock Brake System (ABS) and electronic brake valve for electronic control of the brake system. The trucks were equipped with 295/75R22.5 Goodyear tires. The cold inflation pressure of the tires was set at 690 kPa (100 lbf/in^2). Each axle of the vehicle train was loaded to 89 kN (20,000 lb), except for the front axle which was 53 kN (12,000 lb). The test speed around the track was 64 km/h (40 mi/h).

2.6.2.1 *Guide-by-Wire System*

The driverless vehicle system used a guide-by-wire system for the lateral and longitudinal control of the trucks. Additionally, every truck control system was designed with a backup in the event of primary system failure. All track and control room systems were connected to uninterruptible power supplies in the event of mainline power loss.

Primary and backup wires, buried under the asphalt, gave a continuous feedback signal to the steering controller to guide the trucks. Audio amplifiers powered the two continuous wire loops installed around the track. Each vehicle was equipped with guidance antennas mounted to the front bumper to acquire the guide tones emitted by the redundant wires (Figure 15). The vehicle antennas were capable of reading either primary or alternate wire paths. A Proportional Integral Differential (PID) control loop was used within the control system to guide the trucks. A robust stepper motor (Figure 16) was connected to the steering gear box to control steering based on feedback from the antenna and the error signal generated when the antenna was displaced from the center of the wire.

2.6.2.2 *Traffic Control*

The four trucks were controlled and monitored from a control room located beside the test track. Computers within the control room (Figure 17) initiated the starting and stopping of the vehicles and regulated vehicle spacing and speed for traffic control purposes.

The traffic control and longitudinal control used radio frequency (RF) serial modems to communicate with the four trucks. Each truck had a RF serial modem for sending and receiving information packets to the control room. The control room had four RF serial modems for sending and receiving information packets from each truck. Each modem was operated on a separate frequency. As a final judge of the vehicle spacing, a Differential Global Positioning System (DGPS) independently monitored the position of the trucks and provided a fail-safe input to the traffic control computer.

Traffic was managed by referencing very-high-resolution odometer positions that the trucks reported twice per second to the control room. The trucks also reported their odometer positions once per lap when they passed over a radio beacon located on the track surface. Truck spacing control was maintained by adjusting the directed speed of the individual trucks so that they remained equally spaced around the track. This control system resided in the first of the two computers; it was backed up by a second computer, which used the DGPS data reported by the trucks once per second. The second computer compared the DGPS data with the odometer-generated positions and used both to continuously verify that the spacing tolerance between trucks was not violated.

In addition to controlling the truck spacing, the traffic management computer periodically, or on manual request, commanded the vehicle equipped with strain gauges and accelerometers to take high-resolution dynamics data as they passed strain gauges in the pavement at specific locations along the track.

This computer also provided a graphical display of the steering deviation of the trucks relative to the guide wire. For each truck, the positions of the tractor and the last trailer were displayed so that the tracking performance of each triple-trailer combination could be examined as it circled the track.

The safety-monitoring computer verified the truck spacing from the DGPS data and interfaced with a lock board that contained safety keys for all of the trucks. The truck control authority was passed to the control system only when the safety key was inserted and locked. This provided a hardware method of removing a truck from automatic operation. When the truck was in maintenance, the safety key for that truck was removed from the lock board and the commands could not be sent to that truck.

In addition to the traffic management and safety computers, vehicle-monitoring computers were located in the control room. These computers continuously verified that all truck and control systems were operating properly. The screens of the vehicle-monitoring computers were displayed in a “red-yellow-green” format. If a parameter display was green, it indicated that the system was operating well within the tolerances set for that system. If a parameter was yellow, the system was getting close to the upper or lower limit. If the parameter was red, all trucks were stopped automatically, and the control room operator received an instant, visual reading of the problem.

2.6.2.3 *On-Board Control Computer*

Each vehicle had two computers (Figure 18), one for vehicle control and one for vehicle monitoring. The computers were located in the sleeper of the truck in a shock-mounted cabinet. The guidance and steering control activated the steering actuator connected to the steering system. The monitoring computer checked truck health that included more than 160 parameters normally evaluated by a driver.

The control room operator had one computer for each truck that displayed the status of the truck in an easy-to-read format and had diagnostics to aid in monitoring and correcting critical control parameters. A vehicle-monitoring computer checked the critical components and provided decisions to the control computer on the status of the vehicles. If a critical parameter was out of bounds, the vehicle-monitoring computer transmitted a shutdown signal to the control computer.

2.6.2.4 *Speed Control*

The control of the throttle, engine, and transmission were automated by using advanced electronics on the engine and an automatic transmission electronic control unit (Figure 19). The brake interface was built upon Haldex Brake Systems’ electronic brake valve and controlling circuitry. Haldex Brake Systems' ABS were installed on the truck and all trailers to allow controlled stopping during normal and emergency braking.

2.6.2.5 Brake Controls

The final set of truck controls (for the brakes) performed three separate functions. The first was for routine stops and was performed by computer control of a conventional air-brake system. A proportional valve provided an air pressure that was proportional to an analog command from the control system. In addition to the proportional valve, a parallel solenoid valve conveyed the full-system air pressure to the brakes in the event of a detected failure. The secondary computer in the truck controlled this solenoid valve. The solenoid valve was normally open, and it had to be continuously energized to keep the valve closed and to prevent the brakes from being applied. In the event of a loss of power to the control system, the solenoid valve opened, and the brakes were applied. This system was also used for several emergency-braking scenarios.

The second function of the braking interface was to control the whip of the triple-trailers. Steering adjustments, if severe and rapid, can be amplified through the length of the trailers and potentially result in a loss of control. Such adjustments might occur in the event of a steering-tire blowout. To control whip, the brakes on the last axle of the third trailer were applied by a solenoid valve located on the third trailer and were controlled by one of the computers in the truck.

The final function of the brake controls was anti-lock braking. An ABS was included in the trucks to ensure their stability under emergency braking conditions. It operated on all of the truck axles except the steering axles, where the brakes had been disconnected. The ABS performed well throughout the trafficking period. The system was tested in many hard-braking modes, including application of full air pressure to the service brakes through the backup solenoid valve. This last test produced a very short stop, but, as in all of the other tests, the tires did not lock.

2.6.2.6 Lateral and Longitudinal Location

WesTrack implemented a unique pavement measurement capability not available at any other pavement research facility. Through the driverless vehicle controls, the lateral and longitudinal location of the truck was precisely defined at all locations around the track. This allowed the longitudinal location of the trucks to be defined within 50 mm (2 in.) of any measurement sensor installed in the pavement. This exact alignment included all the phase delays associated with the data acquisition computers on the truck and data acquisition computers for the track.

2.6.2.7 Vehicle Instrumentation

One of the triple-trailer vehicle combinations was instrumented at each axle end with accelerometers and shear strain gages. This instrumentation allowed for investigation of vehicle dynamics with respect to stationing along the track and measurement of dynamic loading as each truck axle passed over the pavement strain gages. The alignment and correlation of pavement strains with dynamic forces on the vehicle provide a unique investigation and model validation tool for future research.

2.6.2.8 Truck Health Data

To be sure that the trucks operated safely and without mechanical or electrical problems, the health of the systems in each truck was checked and evaluated every $^1/_2$ sec. Data were acquired from four sources aboard the truck and then transmitted to the control room. The Detroit Diesel electronic control (DDEC) from the Series 60 engines provided engine data and some vehicle data over an SAE J1708 data bus. The system monitored this bus and extracted data of interest. Additional pressure, temperature, and voltage sensors were installed in the vehicle to measure truck parameters, such as cab temperature and power steering fluid temperature, that are not monitored by the DDEC III system. One of the trucks was equipped with a set of accelerometers and strain gauges to measure the forces on the axles of the truck. These accelerometers and strain gauges had very stringent signal-processing requirements to maintain precise timing alignment with other pavement signals.

The data from the four sources were accumulated by the two computers in each truck, combined with the vehicle control information, and sent back to the control room over the spread spectrum RF modem links. The data rate was 9600 baud. In the control room, these data were captured by a data acquisition computer and were downloaded once a week to CD-ROMs for permanent storage.

The dual guidance antennas were mounted in a wooden housing attached to the front bumper of each of the four test vehicles.

2.7 PAVEMENT INSTRUMENTATION

As discussed previously, the WesTrack installed a limited amount of pavement instrumentation to monitor climate conditions, pavement temperatures and moisture conditions, strains on the underside of the HMA, and subsurface permanent deformation. The type of instrumentation installed at WesTrack is summarized below. More detailed information on pavement instrumentation is available in reference 3. Information collected with this instrumentation is discussed in Section 4.2.

2.7.1 LTPP Weather Station

An LTPP-type weather station was installed at WesTrack near the vehicle staging and maintenance area. The equipment was used extensively in the SHRP LTPP program to monitor climate at the specific pavement studies (SPS) test sites. The equipment records the following information:

- Air temperature.
- Relative humidity.
- Wind speed.
- Wind directions.
- Solar radiation.
- Precipitation (water equivalent).

The equipment was installed by WesTrack staff after attending installation, operation, and maintenance training sessions.

2.7.2 LTPP Seasonal Instrumentation

Instrumentation packages developed by the SHRP LTPP program for measurement of moisture, temperature, and frost profiles in pavement sections were placed at two locations at WesTrack. This equipment is used extensively by LTPP on their Seasonal Monitoring Program sections. The equipment was installed at the edge of the test lane for section 12 and section 25. The following sensors were placed at each of these two locations:

- 10 time domain reflectometer (TDR) probes.
- 18 probes to measure pavement surface temperature.
- 35 electrical resistivity probes to measure frost penetration.

Data from the temperature and resistivity probes were continuously recorded using a data logger in a cabinet at the test site. Data from the TDR probes were recorded at approximately 2-week intervals.

A piezometer/observation well was installed near the SHRP seasonal instrumentation package on the south tangent to monitor the elevation of the groundwater table. A second HMA temperature sensor thermocouple tree was installed in section 19. Readings were monitored continuously at 12-mm (0.5-in.) intervals in the pavement.

2.7.3 Strain Gages

As a loaded vehicle passed over a pavement section, strains in the pavement were measured using strain gages. H-gages Model No. KM-120-120-H2-11-W1M3 were used for the strain measurements. The following modifications were made by the UNR staff prior to the installation of the gages at WesTrack.

- Metal plates with 0.8-mm (0.03-in.) thickness were attached to the top and bottom of each strain gage to protect the strain gage strips from bending stresses.
- The strain gage strips attached to the metal plates were enclosed in watertight plastic to protect against moisture damage.
- Anchors were attached to the ends of the strips to allow the strain gages to be secured to the asphalt concrete (AC) layer.
- The lead wires on each gage were extended to allow for field installation and connection to the signal conditioning and data acquisition station alongside the track.

The strain gage calibration was done by UNR. The calibration values were incorporated into the data acquisition software so that the output would be in microstrains.

The field installation process involved the laying down of the gages, their protection from construction equipment, and connection of the gage wires to the junction box. UNR installed 260 strain gages (10 gages per test section (Figures 20 and 21) and 26 junction boxes.

On track sections 1, 2, 3, 4, 14, 15, 16, and 17, the outer strain gages were placed 150 mm (6 in.) to the right of the centerline and 450 mm (18 in.) inward from the inside shoulder edge.

On track sections 5, 6, 7, 8, 9, 10, 11, 12, 13, 18, 19, 20, 21, 22, 23, 24, 25, and 26, the outer strain gages were placed 450 mm (18 in.) to the right of the centerline and 450 mm (18 in.) inward from the inside shoulder edge.

2.7.4 Data Structure

For each truck/trailer pass, 4,096 data points were collected per strain gage. This information was filtered through a moving average and reduced to 251 data points per pass. The selection of the moving average range is such that "peaks and valleys" were captured without losing resolution in the data.

Pavement strain gage data were collected once each month during the months of June through September 1996. Although the strain gages responded well at the early stage of measurement, all gages may not have responded at all times. This could be due to truck wander or failure of the gages. Because of the severe environment and rapid failure of the test track, some of the gages failed early. The WesTrack database provides the data collected without excluding responses from bad gages.

2.7.5 Temperature in Hot-Mix Asphalt

Thermocouple temperature gages were installed in section 19 after construction as shown in Figures 20 and 21.

Temperature data were collected at depths of 12.7 mm (0.5 in.), 38.1 mm (1.5 in.), 88.9 mm (3.5 in.), 114.3 mm (4.5 in.), and 139.7 mm (5.5 in.) within the HMA layer. The temperature data was recorded continuously. The WesTrack database contains temperature profile information at various time intervals.

2.7.6 Subsurface Permanent Deformation

A device developed by the U.S. Forest Service was installed to measure permanent deformation at the interface of the engineered fill and base course and at the interface of the base course and HMA. The device is referred to by several names but is commonly called a liquid level gage.

The liquid level gage measures elevation differences with respect to a fixed datum, which is a temporary bench mark located along the side of the pavement. The elevation is measured with a temperature-compensated differential pressure transducer which moves through a 25-mm (1.0-in.) outside diameter reinforced hydraulic hose installed in the pavement. The pressure transducer is used to calculate vertical elevation differences based on a column of liquid pressure head. Measurements are recorded using a laptop computer. Two hoses are installed in each section, one each at the interfaces between the engineered fill and base course and between the base course and HMA. Under ideal conditions, the liquid level gage has an accuracy of 0.5 mm (0.02 in.) with a normal field use accuracy of approximately 2.5 mm (0.1 in.).

2.8 GEOTECHNICAL INVESTIGATION

The track site is a flat area of land adjacent to the Carson River. Most of the area considered for the site of the test track (approximately 1.8-km [1.1-mi] long and 0.3-km [0.2-mi] wide) was currently or previously used for agricultural purposes and was currently or previously irrigated. A geotechnical investigation and nondestructive test program was undertaken to characterize the soil conditions at the site to locate the track within the available area of the most uniform subgrade conditions, and to provide some basis for the structural design of the pavement section. Test pits, borings, and falling weight deflectometer (FWD) testing were performed at the site as discussed below.

2.8.1 Test Pits

In late October 1994, ten test pits were dug in locations uniformly spaced along both sides of the proposed site of the track. The test pits were excavated to a depth of 1.5 m (5 ft) using a backhoe. The soil profile to this depth was logged and bulk samples were obtained for gradation analysis, Atterberg Limits, soil classification, in situ moisture determination, and compaction density. The results of the testing are displayed in Figure 22.

Consistent with the depositional process of the Carson River, the soils at the site consisted of varying proportions and blends of fine-grained clays, sands, and silts. In situ moisture contents ranged from 4 to 22 percent with the higher moisture contents found in the soils nearest the areas previously irrigated. The optimum moisture content and maximum dry density (based on AASHTO T 99) were in the range of 16 to 20 percent and 1,600 to 1,746 kg/m^3 (100 to 109 lb/ft^3), respectively.

2.8.2 Boring Logs

Because of concerns over the relatively shallow water table and the presence of multiple underlying soil layers, an additional subsurface investigation was performed on February 23, 1995. A drill rig was used and a boring log was prepared (Figure 23). The water table was determined to be approximately 2.5 m (8 ft) from the surface. The relatively high water table can be attributed to the high water flow in the Carson River which resulted from the relatively wet winter of 1994–95 (200 percent of average precipitation) in the Sierra Nevada mountains. The Sierra Nevada mountains provide the watershed for the Carson River.

2.8.3 Falling Weight Deflectometer

FWD tests were performed at relatively close intervals along the planned alignment of the track. The FWD tests were conducted at the top of the bladed surface (bladed to remove vegetation). Deflections and backcalculated modulus values were used to evaluate uniformity of the subgrade, to identify potential areas for overexcavation and recompaction prior to embankment placement, and to estimate the resilient modulus of the natural soil for pavement design purposes.

FWD measurements were taken at three different times prior to the initiation of track construction: October 24, 1994, February 15, 1995, and April 24, 1995. The reason for the second and third rounds of FWD data collection were in response to the slight alignment shifts of the track; the concerns raised by the FHWA technical panel members at their meeting on February 2–3, 1995; and the potential effects of any seasonal changes in soil moisture on stiffness of the subgrade soil. Details of the location and frequency of testing can be found in WesTrack Work Activity Report 95-1.

2.8.3.1 Deflection Data Analysis

Representative deflection data from the February 15, 1995, FWD testing are shown in Figures 24 and 25. These data suggest that there is a large difference between the deflection at sensor 1 (beneath the loading plate) and sensor 2 (just outside of the loading plate). This relatively large difference is likely due to the shearing of the soil along the edge of the loading plate.

There is a significant difference in the deflections measured during the three periods. These differences are likely due to differences in the seasonal moisture content of the soil.

2.8.3.2 Resilient Modulus Analysis

Various methods for backcalculating the resilient modulus of the subgrade soil were used for the three sets of FWD data. Difficulty was experienced with the use of these methods because of the level of the groundwater table.

Since the October 1994 FWD data were collected during a dry period of the year, a backcalculation method provided in the 1993 *AASHTO Guide for Design of Pavement Structures* was used (*14*). Examination of these results indicates that there is a difference in deflection among some of the sections (Figures 26 and 27). This difference can be partially explained by the fact that the area at the west end of the site had been previously deep disced and irrigated for agricultural purposes. This information was used to site the track as far to the east as possible. The mean and standard deviations of the backcalculated resilient modulus values for the north, south, and combined tangents are shown in Table 13. The reported values are considered to be representative of moduli for the 2.5-m (8-ft) soil layer above the water table.

The February 1995 data were obtained to define the effect of moving the track location slightly and to investigate the effect of soil moisture on deflection and resilient modulus. The presence of the water table at a depth of about 2.5 m (8 ft) below the surface and the high soil shear at the edge of the load plate negated the use of the single layer AASHTO backcalculation equation to determine resilient modulus from the deflection data. An interactive backcalculation program (MATCH) (*15*) was used on the three deflections furthest from the FWD load plate to determine resilient modulus. Figures 28 and 29 and Table 14 contain a summary of these data. These data are considered representative of backcalculated moduli for the 2.5-m (8-ft) soil layer above the water table.

The April 1995 data were obtained to more precisely define the subgrade support for pavement design purposes and to further define the effects of soil moisture content. The interactive MATCH program was used to determine the resilient modulus. Figures 30 and 31 and Table 15 contain a summary of these data. These data are considered representative of backcalculated moduli for the 2.5-m (8-ft) soil layer above the water table.

Based on deflection data results and backcalculated resilient modulus values and historic information of the subgrade soil at the test site, areas of subgrade were recommended for over-excavation. Details of this recommendation are contained in reference 3. In addition, a decision was made to mix the subgrade soils with depth and along the tangents to provide a uniform subgrade for the test track.

2.9 PAVEMENT THICKNESS DESIGN

2.9.1 Introduction

Pavement thickness designs were determined for three distinct areas of the WesTrack facility; test or tangent sections, turnarounds, and ramps. The methodology used to determine the thicknesses for each of these parts of the test track is discussed below. The discussion focuses first on the pavement design similarities and then defines the different requirements for each part of the facility.

The design of the pavement thicknesses for the WesTrack facility was somewhat different than the design approach used for typical pavement projects. These differences are summarized below.

- In order to develop pavement performance models from test track performance data, it is necessary to have a significant number of the pavement test sections “fail” during the life of the track. Therefore, the pavement structural section was designed to provide for a high probability of failure for all pavements placed on the project during the trafficking period of the project.
- In order to provide structural sections with a high probability of failure within the trafficking period, mechanistic pavement structural design tools were used which are capable of considering seasonal variations in the properties of the materials used to construct WesTrack (subgrade, engineered fill, granular base course, and HMA).
- The modes of failure of interest on the WesTrack project were fatigue or alligator cracking and permanent deformation or rutting of the HMA. Pavement roughness models were also investigated.
- It was desirable to perform a “preliminary” and a “final” structural design for the facility. The preliminary structural design was based on material properties of the layers obtained from material sampling prior to construction and laboratory characterization testing. The final design was directed toward adjusting the thickness of the HMA depending on the in-place or constructed properties of the pavement materials as measured from backcalculated FWD values for the various pavement layers.

The overall approach used for developing the pavement structural or thickness design involved the use of a microcomputer-based design procedure for mechanistic pavement design (McPave) (*16*). This approach incorporates reliability-based pavement design/performance prediction models and includes seasonal material properties.

2.9.2 Design Models

Three independent performance prediction models were used to establish the pavement thickness design: fatigue or alligator cracking; permanent deformation or rutting at the surface from overstressing the underlying subgrade soil; and serviceability or roughness. All three models were formulated to predict the allowable 80-kN (18,000-lb) ESAL application before a certain pavement failure level inherent in the model was reached.

2.9.2.1 Fatigue Cracking

The asphalt tensile strain-based relationship originally developed under NCHRP Project 1-10B (*17*) and often referred to as the Finn Equation (after one of its authors, Fred N. Finn) was selected for use in predicting the allowable 80-kN (18,000-lb) ESALs to a certain level of fatigue cracking. The model is based primarily on a comprehensive laboratory study of asphalt mix fatigue where the results were ultimately calibrated to the observed performance of mixes constructed in the field.

Fatigue cracking was one of the two desired modes of failure for the target (optimum) WesTrack mixes and the Finn Equation is the one mechanistic-based model most frequently used in the United States. The failure criteria for this model is an areal fatigue cracking level of 45 percent in the wheelpath (22.5 percent of the total pavement area). This equation (originally derived in U.S. customary units) is shown below:

$$\log N_f = 16.086 - 3.291 * \log(\varepsilon_{AC}/10^{-6}) - 0.854 * \log(E_{AC}/1000) \quad (1)$$

where

N_f = allowable 80-kN (18,000-lb) ESALs to 45 percent areal cracking,
ε_{AC} = maximum tensile strain in the HMA layer in mm/mm (in./in.), and
E_{AC} = design elastic modulus (ksi) of the HMA layer as determined from unconfined triaxial testing.*

*1 ksi = 6.9 MPa

2.9.2.2 Permanent Deformation

The equation developed by the Asphalt Institute and incorporated into its pavement design procedures to account for the potential of rutting (or permanent deformation) in the underlying layers was selected for use in estimating the allowable number of 80-kN (18,000-lb) ESALs to a certain level of distortion in the pavement structure, as observed in the surface of the HMA. The failure level inherent in the model is a rut depth of 12.7 mm (0.5 in.). The mode of failure is associated with overstressing the subgrade soil layer. Rutting in the underlying pavement layers is not a desired mode of deterioration for the test track and the structural design emphasized minimizing its likelihood. The equation for rutting is as follows (*18*):

$$N_r = 1.388 * 10^{-9} * (\varepsilon_{SG})^{-4.484} \quad (2)$$

where

N_r = allowable 80-kN (18,000-lb) ESALs to 12.7 mm (0.5 in.) of rutting (as displayed in the HMA surface).
ε_{SG} = maximum vertical compressive strain at the top of the subgrade soil (mm/mm or in./in.).

It should be noted that this equation was rearranged from its original form where the maximum (limiting) vertical compressive strain was the dependent variable.

2.9.2.3 Serviceability

Although not as critical to the performance comparisons between mixes as fatigue cracking and rutting, serviceability was included in the design process to provide a basis for comparing the results with those of a typical AASHTO pavement design guide solution (*14*). The serviceability loss selected for use in the design model is 3.0. This is based on an assumed initial serviceability of 4.5 and a terminal value of 1.5. To keep comparisons with fatigue and rutting on a similar basis, design reliability was not applied in generating the serviceability-based solutions. The equation for allowable 80-kN (18,000-lb) ESAL applications is as follows (*14*):

$$\log ESAL_t = 9.36 * \log(SN + 1) - 0.20 + \frac{\log\left[\dfrac{\Delta}{4.2 - 1.5}\right]}{0.40 + \dfrac{1094}{(SN + 1)^{5.19}}} + 2.32 * \log M_R - 8.07 \quad (3)$$

where

$ESAL_t$ = allowable 80-kN (18,000-lb) ESALs on a given flexible pavement with a given (design) serviceability loss,
SN = flexible pavement structural number,
ΔPSI = design serviceability loss, and
M_R = subgrade soil resilient modulus (psi).*

*1 psi = 6.9 kPa

2.9.3 Linear Damage Model

One of the primary reasons for selecting the site of the test track is the relatively mild, high desert climate in northern Nevada. There is little precipitation (the average annual is less than 100 mm/year [4 in.]) and the winters and summers are relatively mild. Nonetheless, there are some changes that can take place seasonally, particularly in the groundwater level, moisture content of the natural soil, and stiffness of the HMA that have a demonstrated impact on the measured deflections and overall support. Because of the effect that these seasonal support variations might have on pavement performance, a method was developed to estimate the pavement damage on a month-by-month basis. The method is based on Miner's linear damage hypothesis and makes it possible to accumulate the damage from individual loads of varying magnitude.

$$\text{Total Damage} = D = \sum_{i=1}^{m} \frac{n_i}{N_i} \tag{4}$$

where

- D = total damage incurred by a section as a result of the application of multiple loads (or strains) of varying magnitude (in this case, the applied loads are constant, but the induced strains vary because of changes in the seasons),
- n_i = number of actual load (strain) applications during the ith season,
- N_i = number of allowable load (strain) applications during the ith season, and
- m = number of seasons (in this case, the seasons will be identified as individual months).

This linear damage method has traditionally been applied to problems involving fatigue; however, it is also applicable to both the rutting and the serviceability analyses.

The use of this linear damage method for fatigue damage is illustrated in reference 3.

2.9.4 Material Properties

2.9.4.1 Natural Soil or Subgrade

Information presented in Section 2.8, Geotechnical Investigation, and more completely presented in reference 3 provided information on average backcalculated resilient modulus values of the natural soil at the site for three different months since the project beginning. These values are summarized in Table 16.

As can be seen, seasonal variations (such as moisture content and level of groundwater table), even in a desert climate, can have a significant impact on soil properties.

Research at the UNR (*19*) on the effects of seasonal variations within the state has shown that, for state highways near the WesTrack site, natural soil moduli vary from 70 to 102 percent of the summer moduli. This normalized range is shown in Table 17.

This information, along with the average backcalculated soil modulus values for the three separate months, was used to estimate the monthly resilient modulus values for the natural soil (Table 18). These estimates are recognized as being approximations and are intended only for use in developing the preliminary pavement structural design. For purposes of developing the final pavement structural design, these values were intended to be enhanced based on continued resilient modulus testing and backcalculation analysis of FWD data collected as the structure was being built.

For this preliminary design analysis, the following observations were made:

- The October 1994 modulus values are higher than what can be expected after paving. After the HMA surface layer is placed, the soil will have higher moisture content and correspondingly lower resilient modulus values particularly in the extreme dry seasons.
- By comparison, winter 1994–95 and spring 1995 were much wetter than normal. The Sierra Nevada mountains had 200 percent of their average snowfall while in this period the WesTrack site experienced 50 mm (2 in.) more precipitation than the annual average of 100 mm (4 in.). In terms of flow, the Carson River, which influences the groundwater level at the site, experienced its fourth largest spring runoff in 100 years. This leads to the conclusion that the backcalculated modulus values for February and April are lower than normal and lower than what would be expected after the pavement is constructed. As a result, engineering judgment was used to interpret both the backcalculated modulus values and the Nevada seasonal weighting coefficients to estimate the design roadbed soil resilient modulus values for a typical year at the site.

2.9.4.2 Engineered Fill

Assignment of resilient modulus values for the engineered fill was made based upon the laboratory-based resilient modulus testing carried out on select soil samples (see Table 19) and documented in reference 35. The resilient modulus versus deviator stress relationships for the engineered fill (Figures 32 and 33) are used in an iterative (computer-based) process to estimate the engineered fill modulus, taking into account the pavement structure, the design load, and the resilient modulus of the natural soil below.

After several iterations using different pavement structures and varying soil moduli, it was observed that the modulus of the engineered fill was relatively insensitive to the modulus of the underlying soil. The average value was about 64,000 kPa (9,300 psi) for the tangent sections and the turnarounds and about 55,000 kPa (8,000 psi) for the ramps. The state-of-stress associated with these values was on the order of 7 kPa (1 psi) confining pressure and 30 kPa (4 psi) deviator stress.

2.9.4.3 Base Course Material

Reference 35 and Table 20 and Figure 34 in this report provide the results of resilient modulus testing on two potential base course materials. Other base course alternatives were identified which had more uniformity and higher stiffness than the base course material tested.

The base course material used on the track was a blend of four different aggregate stockpiles generated by crushing rock obtained from a quarry at the site (NATC pit). No resilient modulus values (either from laboratory or backcalculation) were available for thickness design prior to construction. However, the stockpiles were blended to achieve maximum uniformity and high stiffness. Consequently, for purposes of the preliminary pavement structural design, a modified resilient modulus versus bulk stress relationship was used. The slope of the bulk stress relationship (306) remained the same; however, the intercept was increased from 60,749 kPa (8,817 psi) to 220,000 kPa (32,000 psi) to reflect the anticipated stiffness gain from the new blended base course. This translates into a base course resilient modulus range of 210,000 to 240,000 kPa (31,000 to 35,000 psi). The state-of-stress associated with tangent section conditions is a bulk stress on the order of 0 to 70 kPa (0 to 10 psi).

The level of QC anticipated for the base course on the turnarounds and ramps is reduced as compared with the QC for the base course used for the tangent or test sections. Thus, the intercept for the resilient modulus-bulk stress was increased only to 138,000 kPa (20,000 psi) for the base course on the turnarounds and ramps as compared with 220,000 kPa (32,000 psi) for the tangent base course.

2.9.4.4 Hot-Mix Asphalt

HMA mixture designs were not completed when the preliminary pavement thickness design was performed. For purposes of this preliminary structural design/analysis, resilient modulus values were assigned for each month based on engineering judgment and experience in Nevada (Table 21).

2.9.5 Structural Designs

Reliability concepts developed for the AASHTO pavement design guides (*14*) were used to introduce reliability into the pavement design methodology. This reliability concept allows the engineer to treat the uncertainty in pavement material properties, construction, environment, and performance model lack-of-fit. The approach used at WesTrack is detailed in Chapter 8 of reference 35.

Figure 35 and Table 22 describe the pavement thickness and layer designs for the tangents, turnarounds, and ramps. The designs were performed by an iterative process using a computer program developed specifically for this project. The program uses the elastic layer model (ELSYM5) (*20*) as the basis for the layer stress and strain calculations. Design details for the tangent, turnaround, and ramp structural designs follow.

2.9.5.1 Tangent Sections

The tangent sections were designed for 10 million 80-kN (18,000-lb) ESALs with a 10 percent reliability. The 10 percent reliability was used to provide a high potential for distress to occur on the track. The layer thicknesses were selected such that rutting in the pavement would likely not occur because of subgrade and base course permanent deformation. The anticipated fatigue life of the pavement at the 50 percent reliability level was 3.3 million ESALs for a typical HMA.

2.9.5.2 Turnarounds

The turnarounds were also designed for 10 million ESALs, but with a 90 percent reliability level. Because of the 18 percent superelevations on the curves, the thickness of the engineered fill was substantially greater than the tangent or test sections. In addition, the thickness of the HMA was increased substantially over the tangent sections to ensure that premature fatigue failures would not occur on the tangent sections during trafficking to at least 10 million ESALs.

2.9.5.3 Ramps

The thickness design for the ramps and parking areas were based on the following vehicle loading/traffic assumptions:

- 10.3 ESALs per truck operation.
- Eight truck operations per day.
- Truck operates 365 days per year over 2 years.
- 60,000 ESALs over the design life.

A 70 percent reliability was used for the ramps and parking areas.

2.9.6 Summary

The tangent pavement design was established during the preliminary thickness design process as described above and was not changed when subsequent FWD data were obtained and analyzed during the construction of the subgrade, engineered fill, and base course. The data obtained from these FWD tests did not persuade the WesTrack team that changes in the thickness design were justified. The final structural section for the tangent or test sections is shown below.

• HMA	150 mm	6 in.
• Base course	300 mm	12 in.
• Engineered fill	460 mm	18 in.
• Subgrade-compacted	150 mm	6 in.

2.10 QUALITY CONTROL/QUALITY ASSURANCE TEST PLAN

A significant effort was expended on the project to establish QC/QA sampling and testing plans. Chapter 12 of reference 35 and WesTrack Technical Reports UNR-18 (*21*) and

UNR-19 (*22*) contain the background information used in the development of the plans. A summary of this development and the resulting QC/QA plans are contained in this section of the report.

For the purposes of this report, QC refers to the sampling and testing that was performed during the construction of the test track. QA refers to the sampling and testing that was performed mostly after construction was completed; it supplied the information that "best" described the properties of the pavement materials placed at WesTrack. The QA test results were used for performance modeling and development of the PRS.

It was the intent of the WesTrack team to use conventional QC/QA tests at WesTrack supplemented by a few more state-of-the-art tests. The test frequency was at a much more frequent basis than normal construction because of the desire to define the material properties associated with all of the individual sections. Acceptance levels were fixed such that the project would be constructed at a low level of material property variability.

To establish typical levels of construction variability, QC/QA information was collected from several sources. This information was used to establish acceptance limits for the WesTrack project; it is summarized in Chapter 12 of reference 3 and WesTrack Technical Report UNR-29 (*4*). The proposed WesTrack acceptance criteria were largely based on the construction variability experienced on the AASHO Road Test. It was the goal of the research team to place the pavement materials at WesTrack at or below the variability measured at the AASHO Road Test.

The QC/QA plans for the subgrade and engineered fill, granular base course, and HMA are summarized in individual sections below. Background information for the subgrade and engineered fill and base course can be found in reference 3. Background information for the HMA placed as part of the original construction is contained in reference 21 and for the HMA placed during construction of the replacement sections in reference 22.

2.10.1 Subgrade and Engineered Fill

2.10.1.1 Frequency of Sampling and Testing

For the purposes of establishing the frequency of sampling and testing associated with the construction of the subgrade and engineered fill, a lot was defined as a test section on the tangent portion of the track and as a single turnaround for the ends of the track (Figure 36). For the purpose of sample identification, the codes shown in Figure 37 were used for the subgrade, engineered fill, granular base course, and HMA layers.

Tables 23 and 24 show the sampling and testing requirements for the tangents and turnarounds. Five nuclear density, 2 sand cone density, and 24 FWD tests were to be conducted for each of the 26 sections on the tangents. Moisture density, hydrometer, sieve analysis, Atterberg Limits, natural moisture content, resilient modulus, CBR, R-values, and permeability values were performed at the frequency shown in Table 23 for each of the two tangents. A reduced frequency of sampling and testing was performed on the turnarounds (Table 24).

Density and moisture content were used to control and accept the subgrade and engineering fill material. The other tests were used to characterize the subgrade and engineered fill material as possible input for development of the pavement performance models.

2.10.1.2 Acceptance

The AASHO Road Test used a relative density control range of 95 to 100 percent of maximum dry density and a moisture content of ±2 percentage points of Standard AASHTO compaction (AASHTO T 99). The variability of the construction at the AASHO Road Test for density and moisture content control were 1.9 and 1.2 percentage points expressed as a standard deviation. Information collected from several states (*3*) indicate standard deviations for density in the range from 2.4 to 8.8 and moisture content from 2.9 to 3.6. The AASHO Road Test carefully selected the embankment soil, field pulverized the soil, and carefully controlled the field compaction operation.

The AASHO Road Test acceptance was based on a percent within limits (PWL) of 55 percent for all lifts, except the top lift (PWL of 60). On the AASHO Road Test, 80 percent of the test results were within limits for density and 83 percent of the test results were within limits for moisture content.

WesTrack established target specifications for density and moisture content described below. The contractor was expected to control the relative density for the entire project at a standard deviation below 1.9 percentage points and the moisture content for the entire project at a standard deviation below 1.2 percentage points for the subgrade and engineered fill.

The WesTrack specification identified a target density of 92 percent of modified AASHTO compaction (AASHTO T 180) within a range of ±1 percentage point. Moisture content was to be controlled at a level of –1 to +3 percentage points of optimum. The contractor was expected to have a PWL for the entire project of 80 percent for density and 80 percent and above for moisture content for the subgrade and engineered fill. These are the PWLs associated with the AASHO Road Test.

Acceptance criteria for individual lots was set at 55 percent (PWL) or above for the five density measurements per lot for the subgrade and the first layer of the engineered fill. The top lift of the engineered fill was expected to have a PWL of 60 percent for relative density. Table 25 contains a summary of the acceptance requirements for the subgrade and engineering fill.

2.10.2 Base Course

2.10.2.1 Frequency of Sampling and Testing

The base course was sampled during the crushing operation on a frequent basis (five sublots per lot with a sublot equal to 227 Mg [250 tons]). Sieve analyses were performed on every sample and moisture density relationships, Atterberg Limits, resilient modulus, CBR, R-value, and fractured face counts were determined less frequently. Table 26 shows the property requirements for the base course. They were based on Nevada DOT requirements except that the 0.075-mm (No. 200) sieve requirement was lowered and the R-value was increased to 78.

For the purposes of establishing the frequency of sampling and testing associated with the construction of the base course, a lot was defined as a test section on the tangent portion of the track and as a single turnaround for the ends of the track (Figure 36). For the purpose of sample identification, the codes shown in Figure 37 were used.

Tables 23 and 24 show the sampling and testing requirements for the tangents and turnarounds. Five nuclear density, 2 sand cone density, and 24 FWD tests were conducted for each of the 26 sections on the tangents. Moisture density, hydrometer, sieve analysis, Atterberg Limits, natural moisture content, resilient modulus, CBR, R-values, and permeability values were performed at the frequency shown in Table 23 for each of the two tangents. A reduced frequency of sampling and testing was performed on the turnarounds (Table 24).

Gradation, moisture content, and density were used to control and accept the base course material. The other tests were used to characterize the base course material as possible input for development of the pavement performance models.

2.10.2.2 Acceptance

The AASHO Road Test used a relative density control range of 100 to 105 percent of maximum dry density and a moisture content of ±1 percentage point of Standard AASHTO compaction (AASHTO T 99). The variability of the construction at the AASHO Road Test for density, moisture content control, and gradation has been summarized in reference 3. Information collected from several states (*3*) indicate standard deviations for density, moisture content, and gradation greater than those reported on the AASHO Road Test. The AASHO Road Test carefully produced and controlled the field compaction operation.

The AASHO Road Test acceptance was based on a PWL of 65 percent. Depending on the subbase/base material, the AASHO Road Test reported PWL values of from 70 to 92 percent for density. The PWL or moisture content for the crushed stone base at the AASHO Road Test was 96 percent.

The WesTrack specification identified a density range from 96 to 101 percent of modified AASHTO density (AASHTO T 180). Moisture content was to be controlled at a level of ±1 percentage point of optimum. The contractor was expected to have a PWL for the entire project of 65 percent for density and 65 percent and above for moisture content. These PWLs are those associated with the AASHO Road Test.

Tables 27 and 28 contain a summary of the requirements for the base course at WesTrack. Acceptance criteria for individual lot density and moisture content were set at 65 percent (PWL) or above for the five density measurements per lot and 85 percent for the project. Requirements for gradation and thickness are also shown in Tables 27 and 28.

2.10.3 Hot-Mix Asphalt

Testing programs for the HMA mixtures used on WesTrack are defined in WesTrack Technical Reports UNR-18 (*21*) and UNR-19 (*22*) in considerable detail. This part of the report will only define the QC/QA testing plans used for the original construction and the construction of the replacement sections at WesTrack. Asphalt binder, aggregate, and HMA characterization not performed as part of the QC/QA program is presented in Chapter 5.

QC/QA test plans were developed for the asphalt binders, aggregates, and HMAs used at WesTrack. A "lot" was defined as the amount of HMA placed in a single lift in a test section for the tangent construction and the amount of HMA placed in a single lift for a single turnaround.

2.10.3.1 Original Construction QC Plan

Table 29 shows the daily QC plan for samples taken at the hot-mix plant. A sampling and testing program is described for the asphalt binder, aggregate, and HMA. The number of tests per sublot and the number of sublots per lot are described in this table.

During a construction day, a single lift of hot-mix was placed on the test sections for a particular gradation (fine, fine plus, or coarse). For QC testing, several sections could be combined because they were produced at the same asphalt binder content and gradation. Thus, for QC purposes and time restrictions, lots were created from several sections with the same target asphalt binder contents.

Table 30 shows the actual sampling and testing that was accomplished as part of the QC test program at WesTrack. Adjustments in the QC plan were necessary due to personnel availability, equipment availability, and time constraints. Both the original and adjusted QC/QA plans are included to illustrate the "ideal" versus "practical" plans. The original QC plan (if executed) would have provided more certainty to the data sets and would have allowed for more adjustments during construction.

The QC plan and the QC sampling that was actually performed at the laydown site are shown in Tables 31 and 32.

HMA laydown temperatures and in-place air voids were obtained frequently. Water sensitivity tests were not performed.

2.10.3.2 Original Construction QA Plan

The original and revised QA plans for the original construction are shown in Tables 33 and 34. QA programs were developed and executed for asphalt binders, aggregates, and HMA. Tests per sublot and sublots per lot are defined in these tables. In general, a lot was equal to a test section and five sublots were tested per lot for the important QA parameters. Daily sampling sheets were developed to coordinate the field activities of the sampling and test crews.

2.10.3.3 Replacement Section QC Plan

Tables 35 through 38 show the daily QC plan for sampling and testing of the top lift of the HMA. These tables show the sampling and testing program as well as the amount of material to be sampled for the aggregate and the HMA. The asphalt binder sampling and test plan, together with additional details of the QC and QA sampling and test plans can be found in reference 24. During a construction day, a single lift of a fine-, fine-plus- or coarse-graded hot-mix was placed on the test sections.

2.10.3.4 Replacement Section QA Plan

The QA testing program for the replacement sections was largely performed after construction and hence has been identified as postconstruction testing in Tables 39 through 41. Sampling and testing programs are described in these tables for asphalt binders, aggregates, mix design verification, and HMA testing. Daily sampling sheets were developed to coordinate the field activities of the sampling and test crews. Table 42 contains a typical daily sheet.

2.11 PLANS AND SPECIFICATIONS

The plans and construction specification for the WesTrack project were developed by Harding Lawson and Associates (HLA) with input from the WesTrack team. The plans are available from NCE and HLA. WesTrack Technical Report NCE-2 (*13*) contains plan sheets describing the track geometrics and pavement cross sections.

Specifications for the project are contained in WesTrack Technical Report UNR-31 (*23*). Specifications were developed for the following items:

- Section 201 Clearing and Grubbing.
- Section 203 Excavation and Embankment.
- Section 206 Structure Excavation.
- Section 207 Backfill.
- Section 302 Aggregate Base.
- Section 406 Prime Coat.
- Section 410 HMA Pavement (Dense Graded).
- Section 502 Concrete Structures.
- Section 601 Pipe Culverts.
- Section 603 Reinforced Concrete Pipe.
- Section 610 Riprap.

The excavation and embankment, aggregate base, and HMA specifications contain an extensive guideline for producing uniform paving materials. The AASHO Road Test QC/QA information was used to formulate these specifications. Background information used for the specification development is contained in reference 3 and WesTrack Technical Report UNR-29 (*4*) and has been summarized earlier in this report.

During the construction of the project, some changes were made in the specification requirements to accommodate the materials at the site and the capability of the construction operation. For example, the in-place density of the granular base material was difficult to obtain within the specified range of 96 to 101 percent of modified AASHTO density (AASHTO T 180). The density requirement was reduced to a level that was achievable and that could be uniformly obtained. The success of the project was more dependent on the placement of a uniform base course than on a base course of a relatively high density. The development of performance relationships for HMA materials was dependent on uniform supporting layers.

The properties of the subgrade and engineered fill, base course, and HMA placed with these specifications are contained in WesTrack Technical Reports and in this report.

2.12 HOT-MIX ASPHALT MIXTURE DESIGN

The HMA mixture design consisted of selecting the asphalt binder and aggregates, and establishing target gradations and asphalt binder contents for the mixtures placed during original construction (fine, fine plus, and coarse) and the replacement mixture (coarse-graded). Detailed discussions of this process are presented in the following WesTrack Technical Reports:

- UNR-1 (*12*) Asphalt Binder Properties—Original Construction
- UNR-2 (*24*) Asphalt Binder Properties—Replacement Sections
- UNR-3 (*25*) Hydrated Lime Properties
- UNR-4 (*26*) Aggregate Properties—Original Construction
- UNR-5 (*27*) Aggregate Properties—Replacement Sections
- UNR-6 (*28*) HMA Mixture Design—Original Construction

- UNR-7 (*29*) HMA Mixture Design—Replacement Sections

A summary of the mixture design process and properties of the asphalt binders, aggregates, and HMA mixtures is presented below.

2.12.1 Asphalt Binder Properties—Original Construction

2.12.1.1 Binder Grade Selection

The WesTrack team held several discussions relative to the selection of the asphalt binder for the project. Because of the limited number of test sections (26) available at WesTrack, a decision was made to use a single asphalt binder that would meet the AASHTO specification requirements for the track location. Since the Superpave binder specification was developed primarily on research conducted on "neat" or nonmodified asphalt binders and since some issues were being raised relative to the suitability of the SHRP-developed asphalt binder test methods for use with modified asphalt binders, the team elected to use a nonmodified asphalt binder.

The SHRP-developed "SHRPBIND" software was used to select the asphalt binder grade for WesTrack (*30*). Nine weather stations in the northwest portion of Nevada, with similar elevations, were selected and SHRPBIND was used to determine the performance-graded binder (PG grade) for use at each location. Table 43 contains a summary of the output from SHRPBIND. Reliability for selected grades are shown for each weather station and the PG grades for 50 and 98 percent reliability are given.

The weather stations at Fernley, Lahontan Dam, Wellington, and Yerington are in the same general area (within 50 km [30 mi]) and at approximately the same elevation (1,300 m [4,100 ft]). The Lahontan Dam weather station site is approximately 12 km (7 mi) from the site and about the same elevation as the test track. The 50 percent reliable asphalt binder grade for Lahontan Dam is a PG 58-16, while the 98 percent reliable asphalt binder grade is a PG 64-28.

For the Lahontan Dam weather station, Table 43 indicates that a high temperature grade designation of 58 will provide a 68 percent reliability while a grade designation of 64 will provide a 98 percent reliability. Table 43 also indicates that the low temperature grade designation of –16 will provide a 58 percent reliability, the –22 grade designation a 94 percent reliability, and the –28 grade designation a 98 percent reliability at the Lahontan Dam weather station site. Based on the information presented above, a high temperature grade of 64 should provide a 98 percent reliability throughout the general geographic area of the test track. A low temperature grade designation of –22 should provide for 60 to 90 percent reliability in the general geographic area.

Superpave binder selection criteria available in late 1994 (*31*) indicated that selection of the asphalt binder grade by climate assumes that a binder will be used in an HMA mixture subjected to traffic moving at speeds of approximately 90 km/hr (55 mph). For traffic moving at slower speeds, an increase in the high temperature designation of the binder should be considered. An increase of one or two grades depending on traffic speed was also suggested. The geometrics of the track and the driverless vehicle control systems were designed for a 65 km/hr (40 mph) truck traffic speed. An increase in Superpave asphalt binder grade based on speed could not be justified with available information contained in the SHRP literature.

Superpave binder selection criteria available in late 1994 (*32*) also indicated that selection of the asphalt binder grade by climate assumes that the design traffic level is less than 10 million ESALs. When the design traffic level exceeds 10 million ESALs, the designer is encouraged to "consider" increasing the high temperature grade designation by one grade. When the design traffic level exceeds 30 million ESALs, the designer is required to increase the high temperature by one grade. These guidelines were considered by the WesTrack team. The WesTrack team elected not to increase the asphalt binder high temperature grade since the expected total traffic on the facility was to be 10 million ESALs (over its design life of 2 to 3 years); the structural thickness design was established to produce fatigue failures at 3.3 million ESALs; the basis for this high temperature grade increase was based on little published engineering information; and the guidelines were not specific relative to the design life (20 years). It should be recognized that WesTrack was an accelerated test track experiment and hence the traffic level (average daily truck traffic levels) is relatively high.

Based on the above-described binder selection process, a PG 64-28 neat asphalt binder was the desired binder grade because both high and low temperature reliability were above 98 percent. Through a series of phone calls, data were collected from some western states' refineries and state departments of transportation in an attempt to locate a PG 64-28 neat asphalt binder. However, no source of this grade was located within a reasonable geographic area. Two sources of PG 64-22 were located. A PG 64-22 could reliably be produced from western Canadian crude and was available from either U.S. or Canadian refineries. A second source of PG 64-22 was available from a San Francisco Bay Area refinery and was a blend of a domestic and foreign crude. A decision was made to work with the San Francisco Bay Area refinery because of its proximity and its willingness to produce a refinery "tank" of the asphalt binder and hold it until construction. The price of the asphalt binder from the San Francisco Bay Area was approximately $50 per ton less than alternate sources.

The selection of the PG 64-22 grade, rather than a PG 64-28 grade of asphalt binder, was based on both the Superpave designated grade and the availability of the PG 64-22. Only modified PG 64-28 grades of asphalt binders were reliably available in the western states in late 1994 and early 1995. As indicated above, the high temperature grade designation of 64 provided for 98 percent reliability and the –22 low temperature grade designation provided for an approximately 60 to 94 percent reliability depending on the weather station as shown in Table 43. The Lahontan Dam weather station is only about 12 km (7 mi) from the test track site and indicates a 94 percentile low temperature reliability as shown in Table 43.

The final selection of the –22 low temperature grade was, therefore, based on the following criteria:

- Availability of –22 low temperature neat asphalt binder at reasonable cost.
- Relatively high reliability of 90 percent for the –22 grade.
- Low probability of thermal cracking in 3 years of operation of the test track.
- Low probability of thermal cracking due to the short length of the test sections.

2.12.1.2 Asphalt Binder Properties

The properties of the asphalt binder used during construction of the original 26 test sections are contained in detail in WesTrack Technical Report UNR-1 (*12*) and reference 33. The sampling and testing plans used to define the asphalt binder properties are presented in these reports. Testing was performed to determine the viscosity grade as well as the Superpave grade of the asphalt binder. Superpave properties were determined over a temperature range and at different frequencies.

Preconstruction, construction (QC), and postconstruction (QA) testing was performed on the binder. Multiple samples were, therefore, obtained and tested with a variety of tests. Results of this extensive testing program are contained in references 12 and 33. Representative properties of the asphalt binder used during original construction are shown in Tables 44 and 45 for Superpave properties and conventional viscosity specification properties. These reported values are from a single sample considered representative of the asphalt binder used during original construction. The asphalt binder is graded as a PG 64-22 and AC-20.

2.12.2 Asphalt Binder Properties—Replacement Sections

The properties of the asphalt binder (supplied by Idaho Asphalt) used during construction of the eight replacement sections are shown in detail in WesTrack Technical Report UNR-2 (*24*). The sampling and testing plan used to define the asphalt binder properties are presented in this report. Testing was performed to determine the viscosity grade as well as the Superpave grade of the asphalt binder. Superpave properties were determined over a temperature range.

Preconstruction, construction (QC), and postconstruction (QA) testing was performed on the binder. Multiple samples were, therefore, obtained and tested with a variety of tests. Results of this testing program are contained in reference 24.

Tables 46 and 47 contain Superpave and viscosity graded physical properties representative of the asphalt binder used to construct the replacement sections. These reported values are from a single sample considered representative of the asphalt binder used for construction of the replacement sections.

Tables 48 and 49 compare the properties of the asphalt binders used for construction of the original and replacement sections at WesTrack. These data represent the average values from the QA test samples as tested by UNR. In comparison to the asphalt binder used for the original sections, the asphalt binder used for the replacement sections has higher stiffness at the design rutting temperature (64°C [147°F]), lower stiffness at the intermediate fatigue temperature (25°C [77°F]), and lower stiffness at the thermal cracking temperature (–12°C [10°F]).

Tables 50 and 51 compare properties of the asphalt binders used on original and replacement sections at various temperatures and aging conditions. The data used to develop Tables 50 and 51 are from two representative samples of the asphalt binders used on the original and replacement sections. The original and replacement binders are the same PG grade but they are from different refineries.

2.12.3 Hydrated Lime Properties

Hydrated lime was used on both the original and replacement sections constructed at WesTrack. The hydrated lime was added dry at a rate of 1.5 percent (by weight of dry aggregate) to a damp aggregate (approximately 2 percent moisture content above the saturated, surface dried condition of the aggregate), mixed in a continuous pugmill and conveyed directly to the heating and mixing chamber of the hot-mix plant. The hydrated lime was supplied by the same company for both the original and the replacement section construction.

The physico-chemical properties of the lime used during construction of the original test sections at WesTrack are summarized in Table 52 for the seven lime samples analyzed. Additional details of the sampling and testing of the hydrated lime can be found in WesTrack Technical Report UNR-3 (*25*). The tests for the physico-chemical properties of the hydrated lime were measured by Chemical Lime Company (*34*). The hydrated lime used on the original construction of WesTrack met the ASTM C 1097 specification for hydrated lime. The hydrated lime used on the replacement sections met the specification and was accepted based on the test results supplied by the hydrated lime manufacturer to Granite Construction.

Samples of baghouse fines obtained during original construction were supplied to the company for the determination of the presence of lime in the baghouse fines. About 25 percent of the lime placed on the damp aggregate was diverted to the baghouse during production of the HMA. Thus, the hydrated lime in the mixtures placed at WesTrack during original construction ranged from 1.3 to 1.5 percent by dry weight of total aggregate (depending on the quantity of baghouse fines being returned).

2.12.4 Aggregate Properties—Original Construction

Aggregates used to construct the original test sections at WesTrack were obtained from Granite Construction's Dayton, Nevada, pit. A local field sand from Wadsworth, Nevada, was also used in the fine- and fine-plus-graded mixtures as described in Section 2.3. The geologic description of these aggregates is contained in Table 53. WesTrack Technical Report UNR-4 (*26*) contains details associated with sampling and physical properties of the aggregates used during construction of the original sections at WesTrack.

2.12.4.1 Sampling and Testing

Detailed sampling and testing plans for the aggregates used during original construction are available in WesTrack Technical Report UNR-18 (*21*). The aggregates were supplied and tested prior to construction as part of the mixture design effort and during and after construction. Stockpiles of the aggregates used for original construction were formed in early 1995 and reserved for exclusive use at WesTrack. These aggregates were produced in 1994.

Dedicated aggregate stockpiles were created to supply a constant source of aggregate for both mixture design and construction. The use of dedicated stockpiles for WesTrack was necessary not only for uniformity of the HMA mixtures placed on the track, but also because the crushing operation for the Dayton pit was changed during winter 1994–1995 and samples of the new production would not be available until late spring 1995. Samples of the stockpiled aggregates were obtained and used for mixture design purposes. These stockpile samples were designated as preconstruction samples.

During construction both "cold feed" and aggregate stockpile samples were obtained. A chute sampling device was used to sample the cold feed. All aggregate stockpile samples were obtained with the aid of a front-end loader. Samples of aggregate obtained during construction were used for both QC testing and QA testing. A large amount of stockpile sampled aggregate was obtained and stored at the FHWA's Materials Reference Library (MRL) located in Sparks, Nevada.

2.12.4.2 Stockpile Gradations

Tables 54 and 55 contain stockpile gradation information obtained prior to construction. The stockpile percentages used for the mixture design and for actual production are shown in Tables 56 and 57. The actual construction percentages differ from the mixture design percentages due to the need to adjust to meet the design gradation in the field and to meet the field volumetric requirements. Note that the "Dayton 3/4-in. material" was decreased and the "Dayton 1/2-in. material" was increased.

The last lift to be placed during original construction was the top lift of the coarse-graded aggregate. The supply of Dayton 1/2-in. aggregate was exhausted by this time. Dayton 3/8-in. from the 1994 production year and 1/2-in. from the 1995 production year was substituted for the 1/2-in. 1994 production year material (Table 57).

2.12.4.3 Physical Properties

Tables 58 through 60 contain physical property data obtained on preconstruction and construction samples of the aggregate. Superpave consensus aggregate properties (coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and sand equivalent); and Superpave source properties (L.A. Abrasion, soundness, and deleterious material) were determined. Data are reported for the 1994 and 1995 production year aggregates.

Numerous aggregate specific gravity measurements were made during mixture design, during construction, and after construction. Results from these tests can be found in reference 28. The aggregate specific gravities used for volumetric mixture design calculations associated with mixture design and QC and QA testing are provided in Table 61.

2.12.5 Aggregate Properties—Replacement Sections

Aggregates used to construct the replacement test sections at WesTrack were obtained from Granite Construction's Lockwood, Nevada, pit. The geologic description of these aggregates is contained in Table 62. WesTrack Technical Report UNR-5 (*27*) contains details associated with sampling and physical properties of the aggregates used during construction of the replacement sections at WesTrack.

2.12.5.1 Sampling and Testing

Detailed sampling and testing plans for the aggregates used during construction of the replacement sections are available in WesTrack Technical Report UNR-19 (*22*). The aggregates were sampled and tested prior to construction as part of the mixture design effort and after construction. Stockpiles of the aggregates used for construction of the replacement sections were not stockpiled at the HMA plant site prior to the start of construction of the trial sections. This created the need for numerous mixture designs and the re-mixing of the stockpiles to achieve the desired uniformity. Changes in the crushing operation during the mixture design process also created the need for performing additional mixture designs. The aggregates used for the replacement sections were produced in 1997 at the Lockwood, Nevada, pit.

Samples of the stockpiled aggregates were obtained and used for mixture design purposes and property determination. These stockpile samples were designated as preconstruction samples.

During construction both cold feed and aggregate stockpile samples were obtained. A chute sampling device was used to sample the cold feed. All aggregate stockpile samples were obtained with the aid of a front-end loader. Samples of aggregate obtained during construction were used for QA testing. A large amount of stockpile sampled aggregate was obtained and stored at the FHWA's MRL located in Sparks, Nevada.

2.12.5.2 Stockpile Gradations

Table 63 contains a summary of stockpile gradation information used to establish the final mixture design for the replacement sections. Extensive stockpile sampling information is available in Appendix A of WesTrack Technical Report UNR-5 (*27*). Stockpile percentages used for the mixture design and for actual production are shown in Table 64. The actual construction percentages did not differ from the mixture design percentages.

2.12.5.3 Physical Properties

Table 65 contains physical property data obtained on preconstruction samples of the aggregate. Superpave consensus aggregate properties (coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and sand equivalent) were determined.

Several aggregate specific gravity measurements were made during mixture design, during construction, and after construction. Results from these tests can be found in reference 27. The aggregate specific gravities used for volumetric mixture design calculations associated with mixture design and QC and QA testing are provided in Table 66.

2.12.6 Hot-Mix Asphalt Mixture Design—Original Construction

More than 60 partial or complete Superpave mixture designs were performed by the FHWA and the UNR to establish HMA target values for the original WesTrack sections. WesTrack Technical Report UNR-6 (*28*) contains details relative to the mixture design effort.

The original planning document for WesTrack anticipated the development of a coarse-graded Superpave mixture from a 100 percent crushed, granite aggregate from the central California coast and the development of coarse-graded and fine-graded Superpave mixtures from a local, partially crushed gravel. The coarse-graded, granite aggregate was expected to produce HMA mixtures whose mechanical properties were relatively insensitive (noncritical) to variation in asphalt binder content and percent passing the 0.075-mm (No. 200) sieve. The fine-graded, partially crushed river gravel was expected to produce HMA mixtures whose mechanical properties were relatively sensitive (critical) with respect to variation in asphalt binder content and percent passing 0.075-mm (No. 200) sieve.

Unfortunately, a coarse-graded Superpave could not be developed from the existing production of 100 percent crushed granite. Coarse- and fine-graded Superpave mixtures were developed from the partially crushed river gravel aggregate. A third mixture was developed with the use of the river gravel aggregate by increasing the minus 0.075-mm (No. 200) content of the fine-graded mixture by 2 to 3 percentage points. This third mixture was designed to study the effects of increased minus 0.075 mm (No. 200) on performance.

The gradations of the three mixtures used for original construction on WesTrack are designated as fine, fine plus, and coarse. All three mixtures were prepared with the partially crushed gravel from Granite Construction's Dayton pit. The gradations of the three mixtures that represent the final mixture designs are shown in Table 67 and Figure 6.

The stockpile blends used for the selected mixture designs are shown in Table 68. Adjustments in the stockpile blends were needed during construction to meet the desired gradations and the mixture volumetrics as determined after Superpave gyratory compaction. Construction stockpile blends are shown in Table 69.

Mixture design weights and volumes are shown in Table 70 for the selected mixture designs compacted with the Superpave gyratory compactor. Superpave volumetric mixture design acceptance criteria for 3 to 10 million ESALs are shown in Table 71 and are those associated with 3 to 10 million ESALs. Complete Superpave volumetric mixture designs were performed for the fine- and coarse-graded mixtures (Figures 38 and 39).

The optimum asphalt binder contents were selected at 5.4 percent and 5.7 percent by total weight of the mixture for the fine- and coarse-graded mixtures, respectively (Table 70). These two mixtures met the requirements of the Superpave method for traffic volumes of the 3 to 10 million ESAL category over the design life of pavement. A partial Superpave volumetric mixture design was performed on the fine plus mixture and results are shown in Table 70 and contained in reference 28. The mixture design information shown in Tables 67, 68, and 70 became the target values for the field production on sections placed during the original construction of WesTrack.

Hveem mixture design information was also obtained on the target mixtures. Hveem stability and volumetric data obtained from samples compacted with the Hveem kneading compactor are shown in Table 72. The stabilities for the fine and fine plus mixtures at the design asphalt binder contents were 43 and 41, respectively. The Hveem stability for the coarse-graded mixture at the design asphalt binder content was 38. The Hveem stability values meet commonly used acceptance criteria.

2.12.7 Hot-Mix Asphalt Mixture Design—Replacement Sections

Nine Superpave mixture designs were performed by the FHWA and the UNR to establish HMA target values for the replacement sections placed at WesTrack. WesTrack Technical Report UNR-7 (*29*) contains details relative to the mixture design effort.

The primary purpose for placing the replacement sections at WesTrack was to better define the effect of aggregate characteristics in coarse-graded Superpave mixtures on pavement performance (primarily permanent deformation or rutting). A 100 percent crushed aggregate was selected for the replacement mixture. The properties of this aggregate are described in WesTrack Technical Reports UNR-5 (*27*) and UNR-7 (*29*). The relatively large number of mixture designs were required because (1) the crushing operation and the raw material source at the quarry were changed during sampling and anticipated placement of the sections at WesTrack and (2) the stockpiles varied considerably over time due to the production of aggregates for different end uses.

The gradation of the aggregate used for the selected mixture design is shown in Table 67. The stockpile blends used for these selected mixtures are shown in Table 68. As shown in Table 69, the stockpile blends were not changed during construction.

Mixture design weights and volumes for the selected mixture are shown in Table 70 for samples compacted with the Superpave gyratory compactor. Superpave volumetric mixture design acceptance criteria for 3 to 10 million ESALs over the design life of the pavement are shown in Table 71. Complete Superpave volumetric mixture designs were performed on a number of the mixtures investigated for use as replacement sections. Detailed Superpave mixture design information for the selected mixture is shown in Table 73.

The optimum asphalt binder content was selected at 5.65 percent by total weight of the mixture for this coarse-graded mixture (Table 70). This mixture meets the requirements of Superpave for traffic volumes in the 3 to 10 million ESAL category. The mixture design information shown in Tables 67, 68, and 70 became the target values for the field production of the replacement sections at WesTrack.

Hveem mixture design information was also obtained on one of the mixtures evaluated for use on the replacement sections. The Hveem stability for this mixture is shown in Table 74. The Hveem stability value at the design asphalt binder content for the mixture selected for use is probably about 30 based on this information.

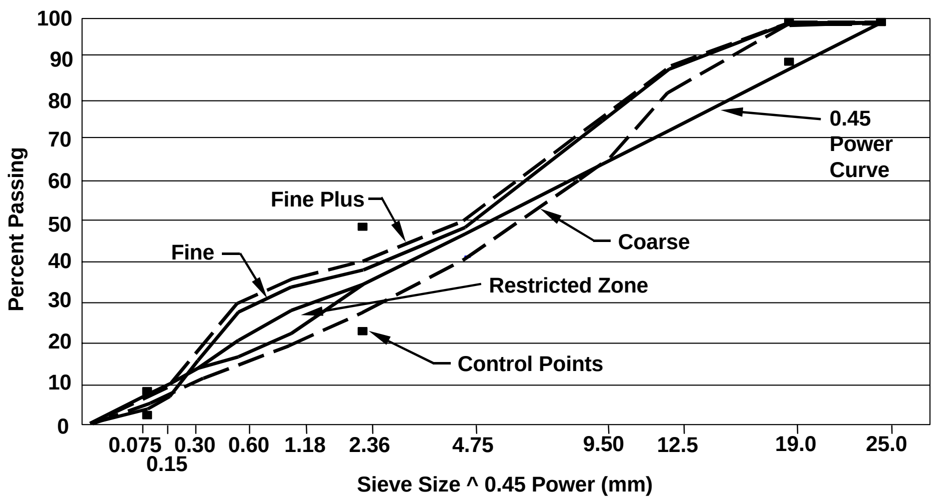


Figure 6. Mixture gradations.

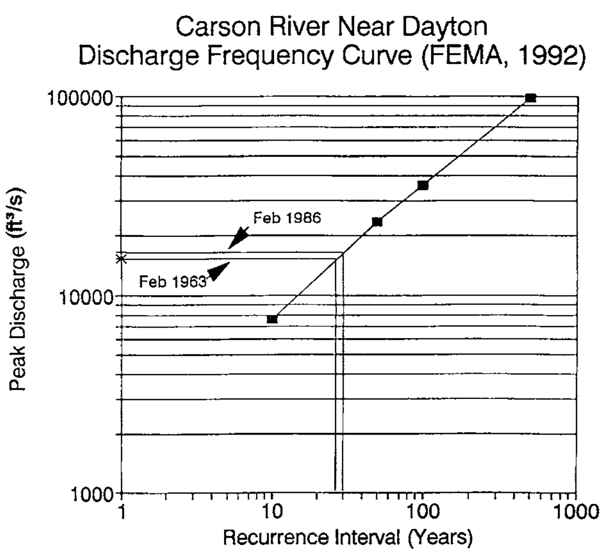


Figure 7. Discharge frequency curve (1 $ft^3/s = 0.028\ m^3/s$).

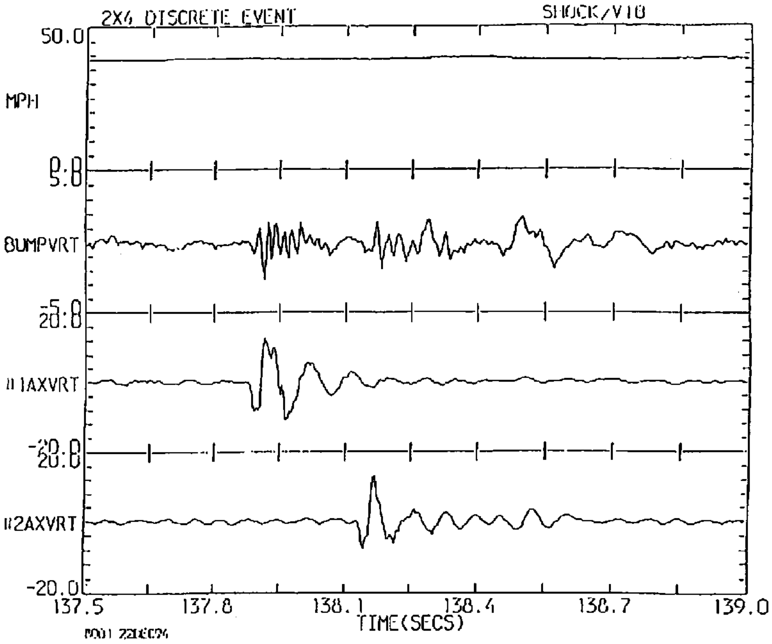


Figure 8. Vehicle dampening associated with 45 mm (1.8 in.) bump in road.

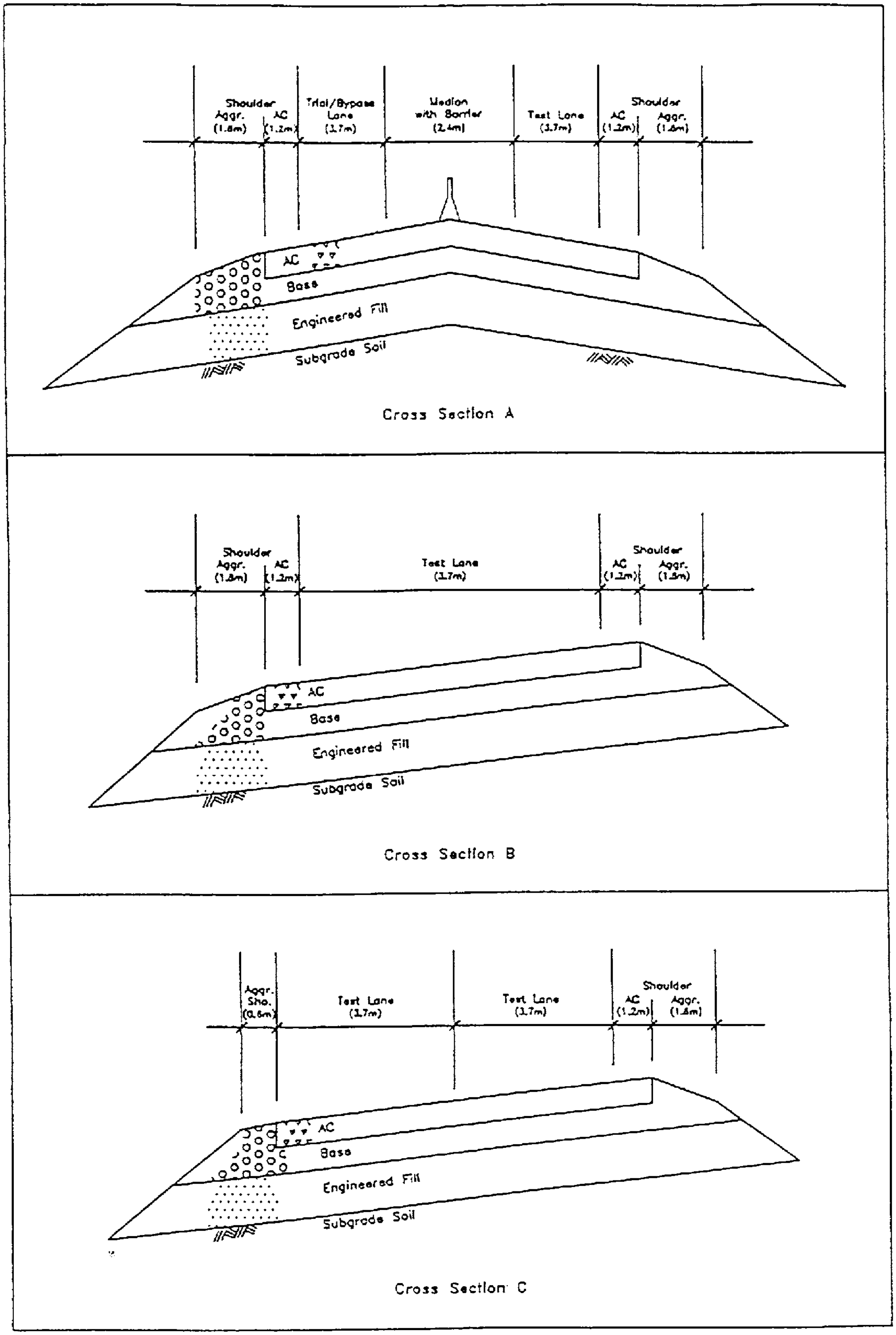


Figure 9. Three alternative pavement cross sections considered.

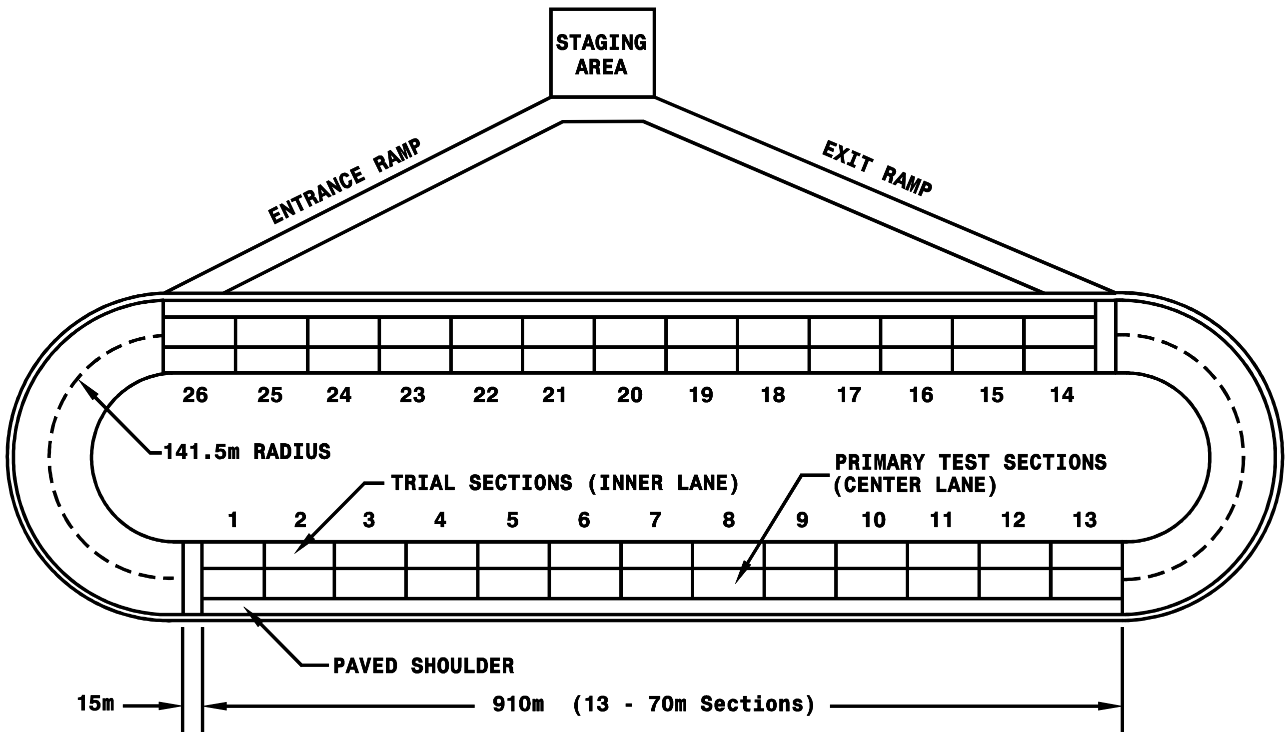


Figure 10. Layout of test track (not to scale).

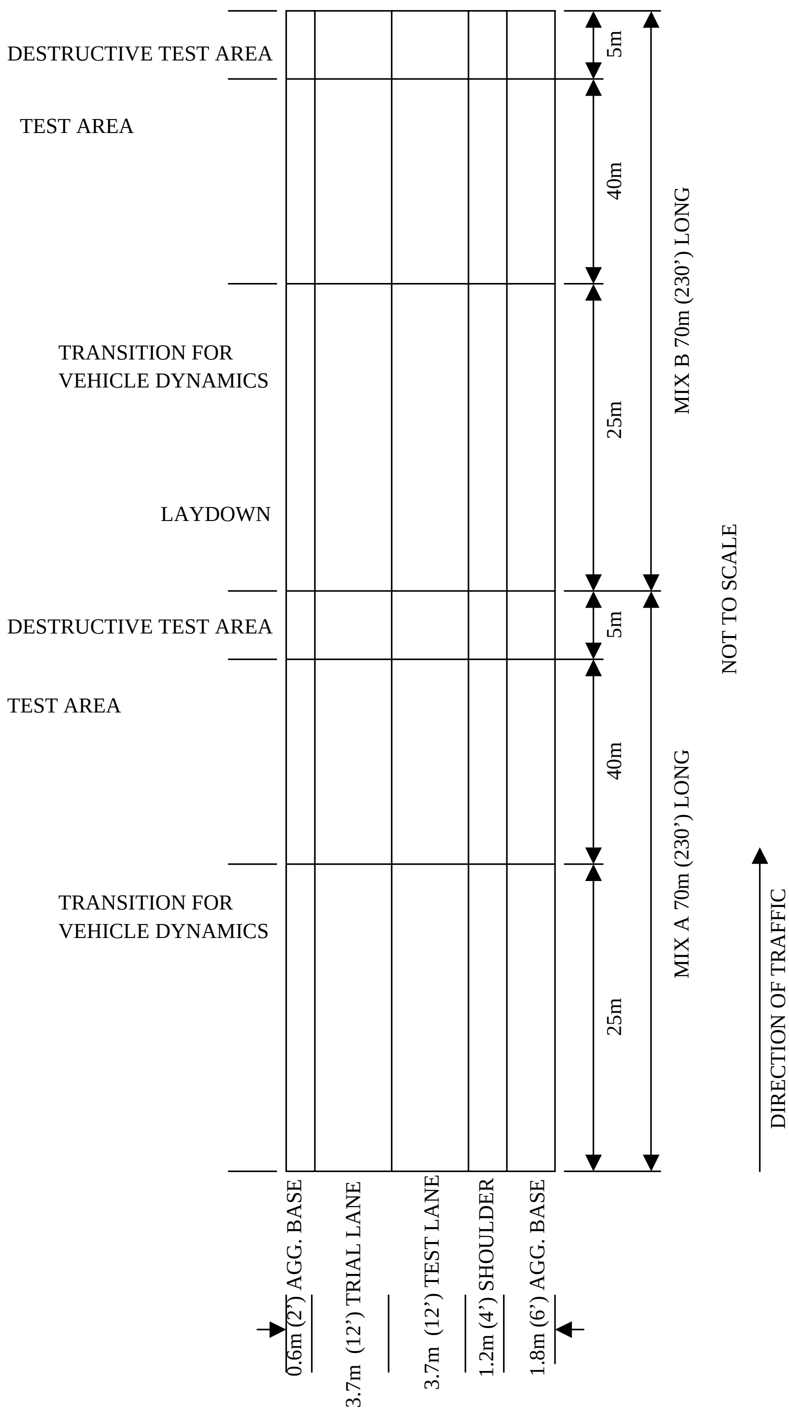


Figure 11. Test section dimensions.

Figure 12. Driverless triple-trailer test trucks.

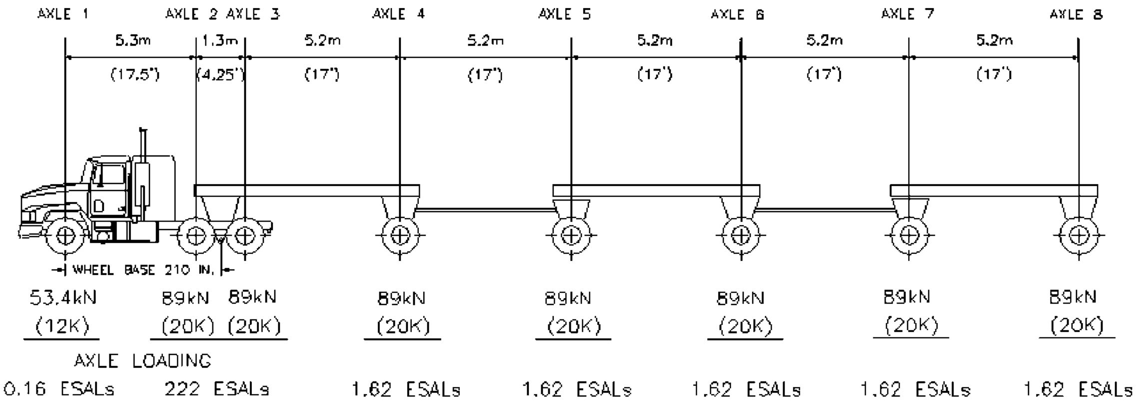


Figure 13. Tractor/trailer configuration.

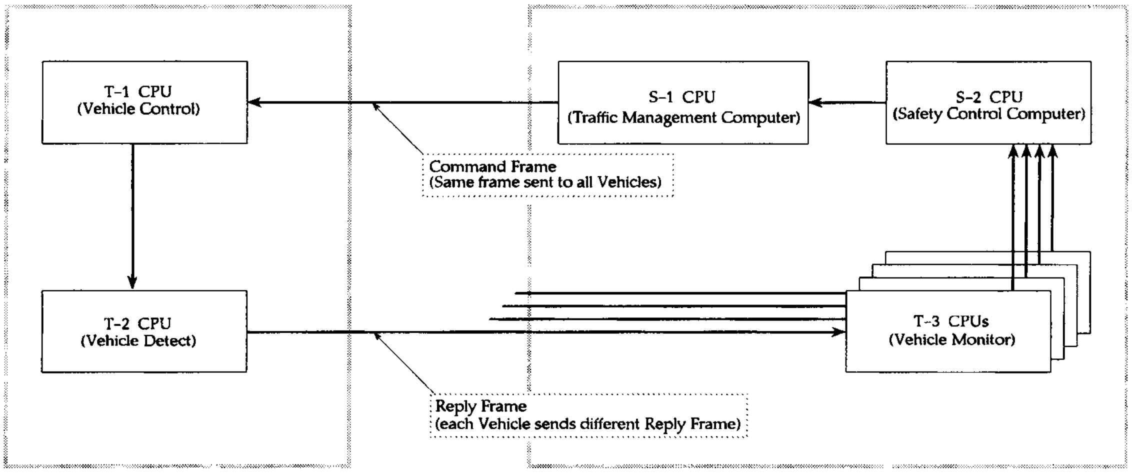


Figure 14. System block diagram.

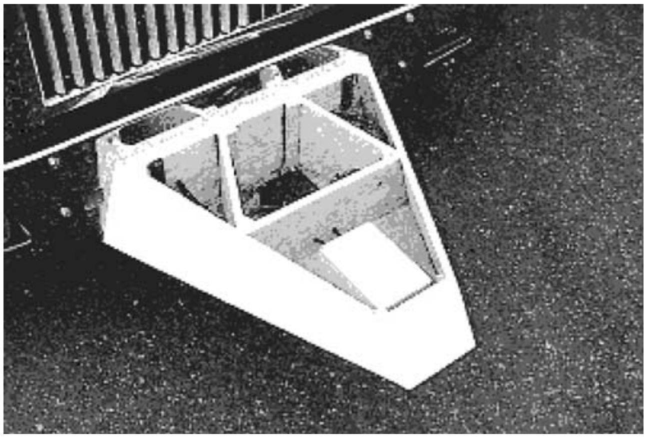


Figure 15. Antenna on truck senses lateral position.

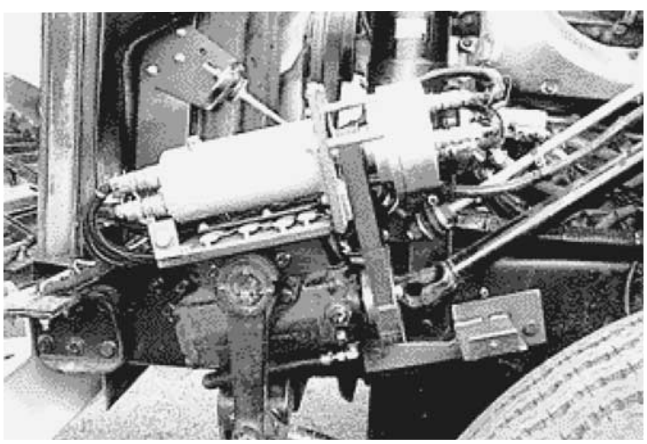


Figure 16. Stepper motor controls steering gear box.

Figure 17. WesTrack control room computers.

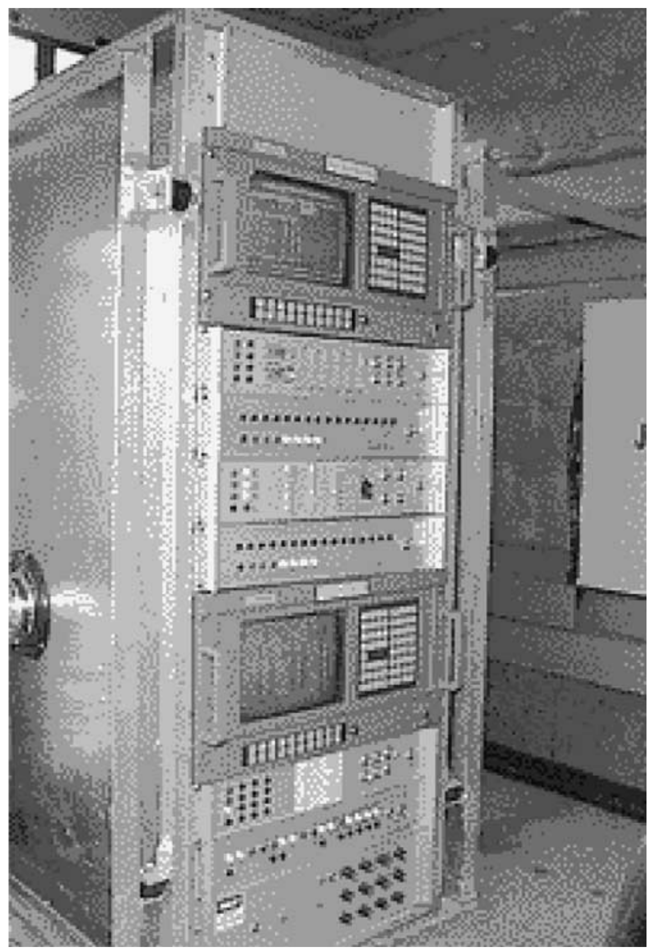


Figure 18. Two on-board computers used for vehicle control and real-time truck health monitoring.

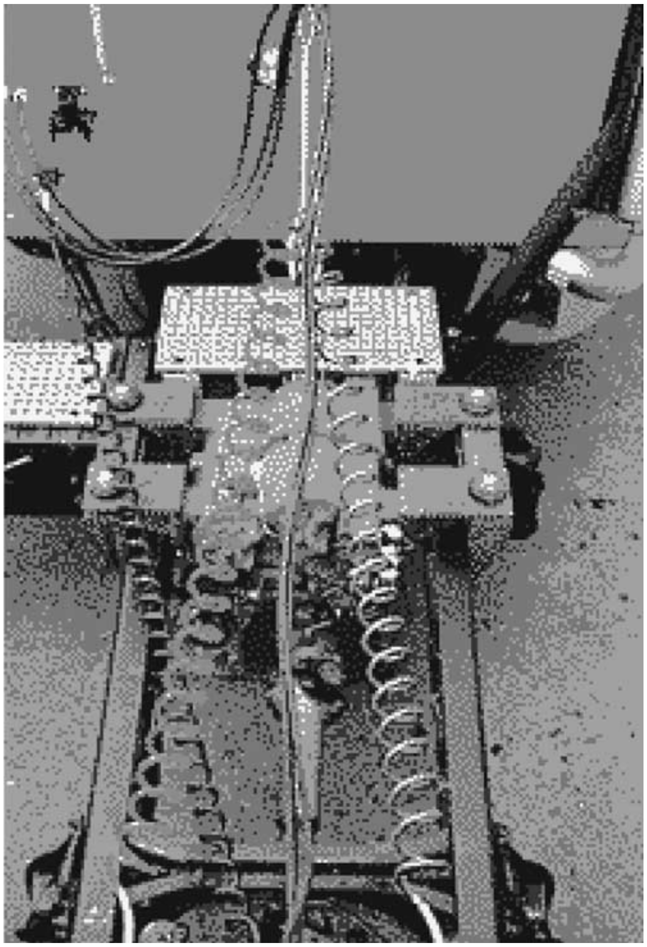


Figure 19. Electronically-controlled automated transmission.

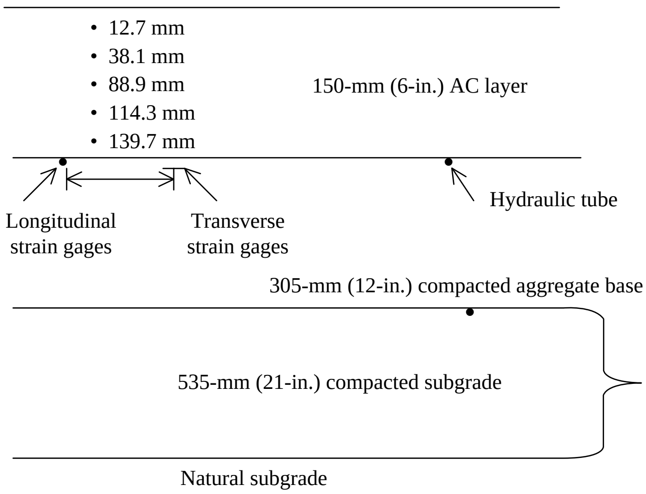


Figure 20. Thermocouple and strain gage location with depth (1 in. = 25.4 mm).

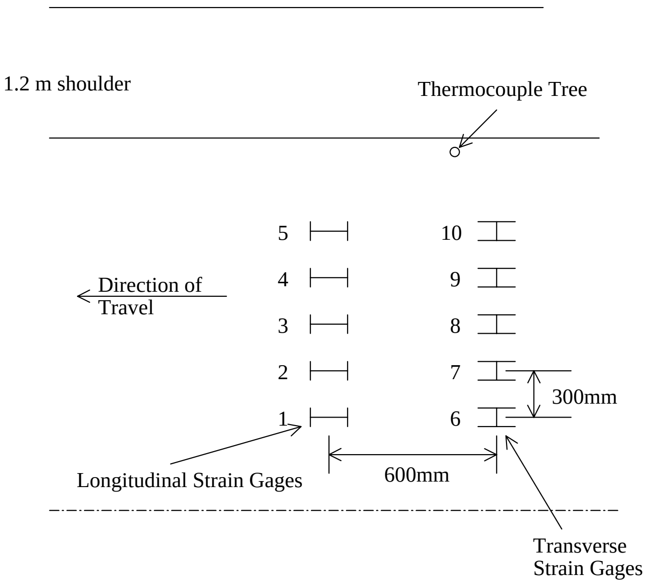


Figure 21. Thermocouple and strain gage location—plan view (1 in. = 25.4 mm).

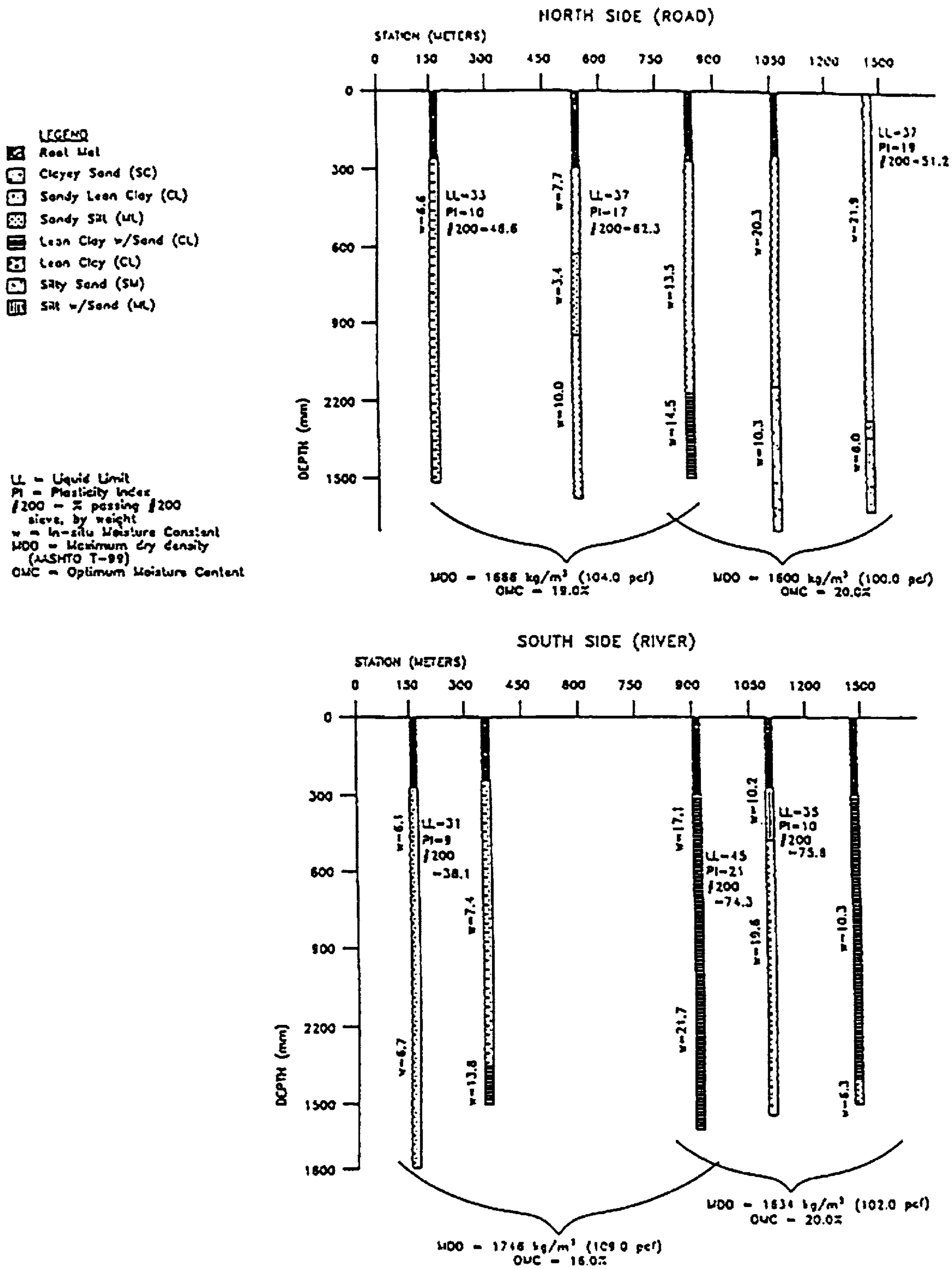


Figure 22. Test pit logs on existing subgrade material (1 in. = 25.4 mm).

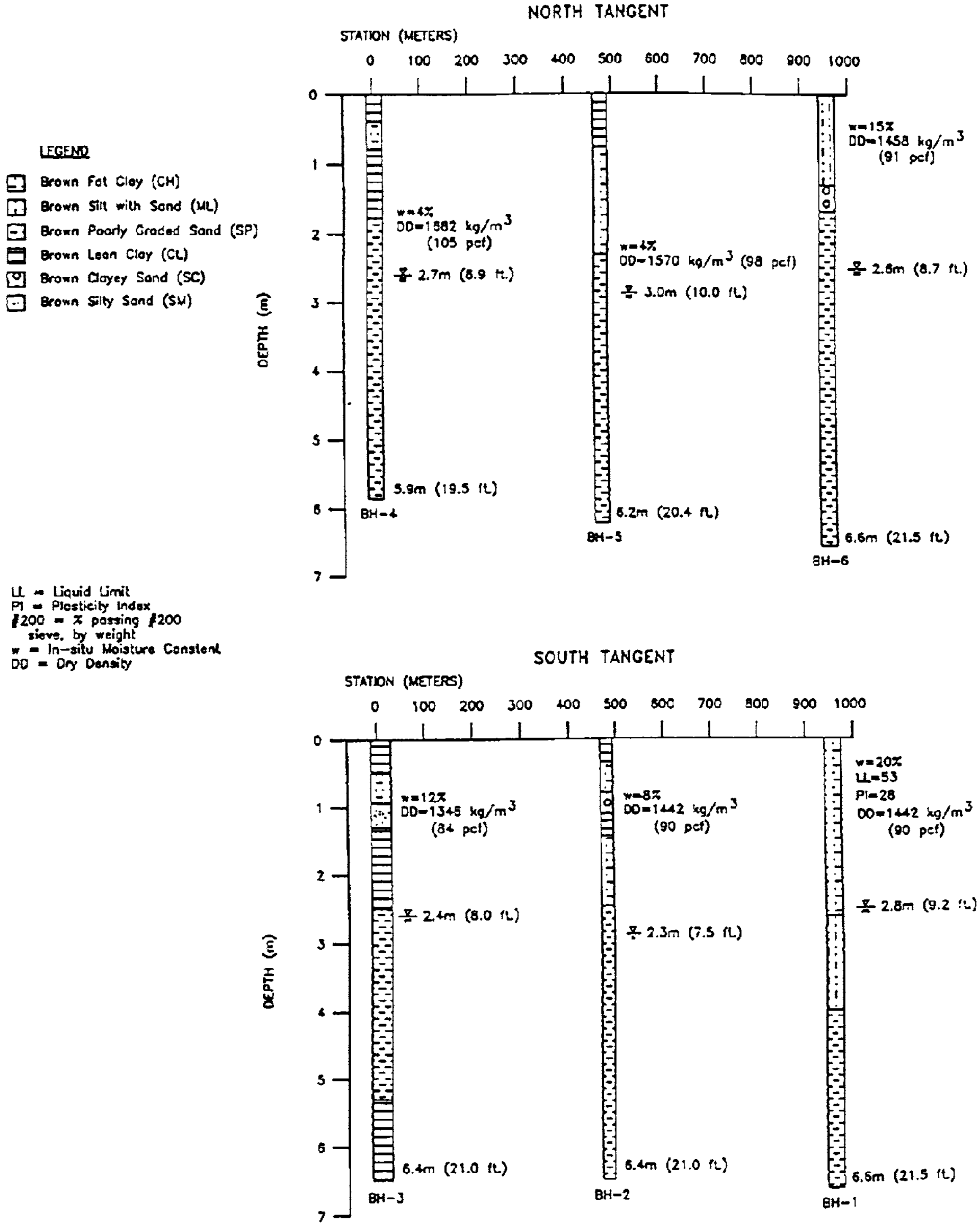


Figure 23. Results of subgrade soil boring logs (1 ft = 0.305 m).

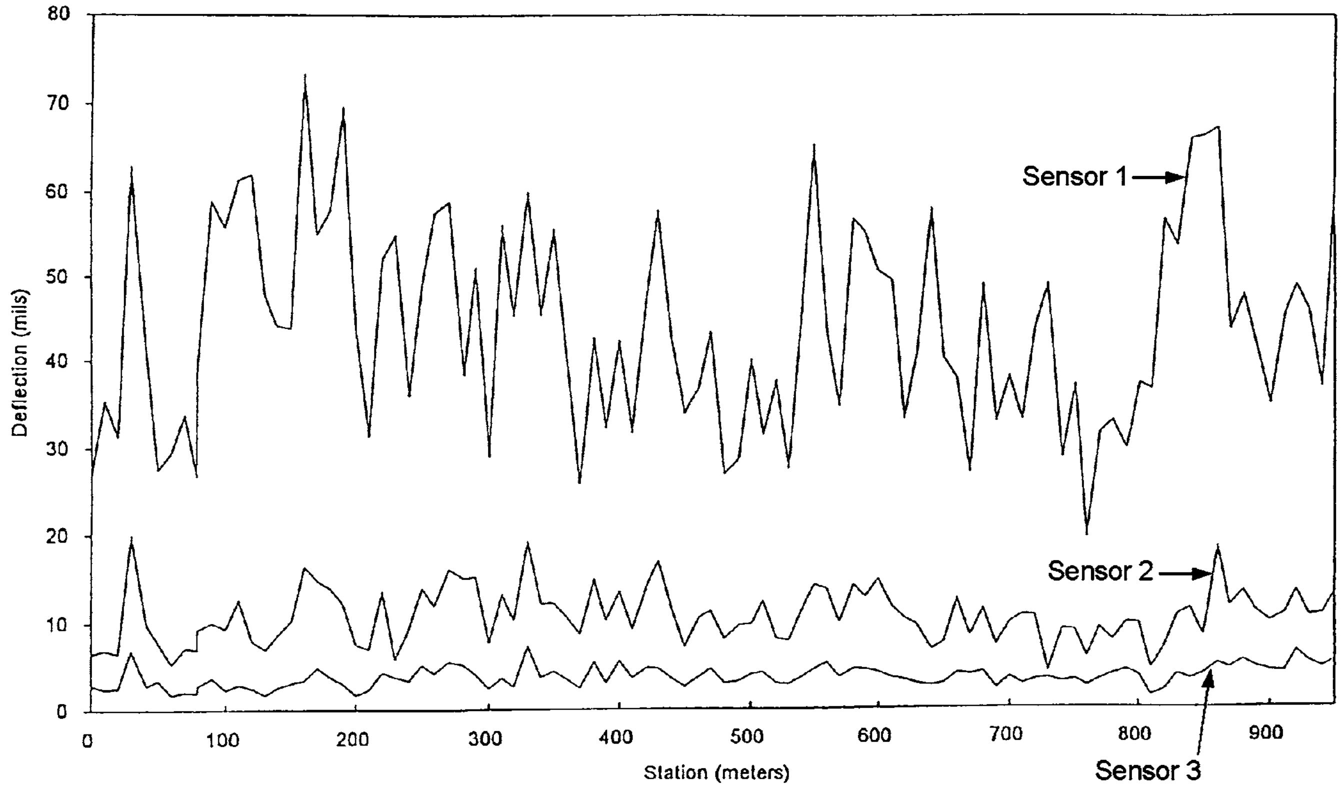


Figure 24. *Deflection plots, south tangent—sensors 1, 2, and 3, February 1995 (1 mil = 25.4 microns, 1 ft = 0.305 m).*

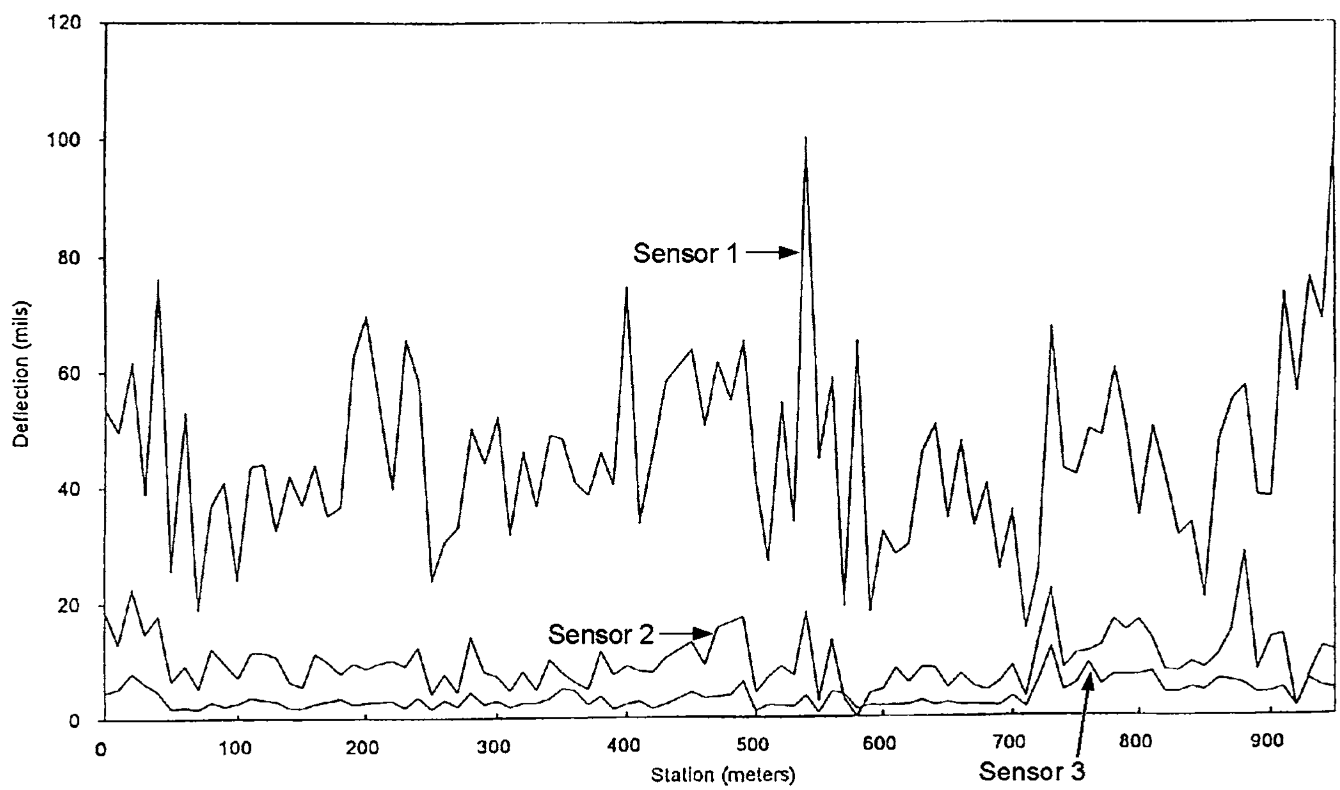


Figure 25. *Deflection plots, north tangent—sensors 1, 2, and 3 February 1995 (1 mil = 25.4 microns, 1 ft = 0.305 m).*

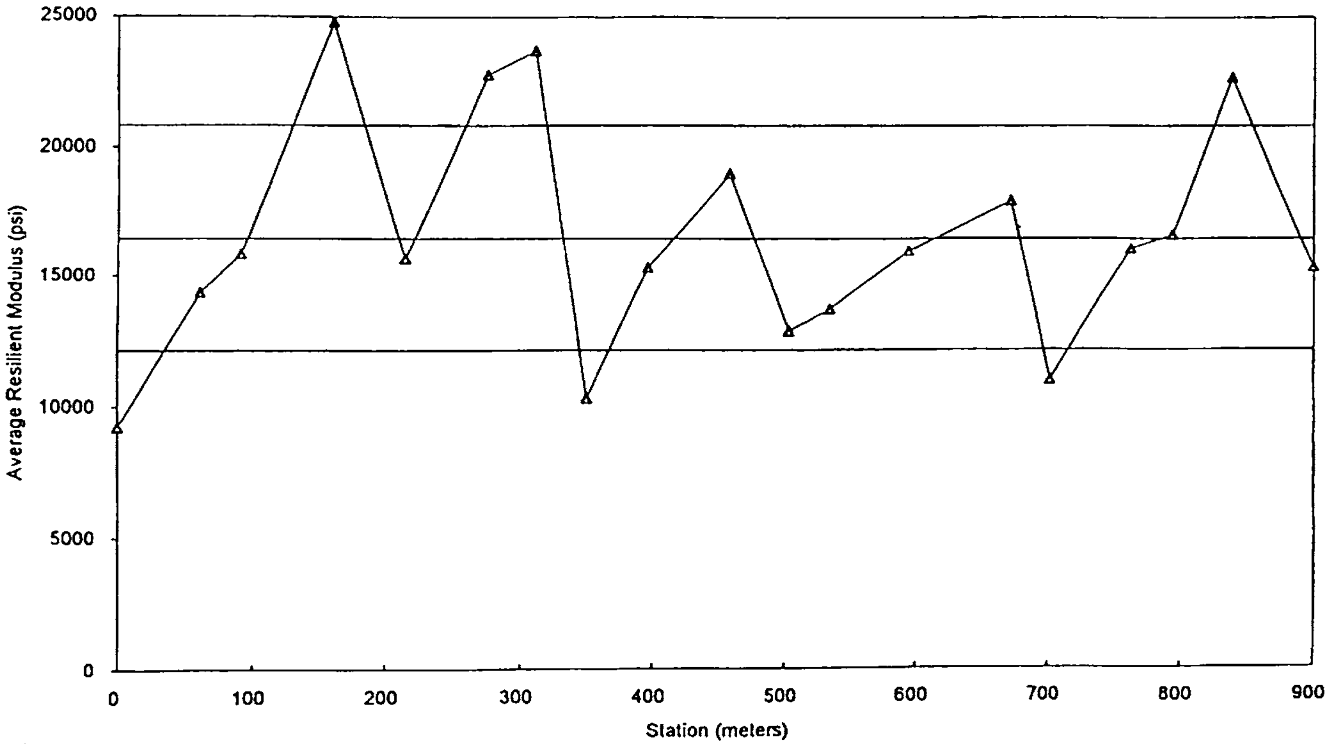


Figure 26. Subgrade soil resilient modulus, north tangent, October 1994 (1 pis = 6.9 kPa, 1 ft = 0.305 m).

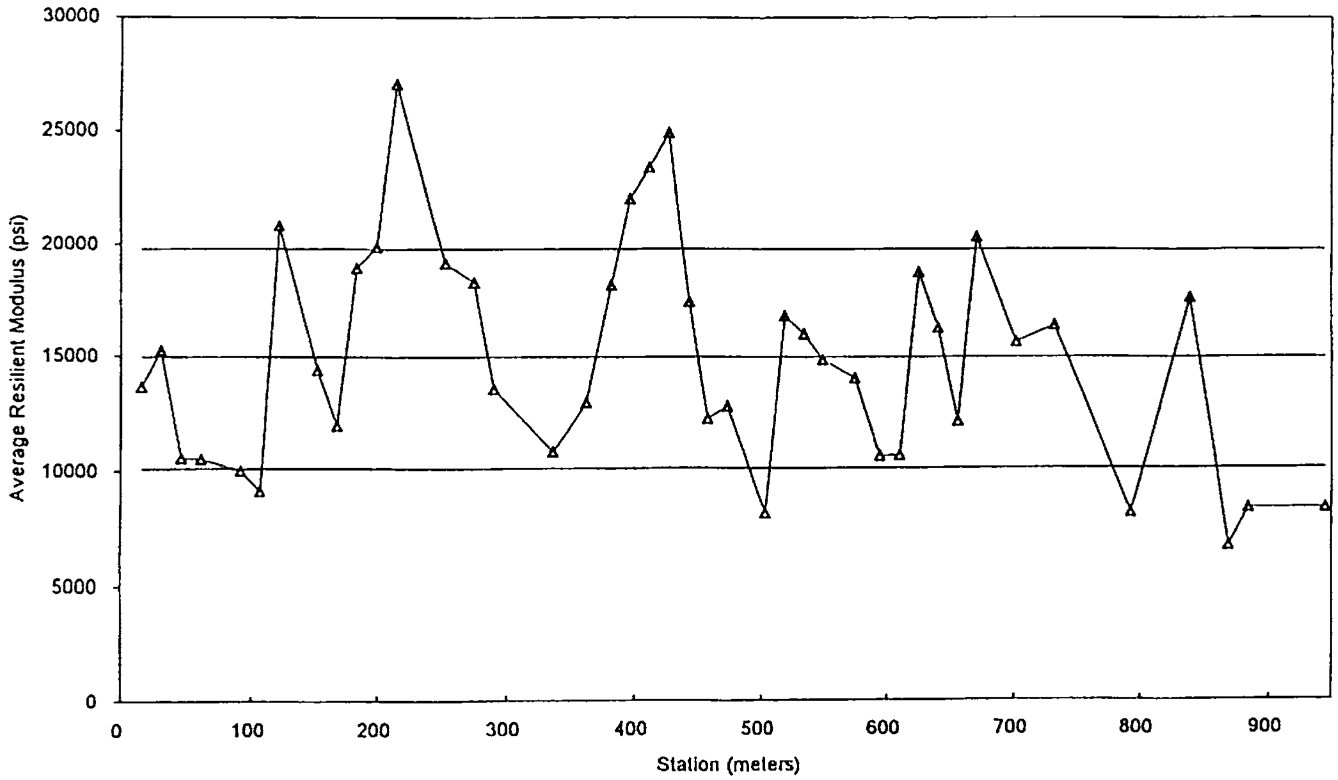


Figure 27. Subgrade soil resilient modulus, south tangent, October 1994 (1 psi = 6.9 kPa, 1 ft = 0.305 m).

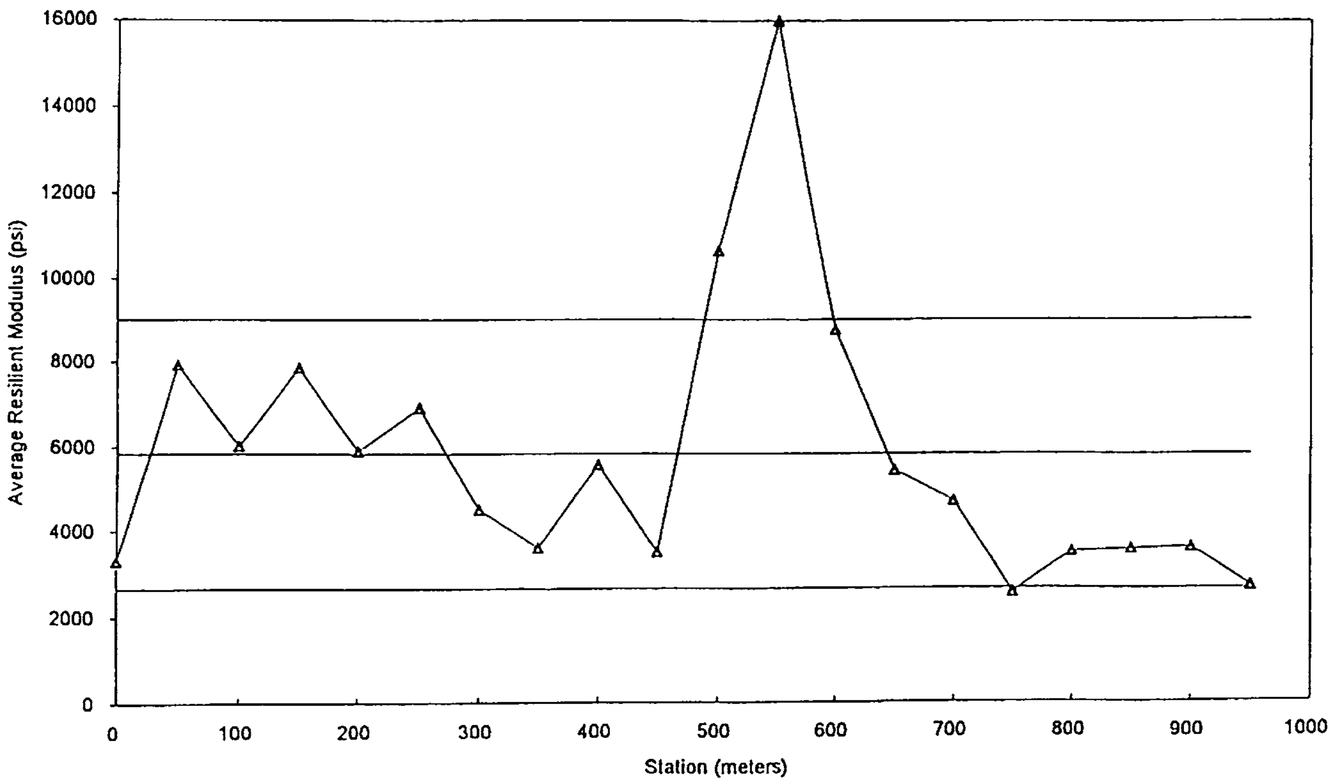


Figure 28. Subgrade soil resilient modulus, north tangent, February 1995 (1 psi = 6.9 kPa, 1 ft = 0.305 m).

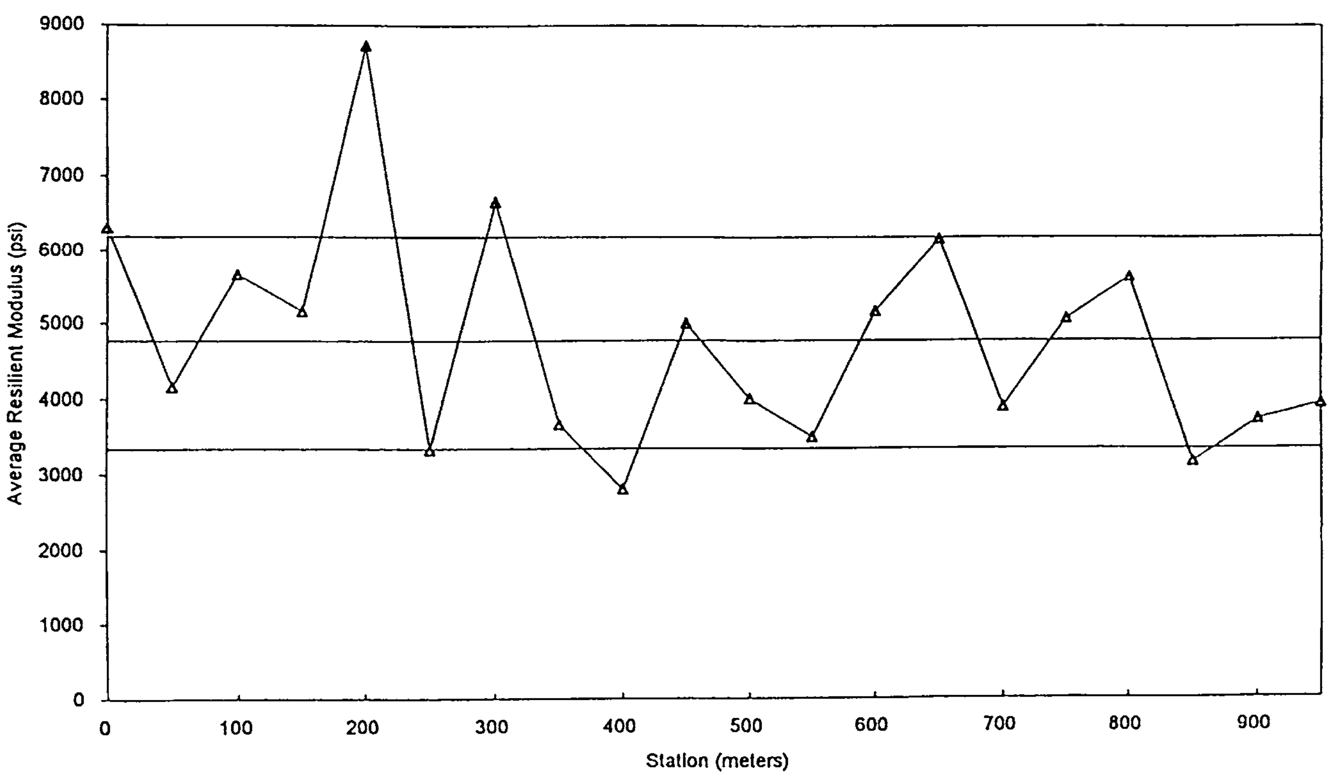


Figure 29. Subgrade soil resilient modulus, south tangent, February 1995 (1 psi = 6.9 kPa, 1 ft = 0.305 m).

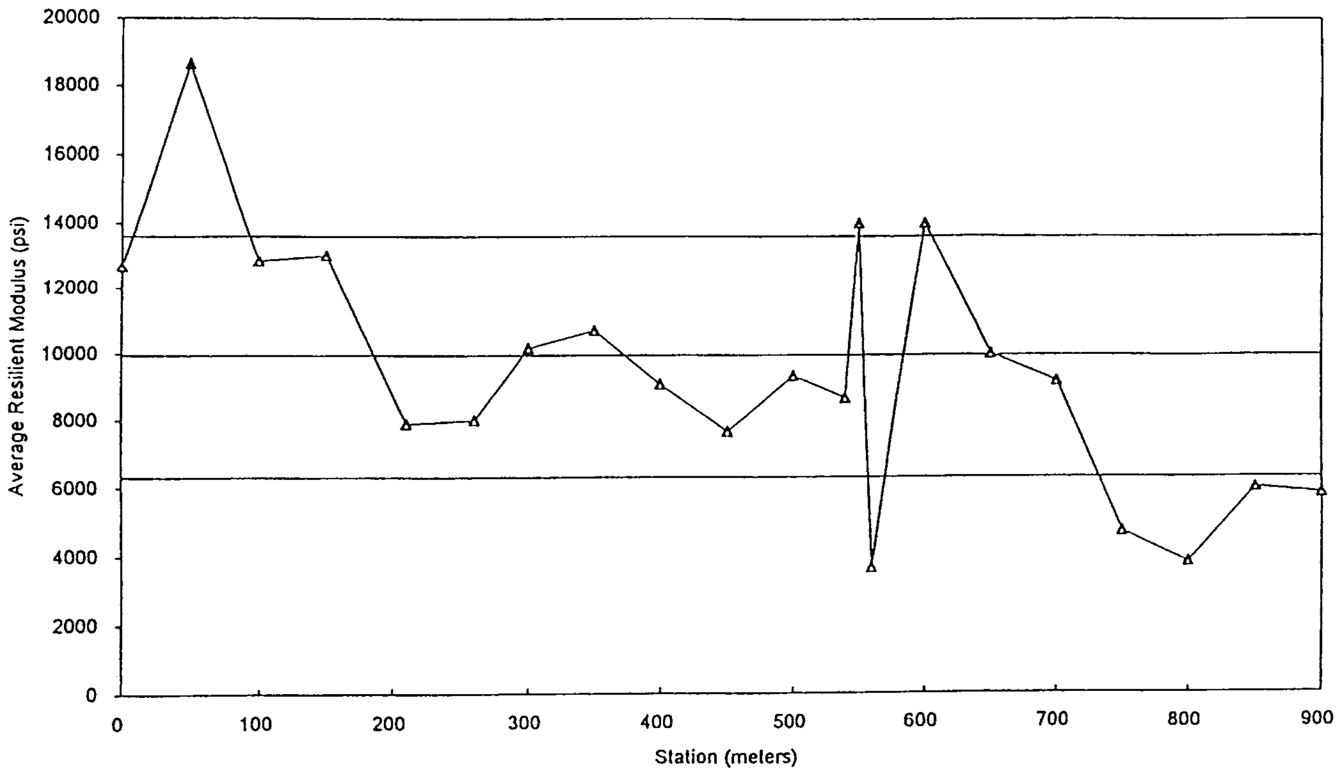


Figure 30. Subgrade soil resilient modulus, north tangent, April 1995 (1 psi = 6.9 kPa, 1 ft = 0.305 m).

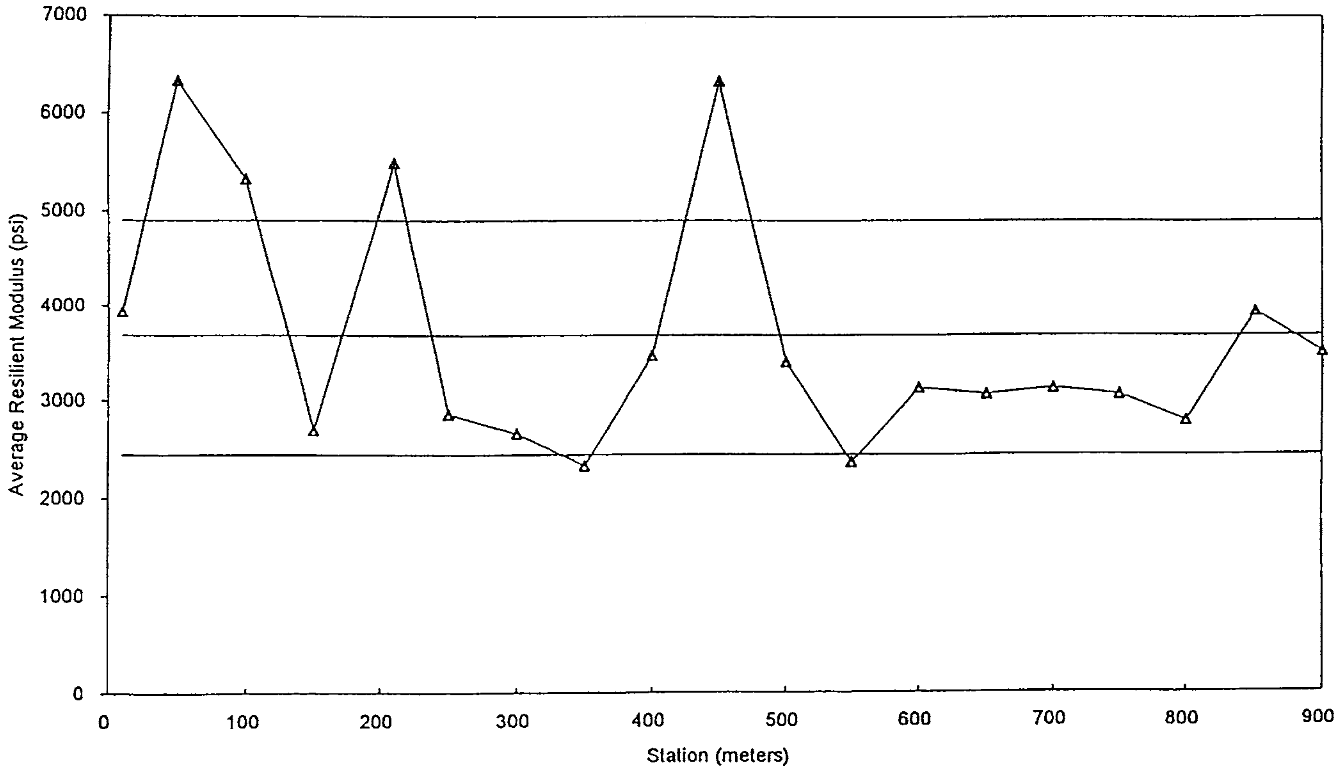


Figure 31. Subgrade soil resilient modulus, south tangent, April 1995 (1 psi = 6.9 kPa, 1 ft = 0.305 m).

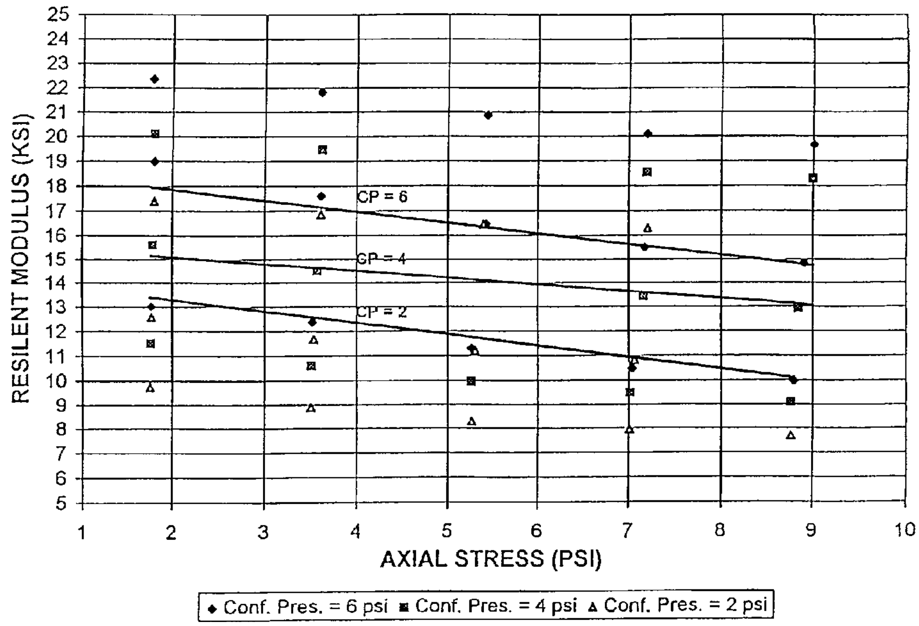


Figure 32. Graph of resilient modulus test results for recompacted soil (engineered fill) along north tangent (May 1995) (1 psi = 6.9 kPa).

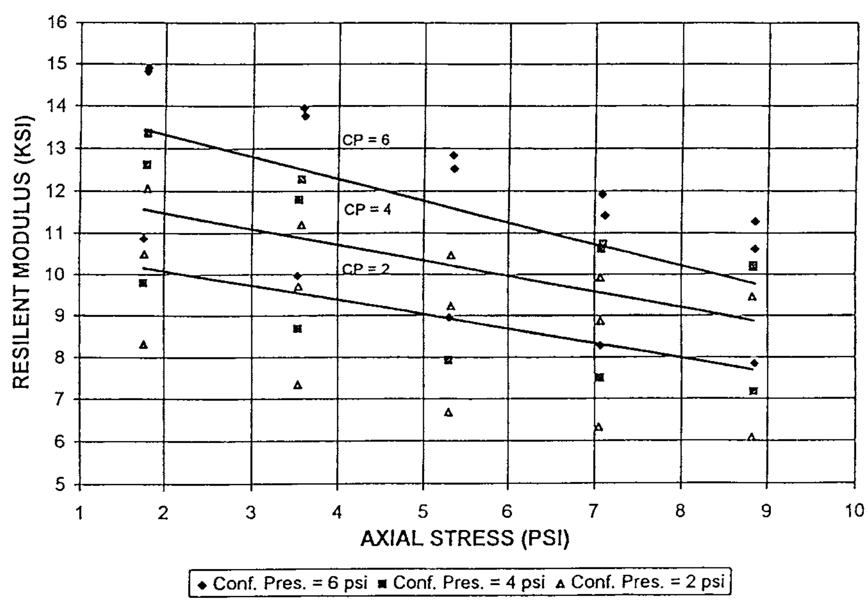


Figure 33. Graph of resilient modulus test results for recompacted soil (engineered fill) along south tangent (May 1995) (1 psi = 6.9 kPa).

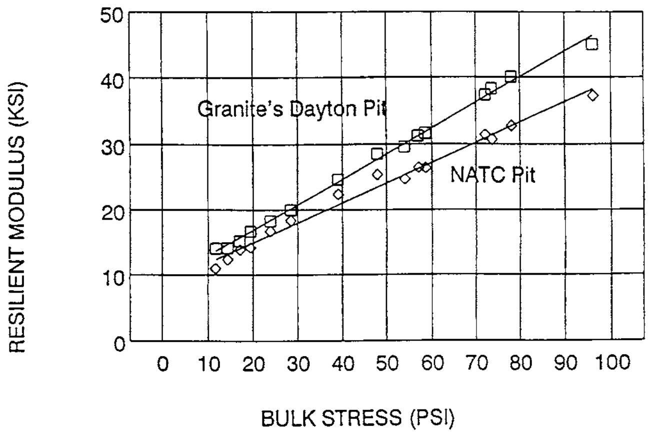


Figure 34. Resilient modulus versus bulk stress relationships for the two alternate base course materials (April 1995) (1 psi = 6.9 kPa).

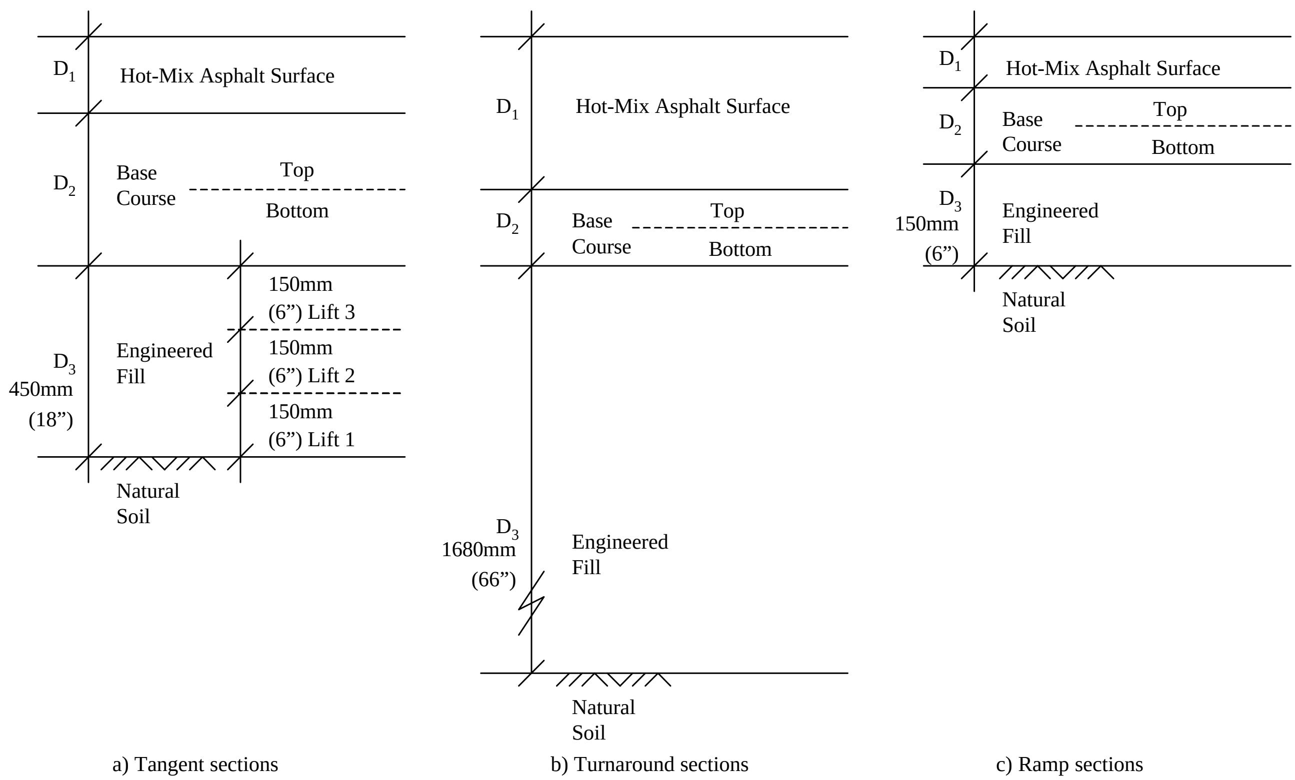


Figure 35. Cross sections for three WesTrack pavement structures.

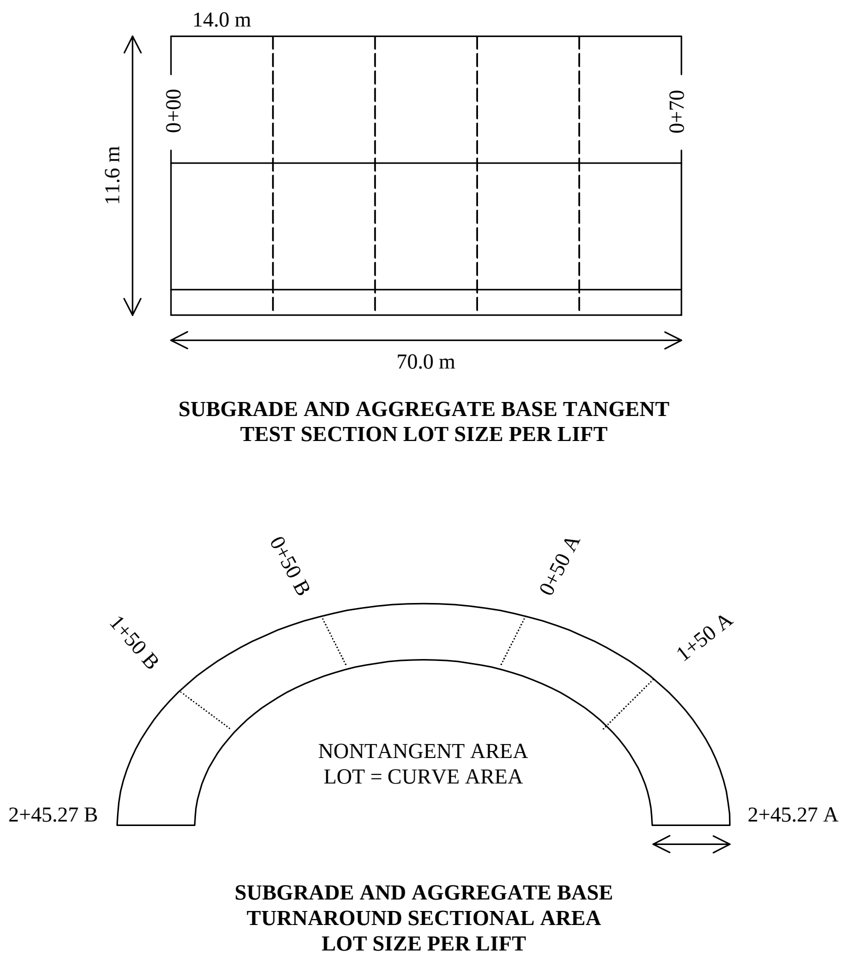
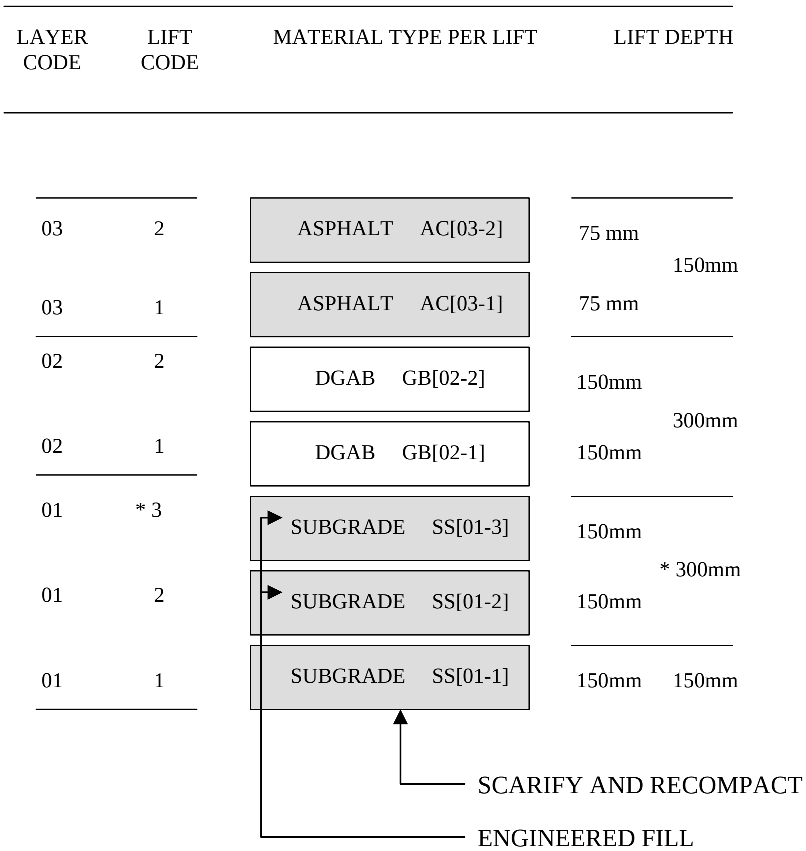


Figure 36. Experimental test section layout tests per lot (1 ft = 0.305 m).



* SUPERELEVATED TURNAROUND SECTIONS WILL HAVE UP TO 10 LIFTS.

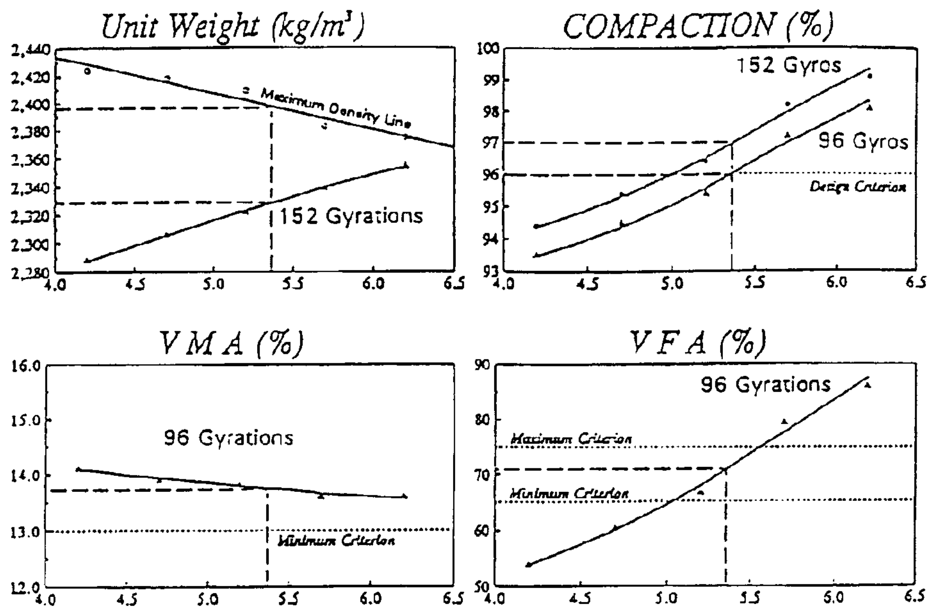
Figure 37. Experimental test section phases layer numbering system (1 in. = 25.4 mm).

FHWA **SUPERPAVE ™** OTA

LEVEL 1 MIX DESIGN

PROJECT #9511B, Blend #(16A-2.0)

Critical (Fine) Mix, 19mm Surface



Asphalt Cement Content, Percent by Total Weight of Mix

			COMPACTION				
P_{asp} of Total Mix	Unit Wt. kg/m³	Max.Den. kg/m³	Initial (%)	Design (%)	Maximum (%)	VMA (%)	VFA (%)
4.2	2,288	2,424	87.6	93.5	94.4	14.1	53.9
4.7	2,306	2,418	88.4	94.5	95.4	13.9	60.4
5.2	2,322	2,409	89.2	95.4	96.4	13.8	66.7
5.7	2,339	2,383	90.7	97.2	98.2	13.6	79.4
6.2	2,354	2,375	91.4	98.1	99.1	13.6	86.0

VALUES AT DESIGN AC CONTENT

5.4	2329	2397	89.8	96.0	97.0	13.7	71.0
Design Criteria =			< 89	= 96	< 98	> 13	65-75

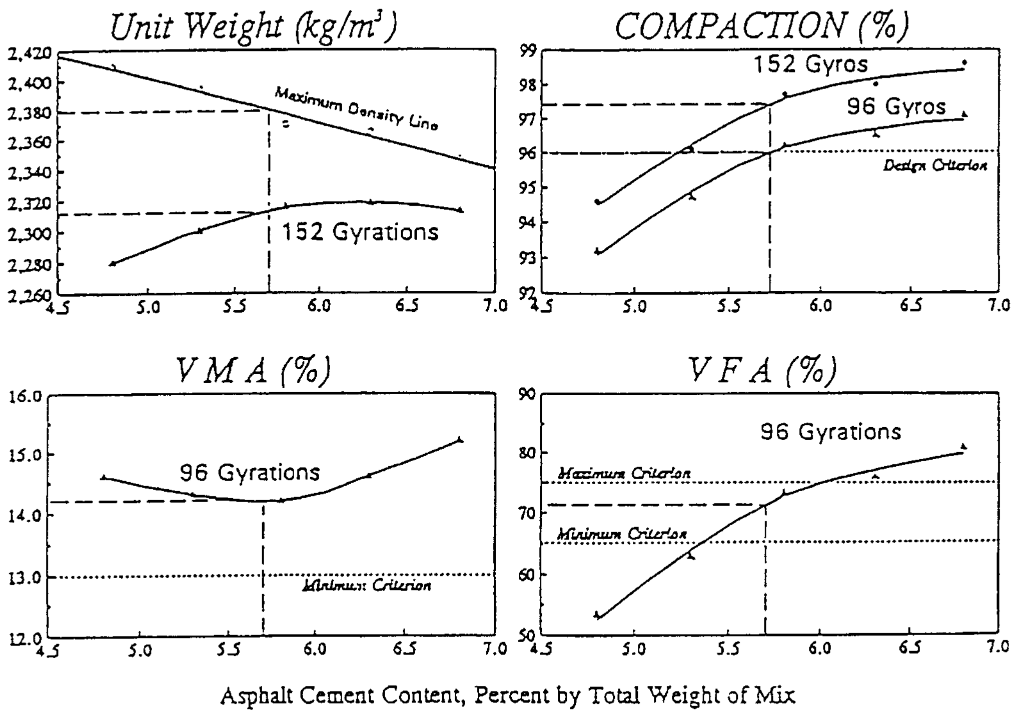
Figure 38. Fine-graded mixture used on original construction (1 lb/ft³ = 16.1 kg/m³).

FHWA **SUPERPAVE ™** OTA

LEVEL 1 MIX DESIGN

PROJECT #9511B, Blend #(18A-2.5)

Non-Critical (Coarse) Mix, 19mm Surface



			COMPACTION				
P_{asp} of Total Mix	Unit Wt. kg/m³	Max.Den. kg/m³	Initial (%)	Design (%)	Maximum (%)	VMA (%)	VFA (%)
4.8	2,280	2,409	84.0	93.2	94.6	14.6	53.4
5.3	2,300	2,394	85.4	94.7	96.1	14.3	62.9
5.8	2,315	2,370	86.6	96.2	97.7	14.2	73.2
6.3	2,318	2,365	86.9	96.5	98.0	14.6	76.0
6.8	2,313	2,347	88.0	97.1	98.6	15.2	80.9

VALUES AT DESIGN AC CONTENT

5.7	2312	2380	86.3	96.0	97.3	14.2	71.0
Design Criteria =			< 89	= 96	< 98	> 13	65-75

Figure 39. Coarse-graded mixture used on original construction (1 lb/ft³ = 16.1 kg/m³).

TABLE 4 Typical construction variability—standard deviation

Property	Typical Variability	
	Representative Range	Representative Value
A. Asphalt content	0.15-0.45	0.30
B. Sieve analysis 19-mm (3/4-in.)	1.5-4.0	2.7
12.5-mm (1/2-in.)	1.5-4.0	2.7
9.5-mm (3/8-in.)	2.0-4.5	3.3
4.75 mm (No. 4)	2.5-4.0	3.3
2.36 mm (No. 8) and 2 mm (No. 10)	2.0-3.5	2.8
1.18 mm (No. 16)		
0.600 mm (No. 30)	1.5-3.5	2.0
0.300 mm (No. 50)	1.2-1.9	1.5
0.150 mm (No. 100)	1.0-1.4	1.1
0.075 mm (No. 200)	0.5-1.5	0.9
C. Marshall properties stability	150-400	275
Flow	1.5-4.0	3.0
Air voids	0.3-1.2	0.7
D. In-place air voids	1.0-2.5	1.5

TABLE 5 Review of test track geometrics

Test Track	Total Length (Miles)	No. of Sections	Geometrics	No. of Lanes	Lane Width (Feet)
AASHO Road Test	12.7	820	A,P,F	2	12
Penn State	1.0	21	T,C,F,A,P	1	12
Mn/Road-Interstate	3.5	23	T,F,A,P	2	12
Mn/Road-Closed Loop	1.5	18	T,F,A,U	2	12
Washington State Circular Track	85' Dia.	Variable	C,F,A	1	Dual truck tire
San Diego	0.5	32	Base thickness Exp.	2	12
FS/WES/FHWA in MS	0.7	15	T,C,F,G,A,U	2	12
FS CTI	0.5	11	T,C,F,A,U	2	12
FS Commensurate Share	0.6	4	T,C,F,G,U	3	12-16
LTPP	–	–	–	–	–
WesTrack	1.8	26	T,F,A	2	12

T = Tangent Sections
C = Curved Sections
F = Flat
G = Grade
A = Asphalt
P = Portland Cement Concrete
U = Unpaved (Aggregate or Native)

1 ft = 0.305 m
1 mi = 1.61 km

TABLE 6 Review of test section lengths

Test Track	Section Length (Feet)	Length of Transition (Feet)	Sympathetic Failures	Test Speed (MPH)
AASHO Road Test	100 (Asphalt)	40	Probably	35
Penn State	180	30	?	55-T/45-C
Mn/Road-Interstate	500	50-90	?	65 (Variable)
Mn/Road-Closed Loop	500	--	?	35
Washington State Circular Track	20-50	5-10	Yes (spoke interaction)	20-30
San Diego	200	--	?	Variable
FS/WES/FHWA in MS	125*	0	Yes	20-25
FS CTI	Varied	Varied	No	15-25
FS Commensurate Share	300	500	No	15-20
LTPP	500	0	--	Varies
WesTrack	230	100	--	40

*370 125-ft lengths including aggregate surfaced roads.

1 ft = 0.305 m
1 mph = 1.61 km/hr

TABLE 7 Review of test vehicles

Test Track	No. of Vehicles Per Lane	Type of Vehicle
AASHO Road Test	Varied	2-axle trucks to 5-axle tractor/semi-trailers
Penn State	Varied	Trucks with one or more trailers
Mn/Road-Interstate	Interstate traffic	Varied
Mn/Road-Closed Loop	1	T/Tr, 5Axle, 18-wheel circular
Washington State Circular Track	3 dual tire sets	3 spokes, 120° apart
San Diego	Highway traffic	0-15% trucks
FS/WES/FHWA in MS	1	T/Tr, 5 Axle, 18-wheel
FS CTI	1	T/Tr, 5 Axle, 18-wheel
FS Commensurate Share	1	T/Tr, 5 Axle, 18-wheel
LTPP	Highway traffic	Varied
WesTrack	4	Triple, 8 Axle, 30-wheel

TABLE 8 Experiment design for original 26 WesTrack sections

Design Air Void Content	Aggregate Gradation Designation								
	Fine			Fine Plus			Coarse		
	Design Asphalt Contents (%)								
(%)	Low	Opt.	High	Low	Opt.	High	Low	Opt.	High
Low		04	18		12	09/12		23	25
Medium	02	01/15	14	22	11/19	13	08	05/24	07
High	03/16	17		10	20		26	06	

*Numbers shown in each cell represent WesTrack section numbers.

TABLE 9 Plan view of test section layout according to randomization plan

	North Tangent												
Block No.	6				3				2				
Aggr. Gradation	Coarse				Fine Plus				Fine				
P_{asp}/V_{air} Replicate	LH	HL	MM2	ML	LM	HL1	MH	MM1	HL	MH	LH2	HM	MM2
Mix Placement Sequence	26	25	24	23	13	12	11	10	09	08	07	06	05
Section Designation	26	25	24	23	22	21	20	19	18	17	16	15	14

WEST TURN

EAST TURN

Section Designation	01	02	03	04	05	06	07	08	09	10	11	12	13
Mix Placement Sequence	01	02	03	04	19	20	21	22	14	15	16	17	18
P_{asp}/V_{air} Replicate	MM1	LM	LH1	ML	MM1	MH	HM	LM	HL2	LH	MM2	ML	HM
Aggr. Gradation	Fine				Coarse				Fine Plus				
Block No.	1				5				4				
	South Tangent												

Notations:
- Block No.: Paving block sequence.
- Aggr. Gradation: Code for aggregate gradation used in asphalt mix.
- P_{asp}/V_{air} Replicate: Asphalt content (low/med/hi), air void content (low/med/hi) and number of replicate.
- Mix Placement Sequence: Sequence number for preparation and placement of mix.
- Section Designation: Number that indicates physical location of section and track.

TABLE 10 Experiment design for original 26 WesTrack sections and replacement sections

Design Air Void Content	Aggregate Gradation Designation								
	Fine			Fine Plus			Coarse		
	Design Asphalt Contents (%)								
(%)	Low	Opt.	High	Low	Opt.	High	Low	Opt.	High
Low		04	18		12	09, 21		23 (39)	25 (55)
Medium	02	01/15	14	22	11/19	13	08 (38)	05/24 (35, 54)	07 (37)
High	03/16	17		10	20		26 (56)	06 (36)	

Note: Numbers shown in each cell represent WesTrack original construction section numbers, with replacement section numbers shown in parentheses.

TABLE 11 Summary of historic peak discharges, Carson River near Fort Churchill USGS gauge 10312000

Date	Recorded Peak Discharge, ft^3/s	Approximate Recurrence Interval
1986	470	30
1963	430	28
1956	274	15
1951	222	11
1965	204	9
1983	180	8
1943	178	8
1980	175	8
1914	174	8
1982	157	7
1938	156	7

1 ft^3/s = 0.0283 m^3/s
Source: USGS
Location: Approximately 3 km (1.9 mi) downstream of project site.

TABLE 12 Instrumentation for vehicle dynamics

Channel(s)	Description of Location
1-8	Tractor and trailer axle vertical accelerations (to measure axle accelerations as the truck passes over the pavement strain gauges).
9	Tractor frame vertical acceleration (over the tandem axles to measure accelerations at the fifth wheel).
10	Vehicle speed or vehicle displacement from identification marker.

TABLE 13 Backcalculated resilient modulus—October 1994

Tangent	Resilient Modulus, kPa (psi)	
	Mean	Standard Deviation
North	113,492 (16,472)	29,902 (4,340)
South	102,985 (14,947)	33,313 (4,835)
Combined	108,239 (15,710)	36,500 (5,300)

TABLE 14 Backcalculated resilient modulus—February 1995

Tangent	Resilient Modulus, kPa (psi)	
	Mean	Standard Deviation
North	40,182 (5,832)	21,862 (3,173)
South	32,858 (4,769)	9,804 (1,423)
Combined	36,520 (5,301)	15,833 (2,298)

TABLE 15 Backcalculated resilient modulus—April 1995

Tangent	Resilient Modulus, kPa (psi)	
	Mean	Standard Deviation
North	68,618 (9,959)	25,114 (3,645)
South	25,335 (3,677)	8,468 (1,229)
Combined	46,977 (6,818)	16,791 (2,437)

TABLE 16 Summary of backcalculated resilient moduli for roadbed soil

Tangent	Resilient Modulus, kPa (psi)		
	North Tangent	South Tangent	Combined
October 1994	113,492 (16,472)	102,985 (14,947)	108,239 (15,710)
February 1995	40,182 (5,832)	32,858 (4,769)	36,520 (5,301)
April 1995	68,618 (9,959)	25,335 (3,677)	46,977 (6,818)

TABLE 17 Normalized range of roadbed soil resilient modulus near the WesTrack site

Season	Period	Average Percent of Summer Months
Winter	January - March	81
Spring	April - June	70
Summer	July - September	100
Fall	October - December	102

TABLE 18 Estimated monthly roadbed soil modulus values for WesTrack project

Season	Month	Estimated Soil Modulus Values During Year, kPa (psi)		Percent of Summer Modulus
Summer	September*	57,187	(8,300)	100
Fall	October	58,565	(8,500)	102
	November	58,565	(8,500)	102
	December	58,565	(8,500)	102
Winter	January	46,163	(6,700)	81
	February	46,163	(6,700)	81
	March	46,163	(6,700)	81
Spring	April	39,962	(5,800)	70
	May	39,962	(5,800)	70
	June	39,962	(5,800)	70
Summer	July	57,187	(8,300)	100
	August	57,187	(8,300)	100

*Note: September was selected as the first year in the sequence, because for the WesTrack project, September was targeted for the first full month of loading.

TABLE 19 Summary of resilient modulus test results on recompacted soil (engineered fill) samples from WesTrack site (May 1995)

	North Tangent - MDD=118.5pcf, w(opt)=13.0%									South Tangent - MDD=113.0pcf, w(opt)=15.0%								
	Sample 1			Sample 2			Sample 3			Sample 1			Sample 2			Sample 3		
	w(sample)		13.5%	w(sample)		13.0%	w(sample)		12.6%	w(sample)		15.5%	w(sample)		15.4%	w(sample)		14.5%
	DD(sample)		107.4	DD(sample)		107.1	DD(sample)		107.5	DD(sample)		102.5	DD(sample)		101.8	DD(sample)		104.3
	Rel Density		90.6%	Rel Density		90.4%	Rel Density		90.7%	Rel Density		90.7%	Rel Density		90.1%	Rel Density		92.3%
Seq	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)	Conf Pres (psi)	Axial Strs (psi)	Resilient Modulus (psi)
1	6.0	1.77	13,029	5.9	1.78	22,366	6.0	1.80	19,007	6.0	1.76	10,873	6.0	1.79	14,815	6.0	1.80	14,904
2	6.0	3.52	12,376	6.0	3.61	21,818	6.0	3.61	17,596	6.0	3.53	9,961	6.0	3.60	13,772	6.0	3.59	13,944
3	6.0	5.26	11,300	6.0	5.43	20,873	6.0	5.42	16,467	6.0	5.30	8,957	6.0	5.35	12,526	5.9	5.34	12,838
4	6.0	7.04	10,484	6.0	7.20	20,111	6.0	7.18	15,494	6.0	7.07	8,283	6.0	7.12	11,420	5.9	7.09	11,925
5	6.0	8.80	9,977	6.0	9.01	19,673	6.0	8.91	14,827	6.0	8.85	7,849	6.0	8.86	10,619	5.9	8.86	11,280
6	4.0	1.76	11,508	4.0	1.80	20,121	4.0	1.78	15,598	4.0	1.76	9,791	4.0	1.80	13,359	4.0	1.79	12,616
7	4.0	3.51	10,600	4.0	3.62	19,465	4.0	3.57	14,501	4.0	3.53	8,685	4.0	3.57	12,275	4.0	3.54	11,806
8	4.0	5.26	9,959							4.0	5.29	7,930	4.0	5.37	11,427			
9	4.0	7.02	9,494	4.0	7.20	18,554	4.0	7.16	13,439	4.0	7.06	7,502	4.0	7.10	10,733	4.0	7.08	10,622
10	4.0	8.77	9,090	4.0	9.00	18,305	4.0	8.85	12,924	4.0	8.84	7,166	4.0	8.84	10,166	4.0	8.84	10,215
11	2.0	1.75	9,749	2.0	1.80	17,392	2.0	1.77	12,591	2.0	1.76	8,319	2.0	1.80	12,060	2.0	1.77	10,499
12	2.0	3.50	8,878	2.0	3.61	16,855	2.0	3.53	11,676	2.0	3.53	7,346	2.0	3.57	11,203	2.0	3.54	9,710
13	2.0	5.26	8,299	2.0	5.39	16,456	2.0	5.30	11,203	2.0	5.29	6,682	2.0	5.32	10,468	2.0	5.32	9,228
14	2.0	7.01	7,954	2.0	7.21	16,294	2.0	7.06	10,826	2.0	7.05	6,319	2.0	7.07	9,918	2.0	7.07	8,868
15	2.0	8.77	7,686							2.0	8.82	6,077	2.0	8.83	9,454			

1 psi = 6.9 kPa

TABLE 20 Summary of resilient modulus test results on alternate base course samples from WesTrack site (April 1995)

	Granite's Dayton pit Base Material - MDD=133,5pcf, w(opt)-7.0%												NATC Base material - MDD=128.0pcf, w(opt)=11.0%											
	Sample 1			Sample 2			Sample 3			Avg Axial Stress (psi)	Avg Bulk Stress (psi)	Avg Res Mod (psi)	Sample 1			Sample 2			Sample 3			Avg Axial Stress (psi)	Avg Bulk Stress (psi)	Avg Res Mod (psi)
	w(sample)		7.0%	w(sample)		7.4%	w(sample)		7.1%				w(sample)		10.0%	w(sample)		10.8%	w(sample)		10.3%			
	DD(sample)		128	DD(sample)		126.3	DD(sample)		129.4				DD(sample)		125.9	DD(sample)		124	DD(sample)		124.5			
	Rel Density		95.9%	Rel Density		94.6%	Rel Density		96.9%				Rel Density		98.4%	Rel Density		96.9%	Rel Density		97.3%			
Seq	Conf Pres (psi)	Axial Strs (psi)	Res Mod (psi)	Conf Pres (psi)	Axial Strs (psi)	Res Mod (psi)	Conf Pres (psi)	Axial Strs (psi)	Res Mod (psi)				Conf Pres (psi)	Axial Strs (psi)	Res Mod (psi)	Conf Pres (psi)	Axial Strs (psi)	Res Mod (psi)	Conf Pres (psi	Axial Strs (psi)	Res Mod (psi)			
1	3.0	2.64	12,447	3.0	2.66	17,490	3.0	2.69	12,178	2.66	11.66	14,038	3.0	2.62	11,717	3.0	2.64	10,330	3.0	2.66	10,928	2.64	11.64	10,992
2	3.0	5.31	13,712	3.0	5.37	15,440	3.0	5.31	13,305	5.33	14.33	14,152	3.0	5.28	13,048	3.0	5.39	11,426	3.0	5.35	12,559	5.34	14.38	12,344
3	3.0	7.95	15,021	3.0	7.99	15,730	3.0	7.95	14,789	7.96	16.96	15,180	3.0	7.92	14,556	3.0	8.10	12,702	3.0	8.10	14,282	8.04	17.08	13,847
4	5.0	4.43	16,101	5.0	4.45	18,665	5.0	4.41	15,300	4.43	19.43	16,689	5.0	4.37	15,038	5.0	4.45	13,166	5.0	4.44	14,351	4.42	19.44	14,185
5	5.0	8.82	17,937	5.0	8.93	19,207	5.0	8.95	17,806	8.90	23.94	18,317	5.0	8.83	17,526	5.0	8.96	15,414	.50	8.91	17,215	8.90	23.92	16,718
6	5.0	13.30	19,706	5.0	13.50	20,282	5.1	13.47	19,981	13.42	28.48	19,990	5.0	13.36	19,356	5.0	13.46	16,646	5.0	13.48	19,018	13.43	28.47	18,340
7	10.0	8.87	24,159	10.0	9.02	25,905	10.1	9.04	23,896	8.98	39.03	24,653	10.0	8.89	23,331	10.1	8.99	20,735	10.0	9.03	23,109	8.97	39.06	22,392
8	10.0	17.85	28,591	10.0	18.02	28,433	10.0	17.95	28,661	17.94	47.94	28,562	10.0	17.85	26,369	10.0	18.03	23,490	10.0	17.97	26,330	17.95	47.98	25,396
9	10.0	26.80	31,525	10.0	26.92	30,560	10.1	26.89	31,678	26.87	56.97	31,254	10.1	26.91	27,439	10.0	27.07	24,447	10.0	27.06	27,425	27.01	57.16	26,437
10	15.0	8.92	28,711	15.0	9.01	31,392	15.0	8.94	28,655	8.96	53.96	29,586	15.0	8.95	25,445	15.1	9.05	23,142	15.0	9.01	25,434	9.00	54.13	24,674
11	15.0	13.37	31,178	15.0	13.51	32,368	15.1	13.52	31,335	13.47	58.59	31,627	15.1	13.35	27,190	15.1	13.50	24,839	15.0	13.44	27,121	13.43	58.66	26,383
12	15.0	26.83	37,313	15.0	27.01	37,269	15.1	27.11	37,512	26.98	72,12	37,365	15.1	26.88	32,285	15.1	26.99	29,595	15.0	27.03	32,330	26.97	72.15	31,403
13	19.9	13.44	37,778	20.0	13.59	39,543	20.1	13.62	37,584	13.55	73.55	38,302	20.0	13.50	31,443	20.1	13.62	29,261	20.0	13.58	31,421	13.57	73.71	30,708
14	20.0	17.87	39,591	20.0	18.11	41,238	20.1	18.09	39,484	18.01	78.08	40,104	20.0	17.91	33,343	20.2	18.03	31,335	20.0	18.01	33,555	17.98	78.14	32,744
15	20.0	35.76	45,318	20.1	35.90	44,434	20.1	36.13	45,211	35.93	96.12	44,988	20.0	35.80	37,836	20.1	36.03	35,525	20.1	35.97	38,274	35.93	96.18	37,212

Notations: MDD = Maximum dry density, DD = Dry density, w = Moisture content, w(opt) = Optimum moisture content

1 psi = 6.9 kPa

TABLE 21 Preliminary HMA resilient moduli (by month)

Month	HMA Concrete Resilient Modulus, kPa (psi)	
September	2.756×10^6	(400,000)
October	3.445×10^6	(500,000)
November	4.823×10^6	(700,000)
December	8.268×10^6	(1,200,000)
January	11.713×10^6	(1,700,000)
February	6.890×10^6	(1,000,000)
March	4.823×10^6	(700,000)
April	3.790×10^6	(550,000)
May	2.756×10^6	(400,000)
June	2.412×10^6	(350,000)
July	2.067×10^6	(300,000)
August	2.412×10^6	(350,000)

TABLE 22 Summary of designs for WesTrack pavement structural sections (6/30/95)

Pavement Section Identification	Projected ESAL Applications	Design Reliability (%)	Overall Standard Deviation	Target Damage	Layer Thickness Specification (mm [in])					
					Engineered Fill		Base		HMA Surface	
					(mm)	(in.)	(mm)	(in.)	(mm)	(in.)
Tangents	10,000,000	10	0.40	3.26	460	18	300	12	150	6
Turnarounds	10,000,000	90	0.42	0.30	1,680	66	150	6	300	12
Ramps	60,000	70	0.42	0.60	150	6	200	8	100	4

TABLE 23 Subgrade and aggregate base testing, tests per tangent

Tests Per Tangent				
Subgrade				
5	Nuclear density test	@ 3 lifts @	26 sections =	390 tests
2	Sandcone density	@ 3 lifts @	26 sections =	156 tests
24	FWD (8 per line)	@ 3 lines @	26 sections	
6	Moisture density relations	@ 3 lifts @	2 tangents =	36 tests
6	Hydrometer	@ 3 lifts @	2 tangents =	36 tests
6	Sieve analysis	@ 3 lifts @	2 tangents =	36 tests
6	Atterberg limits	@ 3 lifts @	2 tangents =	36 tests
6	Natural moisture content	@ 3 lifts @	2 tangents =	36 tests
3	Resilient modulus	@ 3 lifts @	2 tangents =	18 tests
2	CBR	@ 3 lifts @	2 tangents =	12 tests
2	R-values	@ 3 lifts @	2 tangents =	12 tests
2	Permeability	@ 3 lifts @	2 tangents =	12 tests
Dense Graded Aggregate Base				
5	Nuclear density test	@ 3 lifts @	26 sections =	390 tests
2	Sandcone density	@ 3 lifts @	26 sections =	156 tests
24	FWD (8 per line)	@ 3 lines @	26 sections	
6	Moisture density relations	@ 2 lifts @	2 tangents =	24 tests
6	Hydrometer	@ 2 lifts @	2 tangents =	24 tests
6	Sieve analysis (washed)	@ 2 lifts @	2 tangents =	24 tests
6	Atterberg limits	@ 2 lifts @	2 tangents =	24 tests
6	Particle size analysis	@ 2 lifts @	2 tangents =	24 tests
6	Natural moisture content	@ 2 lifts @	2 tangents =	24 tests
3	Resilient modulus	@ 2 lifts @	2 tangents =	12 tests
2	CBR	@ 2 lifts @	2 tangents =	8 tests
2	R-values	@ 2 lifts @	2 tangents =	8 tests
2	Permeability	@ 2 lifts @	2 tangents =	8 tests

Lot size = 1 (one) test section = 11.6 m x 70.0 m (38 ft x 230 ft) in area at 1 (one) lift of material.
Test material obtained from bulk sample locations.

TABLE 24 Subgrade and aggregate base testing, tests per turnaround

Tests Per Turnaround			
Subgrade			
5	Nuclear density test	@ 3-10 lifts =	50 max. tests
2	Sandcone density	@ 3-10 lifts =	20 max. tests
1	Moisture density relations	@ 3 lifts =	3 tests
1	Sieve analysis	@ 3 lifts =	3 tests
1	Atterberg limits	@ 3 lifts =	3 tests
1	Natural moisture content	@ 3 lifts =	3 tests
Dense Graded Aggregate Base			
5	Nuclear density test	@ 2 lifts =	10 tests
2	Sandcone density	@ 2 lifts =	4 tests
1	Moisture density relations	@ 2 lifts =	2 tests
1	Sieve analysis (washed)	@ 2 lifts =	2 tests
1	Atterberg limits	@ 2 lifts =	2 tests
1	Particle size analysis	@ 2 lifts =	2 tests
1	Natural moisture content	@ 2 lifts =	2 tests

Lot size [+/-] 5,600 m^2 (60,300 ft^2).
Test material obtained from bulk sample locations.

TABLE 25 Summary of acceptance requirements for subgrade and engineered fill

Layer	PWL for Individual Lots		PWL for Project		Standard Deviation for Project	
	Density	Moisture Content	Density	Moisture Content	Density	Moisture Content
Subgrade and first lift of engineered fill	55	55				
Second or top lift of engineered fill	60	60				
All layers			80	80	1.9	1.2

TABLE 26 Summary of aggregate base requirements

Requirements for Base Course Material	
Sieve Size	Specification Limits
25-mm (1-in.)	100
19-mm (3/4-in.)	90 - 100
4.75 mm (No. 4)	35 - 65
1.18 mm (No. 16)	15 - 40
0.075 mm (No. 200)	2 - 10
Plasticity index	6 max.
Liquid limit	35 max.
Resistance (R-value)	78 min.
L.A. abrasion loss, 500 rev.	45% max.
Fractured faces	50% min.

TABLE 27 Summary of gradation acceptance requirements for base course

Sieve Size	Tolerances	PWLs for Individual Lots	PWLs for Project	Standard Deviation for Project
37.5-mm (1.5-in.)	100	60	85	---
25-mm (1.0-in.)	95-100	60	85	3.5
19-mm (3/4-in.)	90-100	60	85	3.5
12.5-mm (1/2-in.)	± 5	60	85	3.5
4.75-mm (No. 4)	± 5	60	85	3.0
2.00-mm (No. 10)	± 5	60	85	2.5
1.18-mm (No. 16)	± 5	60	85	2.5
0.425-mm (No. 40)	± 4	60	85	1.5
0.150-mm (No. 100)	± 3	60	85	1.0
0.075-mm (No. 200)	± 2	60	85	1.0

TABLE 28 Summary of acceptance requirements for base course, density, moisture content and thickness

Measurement	PWL for Individual Lots	PWL for Project	Standard Deviation for Project
Relative Density, Percent*	65	85	1.3
Moisture Content, Percent**	65	85	0.5
Thickness, mm***	65	80	6

*96 to 101 percent of modified AASHTO (T180).
**± 1 percent of optimum moisture content.
***1 in. = 25.4 mm.

TABLE 29 Daily quality control test plan at the hot plant

General Property	Test Method	Asphalt Content					
		Low (Lot A)		Medium (Lot B)		High (Lot C)	
		Tests/Sublots	Sublots/Lot	Tests/Sublots	Sublots/Lot	Tests/Sublots	Sublots/Lot
Asphalt Content	Nuclear	1	3	1	3	1	4
	Extraction	0	0	0	0	0	0
	Ignition	1	5	1	5	1	5
Gradation	Cold Feed	1	2	1	2	1	1
	Extraction	0	0	0	0	0	0
	Ignition	1	2	1	2	1	2
Superpave Gyratory Compaction	Bulk SP GR	1	3	1	3	1	4
	Volumetrics	1	3	1	3	1	4
	M_R	0	0	0	0	0	0
Marshall Compaction	Bulk SP GR	0	0	3	3	0	0
	Volumetrics	0	0	3	3	0	0
	Stability and Flow	0	0	3	3	0	0
Hveem Compaction	Bulk SP GR	0	0	0	0	0	0
	Volumetrics	0	0	0	0	0	0
	Stability	0	0	0	0	0	0
Theo. Max. SP GR		2	1	2	1	2	2
HMA Temperature		1	5	1	5	1	5
Field Simple Shear		1	2	1	2	1	2

TABLE 30 Daily quality control testing which was performed at the hot plant

General Property	Test Method	Asphalt Content: Low (Lot A) Tests/Sublots	Low (Lot A) Sublots/Lot	Medium (Lot B) Tests/Sublots	Medium (Lot B) Sublots/Lot	High (Lot C) Tests/Sublots	High (Lot C) Sublots/Lot
Asphalt Content	Nuclear	3	2 or 3	3	4	3	2 or 3
	Extraction	0	0	0	0	0	0
	Ignition	1	2 or 3	1	4	1	2 or 3
	Ignition 2	5	2 or 3	5	4	5	2 or 3
Gradation	Cold Feed	1	2	1	2	1	1
	Extraction	0	0	0	0	0	0
	Ignition	1	2 or 3	1	4	1	2 or 3
	Ignition 2	5	2 or 3	5	4	5	2 or 3
Superpave Gyratory Compaction	Bulk SP GR	1	2 or 3	1	4	1	2 or 3
	Volumetrics	1	2 or 3	1	4	1	2 or 3
	M_R	0	0	0	0	0	0
Marshall Compaction	Bulk SP GR	0	0	1	4	0	0
	Volumetrics	0	0	1	4	0	0
	Stability and Flow	0	0	1	4	0	0
Hveem Compaction	Bulk SP GR	0	0	0	0	0	0
	Volumetrics	0	0	0	0	0	0
	Stability	0	0	0	0	0	0
Theo. Max. SP GR		1	2 or 3	1	4	1	2 or 3
HMA Temperature		5	2 or 3	5	4	5	2 or 3
Field Simple Shear		0	0	0	0	0	0

TABLE 31 Example daily quality control test plan at laydown

General Property	Test Method	Section Number 1	2	3	4	5	6	7	8	9
HMA Temperature		5	5	5	5	5	5	5	5	5
In-Place Air Voids	Nuclear	5	5	5	5	5	5	5	5	5
	Cores	5	5	5	5	5	5	5	5	5
Water Sensitivity	T283	1	1	1	1	1	1	1	1	1

TABLE 32 Example daily quality control testing which was performed at laydown

General Property	Test Method	Section Number								
		1	2	3	4	5	6	7	8	9
HMA Temperature		5	5	5	5	5	5	5	5	5
In-Place Air Voids	Nuclear	5	5	5	5	5	5	5	5	5
	Cores	5	5	5	5	5	5	5	5	5
Water Sensitivity	T283	0	0	0	0	0	0	0	0	0

TABLE 33 Quality assurance test plan

General Property	Test Method	Tests/Sublot	Sublots/Lot	Number of Lots or Sections to be Tested	Number of Layers or Lifts
Asphalt Content	Nuclear	1	5	26	2
	Extraction	1	1	26	2
	Ignition	1	5	26	2
Gradation	Cold Feed	1	5	26	2
	Extraction	1	1	26	2
	Ignition	1	5	26	2
Superpave Gyratory Compaction	Bulk SP GR	3	5	26	2
	Volumetrics	3	5	26	2
	M_R	3	5	26	2
Marshall Compaction	Bulk SP GR	3	1	20	2
		3	5	6*	2
	Volumetrics	3	1	20	2
		3	5	6*	2
	Stability and Flow	3	1	20	2
		3	5	6*	2
Hveem Compaction	Bulk SP GR	3	1	20	2
		3	5	6*	2
	Volumetrics	3	1	20	2
		3	5	6*	2
	Stability	3	1	20	2
		3	5	6*	2
Theo. Max. SP GR		3	1	26	2
Plant HMA Temperature		1	5	26	2
Laydown HMA Temperature		1	5	26	2
In-Place Air Voids	Nuclear	2	5	26	2
	Cores	1	5	26	2
Water Sensitivity	T283	3	1	20	2
		1	5	6*	2
Field Simple Shear		1	5	26	2

*Replicate mixes only (i.e., MM1 and MM2).

TABLE 34 Revised quality assurance test plan

General Property	Test Method	Tests/Sublot	Sublots/Lot	Number of Lots or Sections to be Tested	Number of Layers or Lifts
Asphalt Content	Nuclear	1	5	26	2
	Extraction	1	1	26	2
	Ignition	1	5	26	2
Gradation	Cold Feed	1	5	26	2
	Extraction	1	1	26	2
	Ignition	1	5	26	2
Superpave Gyratory Compaction (N_{max})	Bulk SP GR	2	5	26	2
	Volumetrics	2	5	26	2
Superpave Gyratory Compaction (N_{design})	Bulk SP GR	1	5	26	2
	Volumetrics	1	5	26	2
	M_R	1	5	26	2
Marshall Compaction	Bulk SP GR	3	1	26	2
	Volumetrics	3	1	26	2
	Stability and Flow	3	1	26	2
Hveem Compaction	Bulk SP GR	3	1	26	2
	Volumetrics	3	1	26	2
	Stability	3	1	26	2
Theo. Max. SP GR		1	5	26	2
Plant HMA Temperature		1	5	26	2
Laydown HMA Temperature		1	5	26	2
In-PlaceAir Voids	Nuclear	2	5	26	2
	Cores	1	5	26	2
Field Simple Shear		1	5	26	2

TABLE 35 Test section top lift construction aggregate sampling requirements

Material Type	Number of Samples Required	Individual Sample Size	Use
Cold feeds	5/section	3,000 g	QC
	5/section	5-gallon bucket	QA
	5/section	5-gallon bucket	MRL
Stockpile	4/section (1¾-in., 1½-in., 1⅜-in., 1 RD)	5-gallon bucket	UNR
	10/paving day (3¾-in., 1½-in., 3⅜-in., 3 RD)	55-gallon drums	OSU
	47/paving day (14¾-in., 5½-in., 14⅜-in., 14 RD)	55-gallon drums	UCB
	34/lift/section (10¾-in., 4½-in., 10⅝-in., 10 RD)	5-gallon bucket	MRL
Hydrated Lime	3/paving day	5-gallon bucket	MRL
Baghouse Fines	3/paving day	5-gallon bucket	MRL

1 lb = 450 g

1 gal = 3.78 L

1 in. = 25.4 mm

TABLE 36 Construction aggregate test requirements

Specification	Test Method		Type	Number of Tests
ASTM	Gradation	C117, C136	Cold feeds	2/section/lift

TABLE 37 Test section top lift construction loose HMA and core sampling requirements

Material Type	Number of Samples Required	Individual Sample Size	Use
Cores	5/section	6-in.	QC - V_{air}, Ignition P_{asp} and Gradation
Loose HMA	1/section	5-gallon bucket	QC - Gyratory Volumetrics
	2/section	5-gallon buckets	QA - P_{asp}
	5/section	5-gallon buckets	QA - Gyratory Volumetrics
	1/section	1 box	QA - Marshall
	1/section	1 box	QA - Hveem
	5/section	1 box	QA - Theoretical Maximum Specific Gravity
	1/section	1 box	QA - Moisture Sensitivity
	15/section	5-gallon buckets	OSU/UCB
	20/section	5-gallon buckets	MRL

1 in. = 25.4 mm
1 gal = 3.78 L

TABLE 38 Construction HMA test requirements

Specification	Test Method		Type	Number of Tests
ASTM	Mix BSG	T166	Cores	5/section/lift
	% AC by Ignition	C	Cores	5/section/lift
	Gradation	C117, C136	Cores	5/section/lift
	Nuclear Density	D2950	Mat	5/section/lift
	Mix Temperature		Truck	5/section/lift
	Mat Temperature		Mat	5/section/lift
	Theoretical Maximum Specific Gravity	T209	Loose HMA	1/section/lift
Superpave	Gyratory Compaction	TP4	Loose HMA	3/section/lift
	Mix BSG	T166	Loose HMA	3/section/lift

TABLE 39 Postconstruction aggregate test requirements (mix design verification and QA)

Specification	Test Method		Individual Stockpile Samples	Combined Aggregate*
ASTM and AASHTO	Apparent Specific Gravity	C127, C128	3	3
	Bulk Specific Gravity	C127, C128	3	3
	Absorption Capacity	C127, C128	3	3
Superpave	C.A. Angularity	D5821	3	3
	F.A. Angularity	C1252	3	3
	Flat or Elongated	D4791	3	3
	Sand Equivalent	T176	3	3
	L.A. Abrasion	T96	3	3
	Sodium Soundness	T104	3	3
	Deleterious Materials	T112	3	3

*Only the "target" gradation will be tested.

TABLE 40 Postconstruction mix design verification testing requirements

Specification	Test Method		Mixture		
			Target	Top Lift	Bottom Lift
Superpave	Anti-Strip Treatment	NDOT T320	17	17	17
	Short-Term Oven Aging	TP2	23	17	17
	Gyratory Compaction	TP4	21	15	15
	Theoretical Maximum Specific Gravity	T209	2	2	2
	Bulk Specific Gravity	T166	21	15	15
	Moisture Sensitivity	T283	1*	0	0

*Indicates one set of six specimens, three unconditioned and three conditioned.

TABLE 41 Postconstruction HMA QA test requirements

General Property	Test Method	Tests/Sublot	Sublots/Lot	Number of Lots or Sections to be Tested	Number of Lifts
Asphalt Content	Nuclear	1	5	8	2
	Extraction	1	1	8	2
Gradation	Cold feed	1	5	8	2
	Extraction	1	1	8	2
Superpave Gyratory Compaction (N_{max})	Bulk SP GR	2	5	8	2
	Volumetrics	2	5	8	2
Superpave Gyratory Compaction (N_{design})	Bulk SP GR	1	5	8	2
	Volumetrics	1	5	8	2
	M_R	1	5	3*	2
Marshall Compaction	Bulk SP GR	3	1	8	2
	Volumetrics	3	1	8	2
	Stability and Flow	3	1	8	2
Hveem Compaction	Bulk SP GR	3	1	8	2
	Volumetrics	3	1	8	2
	Stability	3	1	8	2
Theoretical Max. Specific Gravity		1	5	8	2

*Low, medium, and high asphalt content with medium air voids only (sections 35, 37, and 38).

TABLE 42 Section 35, bottom lift test lane sampling plan

Date: 5/30/97 Section Number: **35**
Paving Day: 33 Lane: Test

Lift: **Bottom**

Test to be Performed	Number of Samples	Sample Size (grams)	Samples to be Tested Immediately	Sample Location	Upon Completion Deliver To
Asphalt Cement (OSU)	1	35 buckets		Tank	
Asphalt Cement (UCB)	1	8 buckets		Tank	
Asphalt Cement (UNR)	1	50 buckets		Tank	
Asphalt Cement (MRL)	1	50 buckets		Tank	
BSG (cores)	5	150mm	5	Mat	
%AC by ignition	*	*	5	Mat	
Coldfeed Agg Tests (misc)	1	5 buckets	0	Hot Plant	
Coldfeed Agg (MRL)	1	5 buckets		Hot Plant	
Coldfeed Gradation	5	2,500	5	Hot Plant	
Ignition Gradation (cores)	*	*	5	*	
Gyratory BSG (QC)	3	5,500	3	Mat	
Gyratory AV & Volumetrics	*	*	3	*	
Theo Max SG (Rice) QC	1	2,500	1	Mat	
Gyratory BSG (QA)	5	1 box	0	Truck	
Gyratory AV & Volumetrics	*		0	*	
Theo Max SG (Rice) QA	5	2,000	0	Truck	
Marshall Properties	1	1 box	0	Truck	
Hveem Properties	1	1 box	0	Truck	
HMA Temperature (Truck)	5		5	Truck	
HMA Temperature (Mat)	5		5	Mat	
Water Sensitivity (QA)	1	1 box	0	Truck	
Bulk HMA (OSU)	1	5 buckets	0	Truck	
Bulk HMA (UCB)	1	5 buckets	0	Truck	
Bulk HMA (MRL)	1	10 buckets	0	Truck	
Baghouse Fines (MRL)	1	3 buckets	0	Return	
Lime (MRL)	1	3 buckets	0	Silo	
Stockpile Samples (OSU)	1	7 drums	0	Hot Plant	2-3/4", 1-1/2", 2-3/8", 2-RD
Stockpile Samples (UCB)	1	27 drums	0	Hot Plant	8-3/4", 3-1/2", 8-3/8", 8-RD
Stockpile Samples (UNR)	1	4 buckets	0	Hot Plant	1-3/4", 1-1/2", 1-3/8", 1-RD
Stockpile Samples (MRL)	1	17 buckets	0	Hot Plant	5-3/4", 2-1/2", 5-3/8", 5-RD

*Samples obtained from previous tests.

1 lb = 454 g
1 in. = 25.4 mm

TABLE 43 Asphalt binder grade selection reliability

Weather Station	Reliability for Indicated PG Grade							Grade for Indicated Reliability	
	58-16	58-22	58-28	58-34	64-22	64-28	64-34	Minimum 50%	Minimum 98%
Fallon Exp Stn		85-72*	85-96			98-96	98-98	58-22	64-34
Fernley		68-77			98-77	98-98		58-22	64-28
Gerlach		77-70	77-94			98-94	98-98	58-22	64-34
Lahontan Dam	68-58	68-94			98-94	98-98		58-16	64-28
Reno WSFO AP		95-81	95-98			98-98		58-22	64-28
Sand Pass		72-84			98-84	98-98		58-22	64-28
Virginia City	98-72	98-98						58-16	58-22
Yerington R5		98-60	98-96	98-98				58-22	58-34
Yerington		93-65	93-94			98-94	98-98	58-22	64-34

*High and low temperature reliability.

TABLE 44 WesTrack asphalt binder properties as determined by Superpave binder tests (original construction)*

Aging	Test Method	Temperature of Test, °C	Test Parameter	Value	Specification (AASHTO MPI)*
Original	DSR (AASHTO TP5)	64	G*/sinδ	1.141 kPa	Min. 1.0 kPa
	Flash point (AASHTO T48)			276°C	Min. 230°C
	Viscosity (ASTM D 4402)	135	Viscosity	0.37 Pa.s	Max. 3.0 Pa.s
RTFOT	DSR (AASHTO TP5)	64	G*/sinδ	2.637 kPa	Min. 2.2 kPa
	Mass loss (AASHTO T240)		Mass loss	0.253 percent	Max. 1.0%
RTFOT Plus PAV	DSR (AASHTO TP5)	25	G*sinδ	4270 kPa	Max. 5000 kPa
	BBR (AASHTO TP1)	-12	Stiffness	216.8 MPa	Max. 300 MPa
		-12	"m" Value	0.315	Min. 0.30

*Classified as PG 64-22.

1 psi = 6.9 kPa
°F = 1.8°C + 32
1 Pa.s = 2.089 x 10^{-2} lb-sec/ft^2

TABLE 45 WesTrack asphalt binder properties as determined by viscosity and penetration tests (original construction)*

Aging	Test Method	Temperature of Test, °C	Value	Specification AASHTO M 226 ASTM D 3381*
Original	Viscosity	60	1897 poise	1600-2400 poise
	(D 2171) (D 2170)	135	362 poise	Min. 300 poise
	Penetration (D 5)	25	55.5 dmm	Min. 60 dmm
		4	20.2 dmm	
	Flash point		275°C	Min. 232°C
	Solubility		99.9 percent	Min. 99.0 percent
RTFOT	Viscosity	60	4641 poises	Max. 10,000 poises
	Ductility	25	>50 cm	Min. 50 cm

*AC-20 asphalt cement, ASTM table 2.

°F = 1.8°C + 32
1 poise = 478.8 lbf-sec/ft^2
1 in. = 254 dmm

TABLE 46 WesTrack asphalt binder properties as determined by Superpave binder tests (replacement construction)*

Aging	Test Method	Temperature of Test, °C	Test Parameter	Value	Specification (AASHTO MPI)*
Original	DSR (AASHTO TPS)	64	G*/sinδ	1.493 kPa	Min. 1.0 kPa
	Flash Point (AASHTO T48)			288°C	Min. 230°C
	Viscosity (ASTM D 4402)	135	Viscosity	0.358 Pa.s	Max. 3.0 Pa.s
RTFOT	DSR (AASHTO TP5)	6.4	*/sinδ	3.603 kPa	Min. 2.0 kPa
	Mass Loss (AASHTO T240)		Mass Loss	0.261 percent	Max. 1.0%
RTFOT plus PAV	DSR (AASHTO TP5)	2.5	G*/sinδ	3946 kPa	Max. 5000 kPa
	BBR (AASHTO TP1)	-12	Stiffness	198 MPa	Max. 300 MPa
		-12	"M" Value	0.316	Min. 0.30

*Classified as PG 64-22.

1 psi = 6.9 kPa
°F = 1.8°C + 32
1 Pa.s = 2.089 x 10^{-2} lb-sec/ft^2

TABLE 47 WesTrack asphalt binder properties as determined by viscosity and penetration tests (replacement sections)*

Aging	Test Method	Temperature of Test, °C	Value	Specification AASHTO M 226 ASTM D 3381
Original	Viscosity, P (AASHTO T202)	60	2521	16-2400
	Viscosity, cSt (AASHTO T201)	135	440	Min. 300
	Penetration, dmm (AASHTO T49)	25	65	Min. 60
	Ductility, cm (AASHTO T51)	4	9	Min. 5 (UDOT)
	Toughness, in-lb		57	
	Terracity, in-lb		0	
RTFOT	Viscosity, P (AASHTO T202)	60	6250	Max. 10,000
	Viscosity, cSt (AASHTO T 201)	135	639	
	Penetration, dmm (AASHTO T49)	25	19	
	Ductility, cm (AASHTO T 51)	4	0	
	Mass loss, % (AASHTO T240)		0.28	Max. 1.0

*Tests performed by Utah Department of Transportation and reported December 1, 1997, on sample obtained on June 9, 1997.

1 P = 478.8 lbf-sec/ft^2
1 cSt = 9.29 ft^2/sec
1 in. = 254 dmm
1 in.-lb = 11.5 mm-kg
°F = 1.8°C + 32

TABLE 48 Comparison of asphalt binders used for original and replacement sections at WesTrack

Sample Identification	DSR-Original @ 64°C			DSR-RTFOT @ 64°C			DSR-PAV @ 25°C			BBR @-12°C		PG Grade
	G*, kPa	δ	G*/sin δ, kPa	G*, kPa	δ	G*/sin δ, kPa	G*, MPa	δ	G*/sin δ, MPa	S(t), MPa	m	
Original sections*	1.140	87.1	1.141	2.625	84.3	2.637	5.782	47.7	4.270	217	0.315	64-22
Replacements ections*	1.491	87.0	1.493	3.583	83.9	3.603	5.696	43.9	3.946	198	0.316	64-22

*Average values.

°F = 1.8°C + 32
1 psi = 6.9 kPa

TABLE 49 Comparison of asphalt binders used for original and replacement sections at WesTrack

Sample Identification	Flash Point, °C	Viscosity @ 135°C, Pa.s	DSR Original Temp @ 1.0 kPa	DSR RTFOT Temp @ 2.2 kPa	Mass Loss, Percent	DSR PAV Temp @ 5.0 MPa	BBR		PG Grade
							Temp @ 300 MPa	Temp @ m=0.30	
Original sections	276*	0.31	65.3	65.5	0.25	24.0	-14.3	-13.3	64-22
Replacement sections	288*	0.36	67.3	67.9	0.26	22.8	-16.3	-15.1	64-22

*Average values.

°F = 1.8°C + 32
1 psi = 6.9 kPa
1 ft = 0.305 m
1 Pa•s = 2.089 x 10^{-2} lb-sec/ft^2

TABLE 50 Comparison of original and replacement sections at WesTrack DSR rheologies properties over a range in temperatures

Sample Identification	DSR-Original (G*sin δ) @ Temp °C, kPa				DSR-RTFOT (G*/sin δ) @ Temp °C, kPa				DSR-PAV (G*sin δ) @ Temp °C, kPa			
	52	58	64	70	52	58	64	70	16	19	22	25
Original sections 10-1-95 9:50 a.m.	5.62	2.46	1.14	0.57	13.41	5.85	2.62	1.25	12.11	8.94	6.40	4.35
Replacement sections 6-28-97 REP 1	7.69	3.29	1.45	0.69			3.53	1.64		6.79	5.25	3.73

°F = 1.8°C + 32
1 psi = 6.9 kPa

TABLE 51 Comparison of original and replacement sections at WesTrack—BBR rheologic low temperature properties

Sample Identification	**BBR**			
	Stiffness, MPa @ Temp °C		**m Value @ Temp °C**	
	-12	**-18**	**-12**	**-18**
Original sections 10-1-95 9:50 a.m.	229	410	0.31	0.25
Replacement sections 6-28-97 REP 1	201	322	0.32	0.28

°F = 1.8°C + 32
1 ksi = 6.9 MPa

TABLE 52 Physico-chemical properties of hydrated lime used during construction of original sections

Property	Sample Date and Time							Specification Limits*
	9/14/95 14:17	9/18/95 12:23	9/20/95 14:08	9/22/95 14:05	9/27/95 7:55	9/28/95	10/3/95 16:02	
Ca and Mg Hydroxides (%)	93.2	93.1	93.9	93.1	93.4	91.3	93.4	Min. 90.0
Carbon Dioxide (%)	1.1	1.0	0.7	1.2	1.2	1.6	1.1	Max. 5.0
Unhydrated Ca and Mg Oxides (%)	0.0	0.2	0.6	0.1	0.0	0.4	0.1	Max. 8.0
Free Moisture (%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	Max. 3.0
Retained %, No. 30 sieve**	0.4	3.9	0.0	0.7	0.5	0.7	0.7	Max. 3.0
Retained %, No. 200 sieve***	13	5	4	9	7	8	10	Max. 30

*Specifications per ASTM C 1097-90, standard specification for hydrated lime for use in asphaltic-concrete mixtures.
**0.300-mm sieve
***0.075-mm sieve

Note: Tests performed by Chemical Lime Company's laboratory in Henderson, Nevada.

TABLE 53 Geologic description of aggregate—original construction

Aggregate	Geologic Description
Dayton 3/4-in. to rock dust	Unweathered, to slightly weathered, fine-grained andesite (volcanic), with a few (10%) pieces of granite (quartz monzonite) and a few (10%) pieces of rhyolite (volcanic).
Dayton sand	Weathered fragments of andesite, rhyolite, and granite (in sizes up to .25 in) mixed with a smaller sized fraction consisting of decomposed granite (mica, feldspar and quartz); sizeable component of fines.
Wadsworth sand	Decomposed granite composed of weathered fragments of mica and feldspar, with rounded quartz grains; clean, few fines.

1 in. = 25.4 mm

TABLE 54 Stockpile gradations 1994 production (original construction)*

Sieve Size (mm)	Sieve Size US	Dayton 3/4-in.	Dayton 1/2-in.	Dayton 3/8-in.	Dayton Rock Dust	Dayton Sand	Wadsworth Sand
25.0	1-in.	100	100	100	100	100	100
19.0	3/4-in.	99.8	100	100	100	100	100
12.5	1/2-in.	63.5	99.9	100	100	100	100
9.5	3/8-in.	33.7	82.6	97.5	100	100	100
4.75	No. 4	10.8	19.5	27.5	99.1	96.5	99.7
2.36	No. 8	5.0	4.3	5.1	76.6	78.9	99.0
1.18	No. 16	4.3	3.2	4.2	54.1	62.1	96.0
0.60	No. 30	4.0	2.9	3.8	39.2	45.0	79.9
0.30	No. 50	3.7	2.6	3.5	28.7	28.9	40.1
0.15	No. 100	3.4	2.4	3.1	20.3	17.9	11.0
0.075	No. 200	3.0	2.0	2.5	13.9	11.5	3.3

Aggregate Source columns: Dayton 3/4-in., Dayton 1/2-in., Dayton 3/8-in., Dayton Rock Dust, Dayton Sand, Wadsworth Sand.

*Represents the average of five samples—see reference 2.23 for individual sieve analysis and statistics.

1 in. = 25.4 mm

TABLE 55 Comparison of 1994 and 1995 Dayton aggregate stockpile gradations (original construction)

Sieve Size (mm)	Sieve Size (US)	3/4-in. 1994 Cumulative % Passing (5-Sample Average)	3/4-in. 1995 Cumulative % Passing (16- Sample Average)	1/2-in. 1994 Cumulative % Passing (5-Sample Average)	1/2-in. 1995 Cumulative % Passing (33-Sample Average)	3/8 in. Before Z-Screen 1994 Cumulative % Passing (5-Sample Average)	3/8 in. Before Z-Screen 1995 Cumulative % Passing (30-Sample Average)	3/8-in. After Z-Screen 1994 Cumulative % Passing (5-Sample Average)	3/8-in. After Z-Screen 1995 Cumulative % Passing (8-Sample Average)	Rock Dust 1994 Cumulative % Passing (5-Sample Average)	Rock Dust 1995 Cumulative % Passing (30-Sample Average)
19	3/4"	99.8	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	1/2"	63.5	73.2	99.9	98.8	100.0	100.0	100.0	100.0	100.0	100.0
9.5	3/8"	33.7	49.1	82.6	43.8	97.7	100.0	97.7	100.0	100.0	100.0
4.75	#4	10.8	10.8	19.5	1.7	29.8	29.0	29.8	26.0	99.1	99.8
2.36	#8	5.0	2.6	4.3	1.6	6.4	3.4	6.4	3.5	76.6	76.3
1.18	#16	4.3	2.1	3.2	1.5	5.0	2.5	5.0	2.6	54.1	53.3
0.60	#30	4.0	1.9	2.9	1.4	4.4	2.4	4.4	2.5	39.2	38.3
0.30	#50	3.7	1.8	2.6	1.4	3.9	2.1	3.9	2.4	28.7	26.3
0.15	#100	3.4	1.6	2.4	1.4	3.4	2.0	3.4	2.1	20.3	17.6
0.075	#200	3.0	1.3	2.0	1.1	2.8	1.8	2.8	1.9	13.9	11.3

1 in. = 25.4 mm

TABLE 56 Aggregate stockpile percentages for mixture design (original construction)*

Stockpile	Mixture Designation		
	Fine	Fine Plus	Coarse
Dayton 3/4-in.	31.0	31.0	47.0
Dayton 1/2-in.	19.0	19.0	19.0
Dayton 3/8-in.	10.0	10.0	0.0
Dayton Rock Dust	13.5	13.5	32.5
Dayton Sand	0.0	0.0	0.0
Wadsworth Sand	25.0	25.0	0.0
Lime	1.5	1.5	1.5

*All stockpiles 1994 production.

1 in. = 25.4 mm

TABLE 57 Aggregate stockpile percentages during original construction

Stockpile	Mixture Designation			
	Fine*	Fine-Plus*	Coarse	
			Bottom	Top
Dayton 3/4-in. 1994 Production	25.0	25.0	39.0	39.0
Dayton 1/2-in. 1994 Production	25.0	25.0	27.0	
Dayton 1/2-in. 1995 Production				12.2
Dayton 3/8-in. 1994 Production	10.0	10.0		14.8
Dayton Rock Dust 1994 Production	13.5	13.5	32.5	32.5
Dayton Sand 1994 Production				
Wadsworth Sand 1994 Production	25.0	25.0		
Lime	1.5	1.5	1.5	1.5

*Top and bottom lifts.

1 in. = 25.4 mm

TABLE 58 Preconstruction aggregate properties—1994 production (original construction)

Stockpile/ Mixture Designation	Coarse Aggregate Angularity		Fine Aggregate Angularity	Flat or Elongated, Percent	Sand Equivalent, Percent	LA Abrasion	Soundness	Deleterious Materials
	1 Face	2 Face						
Dayton 3/4-in.	98.9	95.4		0.2		18.9		0.46
Dayton 1/2-in.	99.7	98.9		2.0		20.2		0.42
Dayton 3/8-in.	98.2	96.4				21.3		0.30
Dayton Rock Dust			47.6		49.0			0.46
Dayton Sand			44.5		31.0			
Wadsworth Sand			44.0		74.0			0.15
Fine	98.1	96.5	44.9*	0.6	65.0	20.3		0.21
Fine Plus	98.1	96.5	44.9*	0.6	65.0	20.3		0.21
Coarse	98.8	96.7	45.0	1.3	70.0	20.8		0.34

*Based on FA Gsb of 2.595 from field blend of July 8, 1995.

1 in. = 25.4 mm

TABLE 59 Preconstruction aggregate properties—1995 production (original construction)

	Coarse Aggregate Angularity		Fine Aggregate Angularity	Flat or Elongated Percent	Sand Equivalent, Percent
	1 Face	2 Face			
Dayton 3/4-in.	99.7	99.0		0.9	
Dayton 1/2-in.	99.7	99.2		0.0	
Dayton 3/8-in.	99.7	99.6			
Dayton Rock Dust			47.9		66.0
Dayton Sand			44.5		

1 in. = 25.4 mm

TABLE 60 Properties of aggregate samples from cold feeds (original construction)

Mix Designation	Lift	Coarse Aggregate Angularity	Fine Aggregate Angularity	Flat or Elongated		Sand Equivalent	LA Abrasion	Soundness		Deleterious Materials, No. 4 Sieve**	Plastic Index
				3/4 x 1/2*	1/2 x 3/8*			CA	FA		
Fine	Top	93.6	44.9	0.0	0.05	64.3	21.1	6.8	7.2	0.25	NP
	Bottom	97.1	42.1	0.23	0.33	63.7	22.5	4.0	6.8	0.16	NP
Fine Plus	Top	97.7	44.2	0.37	0.20	66.3	20.7	1.9	5.8	0.99	NP
	Bottom	94.5	44.8	0.0	0.03	65.7	20.9	6.4	8.3	0.42	NP
Coarse	Top	90.5	47.0	0.0	0.03	45.7	20.1	4.7	9.0	0.73	NP
	Bottom	94.9	46.1	0.0	0.11	58.3	21.9	6.0	9.0	0.20	NP

*1 in. = 25.4 mm
**4.75-mm sieve

TABLE 61 Dry bulk specific gravity used for volumetric calculations (original construction)

Mixture Designation	Lift	Dry Bulk Specific Gravity for Combined Aggregate
Fine	Top	2.529
	Bottom	2.529
Fine Plus	Top	2.529
	Bottom	2.529
Coarse	Top	2.504
	Bottom	2.504

TABLE 62 Geologic description of aggregate—replacement sections

Aggregate	Geologic Description
Lockwood 3/4-in. to Rock Dust	Unweathered, to slightly weathered, fine-grained andesite (volcanic).
Patrick Sand	Decomposed granite composed of weathered fragments of mica and feldspar, with rounded quartz grains; clean, few fines.

1 in. = 25.4 mm

TABLE 63 Stockpile gradations—Lockwood aggregate (replacement sections)

Sieve Size	3/4-in. Stockpile	½-in. Stockpile	3/8-in. Stockpile	Rock Dust Stockpile
25-mm (1-in.)	100.0	100.0	100.0	100.0
19-mm (3/4-in.)	96.8	100.0	100.0	100.0
12.5-mm (1/2-in.)	29.4	99.5	100.0	100.0
9.5-mm (3/8-in.)	7.4	49.7	99.5	100.0
4.75-mm (No. 4)	3.0	1.8	31.3	98.5
2.36-mm (No. 8)	2.0	1.3	4.7	73.0
1.18-mm (No. 16)	1.6	1.1	2.4	47.9
0.600-mm (No. 30)	1.5	1.0	2.0	33.6
0.300-mm (No. 50)	1.3	0.9	1.8	24.8
0.150-mm (No. 100)	1.2	0.8	1.6	19.4
0.075-mm (No. 200)	1.0	0.7	1.5	15.6

1 in. = 25.4 mm

TABLE 64 Stockpile blends for mixture design and plant production—replacement sections—Lockwood aggregate

Stockpile Designation	Mix Design	Plant Production
3/4-in.	24.0	24.0
1/2-in.	15.5	15.5
3/8-in.	27.5	27.5
Rock dust	31.5	31.5
Hydrated lime	1.5	1.5

1 in. = 25.4 mm

TABLE 65 Chronological summary of HMA blend coarse and fine aggregate angularity, flat and elongated particles, and sand equivalent determinations (replacement sections)

Property	Sampling Location	Sampling Date	Blend	Tech	Fraction	Average Measured Value	STD	COV	Range	Number of Observations
Coarse Aggregate Angularity	Lockwood	04-14-97	TBE	TO	+#4	99.9 / 99.9	0.10 / 0.10	0.1 / 0.1	0.2 / 0.2	3
	Patrick*	05-23-97	P7	WC	+#4	85.7 / 84.5	7.1 / 7/6	8.3 / 9.0	13.3 / 14.6	3
	Patrick	06-07-97	F	WC	+#4	81.3 / 78.8	3.6 / 4.5	4.4 / 5.7	11.9 / 10.3	3
Fine Aggregate Angularity	Lockwood	04-14-97	TBE	TO	-#8	46.1	0.2	0.4	0.4	3
	Patrick	05-23-97	P7	TO	-#8	46.3	0.2	0.5	0.5	3
	Patrick	06-07-97	F	WC	-#8	46.2	0.2	0.4	0.5	3
Flat and Elongated Particles	Lockwood	04-14-97	TBE	TO	+#4	0.0	0.0	0.0	0.0	3
	Patrick	05-23-97	P7	WC	+#4	0.0	0.0	0.0	0.0	3
	Patrick	06-07-97	F	TO	+#4	0.1	0.2	141.4	0.43	3
Sand Equivalent	Lockwood	04-14-97	TBE	TO	-#4	67.0	0.47	0.7	1.0	3
	Patrick	05-23-97	P7	WC	-#4	73.0	0.56	0.8	1.0	3
	Patrick	06-07-97	F	TO	-#4	73.0	1.93	2.6	5.0	3

*Patrick—location of hot-mix plant.

TABLE 66 Combined specific gravities of Lockwood aggregate (replacement sections)

Property	Sample Location	Sample Date	Blend	Combined Specific Gravity
Bulk Specific Gravity	Patrick	06-07-97	F	2.553*
Apparent Specific Gravity	Patrick	06-07-97	F	2.763
Water Absorption	Patrick	06-07-97	F	2.41

*Mixture design "F" utilized a combined bulk specific gravity of 2.599.

TABLE 67 Gradations for mixture design

Sieve Size		Mixture Designation			
		Original Construction			Replacement Sections
mm	U.S.	Fine	Fine Plus	Coarse	Coarse
25.0	1-in.	100.0	100.0	100.0	100.0
19.0	3/4-in.	99.9	99.9	99.9	99.2
12.5	1/2-in.	88.5	88.3	82.4	82.8
9.5	3/8-in.	75.4	76.1	64.6	69.5
4.75	No. 4	48.9	50.4	41.2	41.4
2.36	No. 8	38.4	40.2	27.8	25.6
1.18	No. 16	33.9	35.6	19.7	16.8
0.60	No. 30	27.6	29.7	14.6	12.1
0.30	No. 50	15.7	18.2	10.8	9.1
0.15	No. 100	6.8	9.6	7.7	7.2
0.075	No. 200	3.5	6.4	5.0	5.8

1 in. = 25.4 mm

TABLE 68 Stockpile blends for mixture design

Stockpile Designations	Mixture Designation			
	Original Construction			Replacement Sections
	Fine	Fine Plus	Coarse	Coarse
Dayton (1994)* 3/4-in.	31.0	31.0	47.0	
Dayton (1994) 1/2-in.	19.0	19.0	19.0	
Dayton (1995) 1/2-in.				
Dayton (1994) 3/8-in.	10.0	10.0	0.0	
Dayton. (1994) Rock Dust	13.5	13.5	32.5	
Wadsworth Sand (1994)	25.0	25.0	25.0	
Lockwood (1997) 3/4-in.				24.0
Lockwood (1997) 1/2-in.				15.5
Lockwood (1997) 3/8-in.				27.5
Lockwood (1997) Rock Dust				31.5
Hydrated Lime	1.5	1.5	1.5	1.5

*Indicates production year.

1 in. = 25.4 mm

TABLE 69 Stockpile blends during construction

Stockpile Designations	Mixture Designation				
	Original Construction				Replacement Sections
	Fine	Fine Plus	Coarse		Coarse
			Top**	Bottom	
Dayton (1994)* 3/4-in.	25.0	25.0	39.0	39.0	
Dayton (1994) 1/2-in.	25.0	25.0		27.0	
Dayton 1995) ½-in.			12.2		
Dayton (1994) 3/8-in.	10.0	10.0	14.8		
Dayton (1994) Rock Dust	13.5	13.5	32.5	32.5	
Wadsworth Sand (1994)	25.0	25.0			
Lockwood (1997) 3/4-in.					24.0
Lockwood (1997) 1/2-in.					15.5
Lockwood (1997) 3/8-in.					27.5
Lockwood (1997) Rock Dust					31.5
Hydrated Lime	1.5	1.5	1.5		1.5

*Indicates production year.
**Indicates top and bottom lifts.

1 in. = 25.4 mm

TABLE 70 Mixture design weight—volumes at design asphalt binder content

Property	Mixture Designation			
	Original Construction			Replacement Sections
	Fine	Fine Plus	Coarse	Coarse
Asphalt content, percent by total weight of mixture	5.4	6.0*	5.7	5.65
Air voids, percent	4.0	4.0	4.0	4.0
Voids in mineral aggregate, percent	13.7	13.6	14.2	15.0
Voids filled with asphalt, percent	71.0	75.0	71.0	73.0
% Gmm @ $N_{initial}$	89.8		86.3	86.6
% Gmm @ N_{max}	97.0		97.3	97.5
Fine/asphalt ratio	0.82	1.5**	1.1	1.78
Bulk specific gravity of combined aggregate	2.529	2.529	2.504	2.599

*5.4 used as target asphalt binder content in field sections.
**at 5.4 percent asphalt binder content.

TABLE 71 Superpave mix specifications*

Mixture Property	Criteria
% Air voids	4%
% Voids in mineral aggregate	13% minimum
% Voids filled with asphalt	65% - 75%
Dust-to-binder ratio	0.6 - 1.2
% Gmm @ $N_{initial}$ = 8	less than 89%
% Gmm @ N_{design} = 96	96%
% Gmm @ N_{max} = 152	less than 98%

*3 to 10 million ESALs.

TABLE 72 Hveem mixture design properties for original construction*

Mixture ID	P_{asp}*	Stability	% Air Voids	% VMA	% VFA
Fine	5.4	43	4.1	14.8	72
Fine plus	5.4	41	4.0	13.8	70
Coarse	5.7	38	4.2	14.3	68

*Typical min. stability values of 35 and 37.

TABLE 73 Gyratory volumetrics summary for mixture "F" (replacement sections)

Project:	WesTrack Rehab
Mixture ID:	F
Other:	Lockwood at Patrick (June 7, 1997 sampling)
Date:	June 12, 1997
Technician:	TO

P_{asp}	% Gmmd	V_{air}	% VMA	% VFA	% Gmmi	% Gmmm	F/A
4.0							
4.5	93.8	6.2	14.6	57.6	84.4	95.2	1.56
5.0	95.1	4.9	14.6	66.8	85.5	96.6	1.35
5.5	95.8	4.2	14.9	72.0	86.5	97.2	1.23
6.0	96.3	3.7	15.4	75.9	87.6	97.6	1.12
6.5	97.0	3.0	15.9	81.5	87.3	98.3	1.01
7.0							

Properties at optimum asphalt content

P_{asp}	% Gmmd	V_{air}	% VMA	% VFA	% Gmmi	% Gmmm	F/A
5.65	96.0	4.0	15	73	86.6	97.5	1.18

TABLE 74 Hveem stability test results—TBE mixture—Idaho asphalt binder (replacement sections)*

Asphalt Binder Content, Percent by Total Weight	Hveem Stability
4.75	35.3
5.25	32.2
5.75	30.5
6.25	29.4

*Mix design TBE.