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Accelerated Performance-Related Tests for Asphalt-Aggregate Mixes and Their Use in Mix Design and Analysis Systems

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University of California at Berkeley: Dr. A. A. Tayebali (fatigue), Dr. J. B. Sousa (permanent deformation), Dr. J. Harvey (materials processing and specimen preparation), and Dr. J. A. Deacon (fatigue and permanent deformation). Special note should be made of the significant contributions of Dr. Deacon to the development of the mix design and analysis systems for fatigue and permanent deformation.

Oregon State University: Dr. T. Vinson (low-temperature cracking); Dr. C. Bell (aging); and Dr. R. Terrel and Mr. T. Scholz (water sensitivity).

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Abstract

One of the objectives of the SHRP Project A-003A has been the development of a series of accelerated performance-related tests to define asphalt-aggregate interactions which affect pavement performance. Tests included those for fatigue, permanent deformation, thermal cracking, aging, and water sensitivity.

This report provides a brief summary of the processes which lead to the selection of each of the tests. Inherent in this test selection process was the emphasis on the ability of the tests to measure fundamental material properties which, when incorporated into prediction models, will depend less on empirical correlations than has been traditionally the case. Also included are the results of validation studies for each of the tests and frameworks for the use of the tests in mix design and analysis. Several levels of design are provided for each distress, some of which incorporate reliability concepts.

Executive Summary

One of the objectives of the SHRP Project A-003A has been the development of accelerated performance-related tests to define asphalt-aggregate interactions which significantly affect performance. The resulting tests—considered suitable for standardization and which can be used by the highway industry (both government and private agencies)—include those to measure and quantify fundamental material properties; they can be used to predict fatigue cracking, low-temperature cracking, and permanent deformation (rutting). In addition, procedures have been developed to consider the effects of aging and exposure to water and water vapor in order for the laboratory-determined properties to be representative of in-situ characteristics and thus be capable of accelerating these effects to estimate long-term mix response.

This report provides a brief summary of the processes which lead to the selection of the test methodologies for fatigue, permanent deformation, thermal cracking, aging, and water sensitivity; a summary of validation efforts to demonstrate the efficacy of the tests to predict performance; and the use of these tests for purposes of mix design and analysis.

In Chapter 2, the framework for a mix design and analysis system is presented. The system, recognizes not only mix properties but also in-situ traffic, climatic, and structural conditions. Several levels of design are provided for each distress, some of which incorporate reliability concepts. For routine applications, the testing and analysis is minimal, whereas for more costly projects and the use of unconventional mixes, the testing is more extensive.

The analysis systems for fatigue and permanent deformation are similar. They assume that a trial mix has been identified, that traffic and environmental conditions have been determined, and that the pavement cross-section has been designed. The analysis systems seek to judge, with predetermined reliability, whether the trial mix would perform satisfactorily in service. If it would not, the designer can opt for redesigning the mix, strengthening the pavement section, or repeating the analysis using more refined measurement and/or estimates. The steps in the analysis systems are as follows:

1. Determine design requirements for performance (extent of cracking and/or amount of rutting) and reliability (probability of avoiding the acceptance of a deficient mix).

2. Determine expected distribution of in-situ temperatures.

3. Estimate design traffic demand (ESALs).

4. Design structural section.

5. Select trial mix.

6. Measure stiffness of trial mix.

7. Determine design stresses and/or strains under "standard" axle load.

8. Measure the resistance of the trial mix to fatigue and/or permanent deformation.

9. Compare traffic demand with resistance.

10. If inadequate, alter trial mix and/or structural section and perform another iteration of the analysis.

For thermal cracking, information on low temperatures for the site is the most important design parameter since this determines cracking propensity. Water sensitivity evaluation differs from the systems for fatigue, permanent deformation, and thermal cracking in that mix evaluation for water sensitivity is included in the preliminary (initial) mix design phase. Aging considerations are incorporated in other systems and specimens for testing are either conditioned using short-term or both short- and long-term aging.

In the mix design and analysis process, the compaction method used to prepare specimens is of paramount importance.

Rolling-wheel compaction is recommended for use. It is intuitively appealing for its obvious similarity to field compaction processes. Moreover, extensive studies have demonstrated that it produces uniform specimens with engineering properties similar to those of cores extracted from recently constructed pavements. Rolling-wheel compaction is a comparatively easy procedure to use and enables rapid fabrication of specimens in suitable numbers and shapes for a comprehensive mix design and analysis system. Because specimens produced by rolling-wheel compaction are cored or sawed from a larger mass, all surfaces are cut. Cut surfaces are desirable because air voids can be more accurately measured, comparisons with specimens extracted from in-service pavements are more accurate, specimens are more homogenous, and test results are likely to be less variable. Rolling-wheel compaction also has the advantage that specimens containing large-size aggregate can be produced without difficulty.

Fatigue (Chapter 3)

An improved procedure has been developed for defining the fatigue resistance of asphalt and/or binder aggregate mixes. This procedure permits the determination of the fatigue response of a mix, in 24 hours, at one temperature level, with a reliability of at least 80 percent. Also provided, based on extensive fatigue testing, is a surrogate model utilizing the results of stiffness measurements on the mix.

The results of the program have been incorporated into an innovative design and analysis system for evaluating the fatigue resistance of asphalt-aggregate mixes. This system provides an effective mechanism for the interpretation of laboratory fatigue measurements and for determining the impact of asphalt-aggregate interactions on expected pavement performance. It combines mix testing with traffic loading (repetitions, wheel loads, and tire pressures), environmental conditions (temperature), and the pavement cross-section to assure that, with preselected reliability, fatigue cracking in the asphalt-bound layer will not exceed acceptable limits.

The analysis system assumes that a trial mix has been identified, that traffic and environmental conditions have been determined, and that the pavement cross-section has been designed. It then seeks to judge, with predetermined reliability, whether the trial mix would perform satisfactorily in service. If it would not, the designer can opt for redesigning the mix, strengthening the pavement section, or repeating the analysis using more refined measurements and/or estimates.

For routine mix designs (Level 1), the testing and analysis system has been simplified to the maximum possible extent. Laboratory testing is limited to stiffness measurements, and the primary analysis requires only a single estimate of in-situ strains using traditional assumptions of linear elasticity. Unconventional mixes or uncommon applications, on the other hand, require more extensive testing and analysis for reliable decision making (Level 2). Multiple-temperature fatigue testing must be performed, and analysis must address the complex thermal environment anticipated in situ (Level 3).

Key features of the mix analysis system include the use of temperature conversion factors and quantitative reliability concepts. Temperature conversion factors are used to convert design ESALs to their equivalent at a common reference temperature of 20°C (68°F). These factors have been found to be an effective, simple way of treating environmental temperature effects and of reducing the necessity for extensive multiple temperature testing. Reliability concepts provide a quantitative means for comparatively judging the adequacy of surrogate testing-regression models vis-à-vis laboratory fatigue testing. They permit and encourage a hierarchial approach to mix design which routinely simplifies the process but permits detailed analysis where necessary.

Conceptual development of the mix analysis system has been completed as part of SHRP Project A-003A, and considerable progress has been made toward establishing a readily implementable package for use by material engineers nationwide. In addition to completing the calibration process, one of the key remaining tasks is to validate the analysis system by demonstrating its ability to reliably discriminate among suitable and unsuitable mixes.

Permanent Deformation (Chapter 4)

Results of the SHRP A-003A investigation in the permanent-deformation area provide a new test methodology and equipment to define the propensity of a mix for permanent deformation. The equipment permits the simultaneous application of shear and axial and/or normal stresses to cylindrical specimens as large as 20 cm (8 in.) in diameter and 8.9 cm (3.5 in.) in height. Temperatures of up to 70°C (158°F) can be used and confining pressures to 700 kPa (100 psi) can be applied.

Two levels are recommended for use of the equipment in permanent deformation evaluation. For the Level 1, the simple shear test is performed in the unconfined condition with a single shear stress [e.g. 70 kPa (10 psi)] at the critical temperature T_c. Load is repeatedly applied for one hour (0.1 second time of loading and 0.6 second time interval between load applications) to permit the definition of a relationship between shear strain and stress repetitions. For a particular site the critical temperature corresponds to the temperature at a 5-cm (2-in.) depth at which the maximum permanent deformation occurs for the expected temporal distribution of traffic at the site. Like the fatigue system, temperature conversion factors are used to convert design ESALs to their equivalent at the critical temperature. Reliability considerations have been incorporated as in the fatigue system. The Level 1 methodology permits determination of the suitability of a mix to carry the anticipated traffic in a specific environment.

Level 2 encompasses a suite of tests including constant height shear creep, uniaxial strain, volumetric, and frequency sweep. The first three tests are performed at 40°C (104°F)while the frequency sweep is conducted at temperatures ranging from 4° to 60°C (39° to 140°F). The level 2 procedure permits the estimation of rut depth for some prescribed traffic volume.

The repeated load, constant height, shear test can also be used for mix-design purposes. For a specific asphalt- and/or binder-aggregate mix, specimens are prepared over a range of asphalt contents at an air void content of about 3 percent. A mix is considered suitable in this procedure if it can carry the prescribed number of repetitions in the simple shear test (associated with a specific rut depth) at this 3 percent air void content when tested at the critical temperature for the site.

The test appears to capture the important mix characteristics which define its propensity for permanent deformation and include the following:

- dilation under shear loading;
- increasing stiffness with increasing confinement at elevated temperature;
- negligible volumetric creep;
- residual permanent deformation on removal of load; and
- temperature and rate of loading dependence.

Moreover, by performing the test in repeated loading rather than creep (Level 1 procedure), important differences in the accumulation of permanent deformation are obtained which may be particularly important when modified binders are used.

Low-Temperature Cracking (Chapter 5)

Based on a comprehensive evaluation of existing methods to define the propensity of a mix to low-temperature cracking, the thermal stress restrained specimen test (TSRST) was selected.

The resulting test program confirmed the following:

1. TSRST test results provide an excellent indication of low-temperature cracking resistance of asphalt concrete mixes and are in agreement with rankings based on the physical properties of the asphalt cements used in the mixes.

2. TSRST test results are sensitive to the effects of asphalt source, aggregate type, air void content, degree of aging, and rate of change in pavement temperature. These five variables represent the major factors to be considered in the design of asphalt-aggregate mixes to mitigate low-temperature cracking.

3. TSRST test results can be correlated with specific physical properties of the asphalt cement to facilitate a simplified approach to controlling low-temperature cracking or for preparation of binder specifications.

4. Repeatability of the TSRST is considered to be very good; the coefficient of variation for fracture temperature is less than 10 percent.

5. TSRST more closely simulates in situ conditions.

Results of the validation studies associated with three pavement sites in Alaska, Pennsylvania and Finland together with tests conducted in the Frost Effects Research Facility (FERF) of the U.S. Army Corps of Engineers (COE) provide the following:

1. Cracking behavior of the test roads could be explained with TSRST fracture temperatures for projects in Alaska, Pennsylvania, Finland (one of two investigated), and the United States Army Cold Regions Research and Engineering Lab (USACRREL). For the second project in Finland, there were other factors, in addition to mix properties, affecting low-temperature cracking. Based on the studies it has been concluded that the TSRST can be utilized to predict the low-temperature-cracking propensity of asphalt-aggregate mixes.

2. Preliminary models to predict cracking frequency and temperature for the test roads were developed. Consequently, it appears feasible to develop a model that would predict the development of cracking in all climates as a function of age (time).

For mix design and analysis, three levels are recommended for the thermal cracking system and are differentiated by the amount of testing and subsequent analysis.

Level 1 is to be used for mixes containing conventional asphalts, and requires no mix testing since testing of the pressure-aging-vessel(PAV)-aged asphalt cement residue provides the necessary input data.

Level 2 requires limited testing of the mix using the TSRST. A cooling rate of 10°C per hour is recommended and the tests should be performed on long-term-oven-aged (LTOA) specimens. The analysis consists of using weather data (coldest year in 30) to estimate the pavement surface temperature, and the fracture temperature, T_{frac}, derived from the TSRST. This temperature is used to estimate the propensity for cracking using a regression equation developed from available performance data. By setting a specific level of cracking for one design period, e.g. 10 years, the suitability of the mix can then be judged.

Level 3 requires more detailed testing of the proposed mix in the TSRST using both short-term oven aged (STOA) and LTOA specimens at a cooling rate commensurate with actual site data, e.g., 1°C (1.8°F) per hour. Results of the more extensive test program are used in a viscoelastic analysis system which requires, in addition to the TSRST data, the mix stiffness as a function of temperature and time of loading and the thermal characteristics of the asphalt mix. The program permits an estimate to be made of the increase in crack frequency with time either deterministically or in a probabilistic mode.

While considerable validation has been accomplished, additional effort is required to improve the prediction of cracking frequency.

Aging (Chapter 6)

Procedures have been developed for both short- and long-term aging. Based on the study of new and young field sites, 4 hours of oven aging at 135°C (275°F) appears representative of the short-term aging which occurs in the field during mixing and placement. This is also sufficient for field aging of young projects less than two years old. Two days of long-term oven aging at 85°C (185°F) is representative of a pavement up to five years old depending on the climate. Four days of oven aging at 85°C (185°F) appears to be representative of field aging of about 15 years in a wet-no-freeze zone and about 7 years in a dry-freeze zone. However, it was not possible to develop guidelines for Wet-Freeze or Dry-No-Freeze zones, because no projects of sufficient age could be located. Oven aging at 100°C (212°F) for 1, 2, and 4 days achieves similar stiffnesses as compared to 85°C (185°F) aging for 2, 4, and 8 days, but damages the specimens in the process; therefore, 85°C (185°F) is considered to be more reliable.

It should be noted that the aging of asphalt-aggregate mixes is influenced by both the asphalt and aggregate. Aging of the asphalt alone, and subsequent testing, does not appear to be an adequate means of predicting mix performance because of the apparent mitigating effect aggregate has on aging. Moreover, the aging of certain asphalts is strongly mitigated by some aggregates but not by others. This appears to be related to the strength of the chemical bonding (adhesion) between the asphalt and aggregate. In the mix design and analysis system, short-term aging is recommended for mixes to be tested in Levels 1 and 2 for fatigue, and Levels 1 through 3 for water sensitivity, while both short-term and long-term aging are recommended in Level 3 for fatigue, in Levels 2 and 3 for thermal cracking, and in Level 2 for permanent deformation.

To further analyze the effectiveness of the short-term aging period of 4 hours, additional sites should be selected. In addition, continued monitoring of field projects is needed, particularly for dry-no-freeze and wet-no-freeze zones as well as increasing the number of sites for study. Sites selected should have in-service lives ranging from 1 to 20 or more years to encompass all long-term aging in the field.

The low pressure oxidation (LPO) procedure at 85°C (185°F) also developed in the study should be evaluated further. This approach may be necessary for mixes with relatively low stiffness.

Water Sensitivity (Chapter 7)

For water sensitivity evaluation an Environmental Conditioning System (ECS) has been developed. This system permits cylindrical specimens to be subjected to water conditioning and temperature cycling to reproduce field conditions, and to continuous repeated loading during the high temperature conditioning cycles to simulate the effects of traffic loading. The conditioning procedure includes a series of hot and freeze cycles depending on the climatic regime. For warm climates, three wet-hot cycles of 6 hours duration at 60°C (140°F) are used. For cold climates, a freeze cycle at -18°C (-0.4°F) is added. Resilient modulus tests are performed at the end of a cycle after the specimen has been brought to a temperature of 25°C (77°F). The ratio of the modulus after each cycle to that of the unconditioned modulus, and its trend with number of cycles provide an indication of the effects of water and water vapor on mix performance.

Limited field validation studies have provided criteria for determining whether or not a mix is water sensitive. These criteria include a minimum modulus ratio (0.7) and a measure of the slope of the relationship between the modulus ratio and number of cycles.

Three levels of water sensitivity evaluation are recommended for use in the mix design and analysis system. As with the other systems the three levels are differentiated by the amount of testing and subsequent analysis required. However, this system differs from the others in that the first two levels are associated with the preliminary (initial) mix design, whereas Level 3 additionally provides for a conditioning procedure to be used in conjunction with the accelerated performance-related tests, e.g. permanent deformation (rutting).

Level 1 involves a procedure to test a limited number of specimens of the preliminary mix design (prepared after STOA) subjected to wet-hot cycles (and a freezing cycle if this occurs at site) in the environmental conditioning system (ECS). Criteria which are dependent on the resilient modulus (M_R) ratio (ratio of M_R after conditioning to that in the unconditioned state) have been developed to insure a reasonable level of mix performance in the water-saturated condition.

Level 2 is similar to Level 1 but involves the testing of additional specimens to ascertain the effects of field compaction on performance.

Level 3 requires the same testing as Level 1 to ascertain if water sensitivity is a problem. An evaluation of the performance of the mix is made in the cyclic shear test after a specimen of the mix has been subjected to hot-wet cycling with application of repeated loads during the hot cycles. (N.B., a freezing cycle may be added if the pavement site is subjected to freezing.)

A strong correlation between ECS performance and the number of years of expected field performance has not yet been made due to the relatively young age of the field sections used in the validation studies. A continued program of coring to further validate and refine the role of the ECS test procedure in a mix design program is suggested.

The ECS should be used to provide a systematic look at the effects of variations in volumetric mix proportions, such as gradation, asphalt content, and air voids, on mix performance. The pessimum voids (midrange of voids) concept suggests that mixes with a certain range of air voids level may be prone to water damage due to the structure of the void system. Gradation and asphalt content also will affect the air void structure of a mix.

1

Introduction

A primary objective of SHRP Project A-003A has been the development of a series of accelerated performance-related tests to define asphalt-aggregate interactions which significantly affect pavement performance. While the project included the following three goals, this report describes the results of Items 2 and 3.

1. Extension and verification of findings from related SHRP projects dealing primarily with asphalt properties and their expected effects on performance of asphalt-aggregate mixes.

2. Development of test methods suitable for standardization and which can be used by the highway industry, both government and private agencies, to measure and quantify fundamental material properties which can be used to predict fatigue cracking, low-temperature cracking, and permanent deformation (rutting).

3. Incorporation of procedures to consider the effects of aging and exposure to water in order for laboratory-measured properties to be representative of in-place properties and thus be capable of accelerating these effects to estimate long-term material properties.

A companion report entitled, *Relationship Between Asphalt Properties and Asphalt-Aggregate Mix Performance — Stage 1 Validation*, includes results of research related to Item 1 dealing specifically with validation of the SHRP binder tests and specifications.

In attempting to achieve these goals, a concerted effort has been made to emphasize the measurement of fundamental material properties which, when incorporated into prediction models, would depend less on empirical correlations than has traditionally been the case. Thus, the properties and models should be able to provide reasonably reliable answers to mix design and analysis problems associated with increased frequency and magnitude of traffic loads, higher tire pressures, and effective use of conventional as well as new materials.

While the models developed to predict pavement performance were the responsibility of the SHRP A-005 contractor, it was of paramount importance in the A-003A project to develop performance prediction models as well since the tests to be developed would provide results which could be used to predict mix and/or pavement performance.

The steps used to achieve the goals were as follows:

1. Conduct literature review to identify the state of knowledge relative to test methods, material properties, and prediction models associated with each of the five mix characteristics.

2. Undertake an initial evaluation step involving the selection of viable methods of testing and prediction models based on the literature review and the experience and judgment of the A-003A staff, SHRP staff and representatives of the A-001 contract.

3. Develop methods and procedures for specimen preparation to simulate field conditions which reduce the variability (improve the precision) of each test method or conditioning procedure.

4. Conduct a pilot testing program to obtain test results and to evaluate the ability of each test program to produce data which are sensitive to material properties, useful for prediction models, and feasible for testing laboratories. The result of this step permitted the selection of the candidate test and related analysis.

5. Conduct an expanded testing program using the selected test methods but using a larger number of asphalts from the SHRP Materials Reference Library (MRL).

6. Statistically analyze the test results both from the pilot testing program and the expanded testing program. Planning both testing programs included experiment designs appropriate to the objective of the project.

7. Assess the predictive capabilities of the test results and to evaluate application to engineering problems or considerations.

8. Conduct reliability analyses to provide guidelines for establishing acceptance criteria for the interpretation of test results.

9. Develop mix design and analysis procedures for each of the distress modes which include surrogate or abbreviated methods of testing and analysis for use on less critical construction projects.

This report is intended to provide a brief summary of the processes which lead to the selection of the test methodologies for fatigue, permanent deformation, thermal cracking, aging, and water sensitivity, information to demonstrate the efficacy of the tests to predict performance, and the use of these tests in mix design and analysis.

The comprehensive reports on which the report is based are listed in the References section. These reports provided the requisite details to substantiate the recommendations which have been included herein. In addition, the *References* section contains a listing of some of the published reports and papers as a result of the research performed by the A-003A team. These reports and articles have been indexed for ready reference for the convenience of those interested in continuing the work initiated under the A-003A Project or those requiring more details regarding data acquisition analysis, and interpretation.

Finally, it must be noted that, while the mix testing and analysis procedures which are recommended appear to be more complex and costly than those currently in use, it is believed that the improvement in mix performance made possible by such an approach, particularly for heavy-duty pavements, will quickly justify the expenditures.

Moreover, it is believed that the results of the SHRP program have the potential to upgrade the quality of the entire asphalt pavement industry, both in terms of personnel and materials performance. Accordingly, the authors urge the profession not to provide reasons why the improved technology cannot be implemented, but rather to determine the most efficient strategies for implementing this technology.

2

An Approach to Mix Design and Analysis

Prior to the development of mix testing and analysis procedures it was necessary to develop a framework or model for the entire process. This is illustrated in Figure 2.1. The research resulted in the selection of test methods which include the flexural beam test for fatigue distress, a battery of tests for permanent deformation, the thermal stress restrained specimen test (TSRST) for thermal cracking, and a methodology to evaluate water sensitivity using a specially developed environmental conditioning systems (ECS). To insure that specimens tested in the accelerated performance-related tests are representative of those in situ, special procedures for aging, both short-term and long-term, and mix compaction have also been developed. Secondary test methods involving dynamic and cyclic shear loading have been found to yield useful surrogate measurements for fatigue and permanent-deformation investigations, respectively.

In addition to mix testing, analysis systems are necessary for the proper interpretation of test results and for determining the effect of important asphalt-aggregate interactions on pavement performance. Such systems recognize that mix performance in situ may depend on critical interactions between mix properties and in-situ conditions (the pavement structure, traffic loading, and environmental conditions).

2.1 Outline of Analysis Systems

The analysis systems for fatigue and permanent deformation are similar. For fatigue and permanent deformation it is assumed that a trial mix has been identified, that traffic and environmental conditions have been determined, and that the pavement cross-section has been designed. The analysis system is used to determine, with predetermined reliability, whether the trial mix would perform satisfactorily in service. If it would not, the designer can choose to redesign the mix, strengthen the pavement section, or repeat the analysis using more refined measurement and/or estimates. The several steps in the analysis systems are as follows:

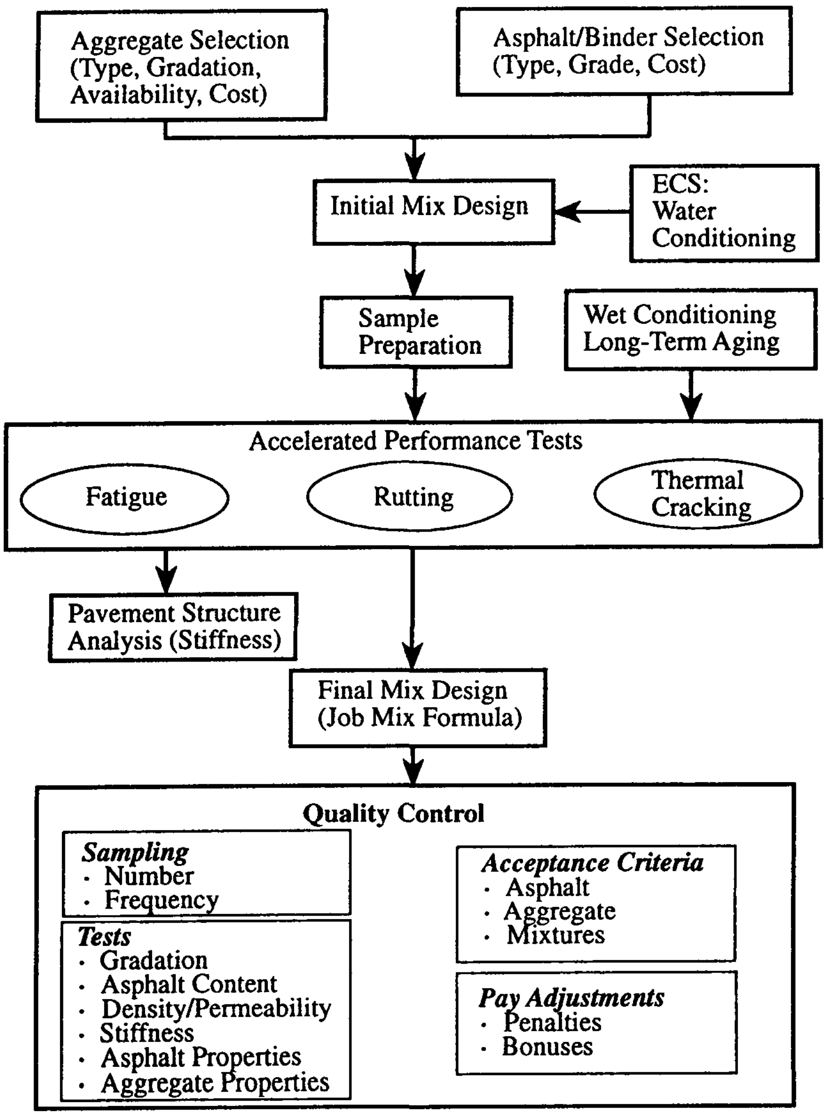

Figure 2.1. Comprehensive mix design and analysis system

1. Determine design requirements for performance (extent of cracking and/or amount of rutting) and reliability (probability of avoiding the acceptance of a deficient mix).

2. Determine expected distribution of in-situ temperatures.

3. Estimate design traffic demand (ESALs).

4. Design structural section.

5. Select trial mix.

6. Measure stiffness of trial mix.

7. Determine design stresses and/or strains under standard axle load.

8. Measure the resistance of the trial mix to fatigue and/or permanent deformation.

9. Compare traffic demand with mix resistance.

10. If inadequate, alter trial mix and/or structural section and perform another analysis iteration.

For thermal cracking, information on ambient air temperature is the most important design parameter since this determines cracking propensity. Water sensitivity evaluation differs from the systems for fatigue, permanent deformation, and thermal cracking in that mix evaluation for water sensitivity is included in the preliminary (initial) mix design phase. Aging considerations are incorporated in the other systems and specimens for testing are either conditioned using short-term or both short- and long-term aging.

The comprehensive system illustrated in Figure 2.1 can be accomplished at different levels. For example, although mix design must incorporate not only mix properties but also in-situ traffic, climatic, and structural conditions, testing and analysis need not be extensive for most routine applications. However, simplistic systems do not yield the greatest possible accuracy, nor are they capable of reliably treating unconventional mixes and uncommon design applications. As a result, testing and analysis details vary depending upon design requirements. For routine use, surrogate or accelerated testing at a single temperature or set of conditions is recommended. For complex designs, the necessary testing is extensive and the full range of in-situ temperatures must be investigated.

2.2 Reliability

Decisions about anticipated mix performance cannot be made with absolute certainty. Although large safety factors can reduce the likelihood of error, the cost consequences can be considerable. Reliability analysis offers the potential for assuring an acceptable level of risk in mix analysis without the costs of excessive safety factors.

For fatigue and permanent deformation, resistance of the mix to loading (N_{supply}) should equal or exceed the traffic demand (N_{demand}) which has been increased by an amount determined by the designer on the basis of a pre-selected level of reliability. The value of N_{demand} is increased by a reliability multiplier, the value of which increases with increases in the level of reliability selected for the design and with increases in the variabilities of mix response and traffic demand estimates. A marginal mix may ultimately be judged to be acceptable by more accurate estimates of mix resistance (for example, by increasing sample size in laboratory testing) or, if possible, by relaxing requirements for the acceptable level of risk.

2.3 Traffic Loading and Temperature Considerations

For purposes of structural design, traffic loading is typically expressed as the number of ESALs per lane that is expected during the pavement's design life. The current proposal is to use this convention for mix design purposes as well. Although load equivalency factors are expected to depend on mode of distress, the AASHTO factors are recommended for initial use. Future development should yield, in short order, refinements that may more accurately account for distress mode.

Testing and analysis over a range of temperatures is considered to be both unnecessary and unacceptable for most routine mix designs. For mixes of typical temperature sensitivity,[1] testing at a single test temperature is recommended. This requires conversion of the design ESALs to its equivalent at the test temperature. Predetermined temperature frequency distributions (by climatic region) and predetermined temperature equivalency factors should suffice for most purposes. For mixes of atypical temperature sensitivity,[2] testing over the full range of temperatures representative of in-situ conditions is considered a necessity.

The temperature for testing normal mixes in fatigue and permanent deformation should be at or near the critical temperature anticipated at highly stressed locations within the pavement structure. More damage occurs at the critical temperature than at any other because of both the frequency of its occurrence and the sensitivity of the mix to damage at this temperature. Any imprecision in temperature equivalency factors is likely to have negligible effect on damage estimates since most of the damage accumulates at temperatures at and near the

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[1]Dense-graded mixes with conventional asphalts.

[2]Mixes with modified binders.

critical temperature, locations where temperature equivalency factors have the smallest possible error.

A temperature of 20°C (68°F) is tentatively recommended for testing typical mixes in fatigue. Not only is this a convenient temperature for production testing in the laboratory, it is also likely to be quite near the critical level. Although it may eventually prove feasible to perform permanent-deformation testing at a single temperature such as 40°C (104°F), for now a site-specific critical temperature such as the temperature at a 5-cm (2-in.) depth is recommended.

For low-temperature cracking, air temperature records of up to 30 years are desirable so that the expected low temperature at the site can be determined with some degree of confidence.

2.4 Specimen Conditioning

In the mix design analysis system two forms of mix or specimen conditioning are utilized, namely *aging* and *hot water plus repetitive loading*.

2.4.1 Aging

Two levels of aging are recommended. A *short-term oven aging (STOA)* procedure has been developed to represent the initial conditions of the mix in the pavement. This procedure consists of curing the loose mix for 4 hours at 135°C (275°F) in a forced draft oven and may be representative of the initial conditions up to one year depending on the severity (temperature regime) of the climate at the site.

The second procedure, termed *long-term aging,* can be accomplished in one of two ways: 1) conditioning the prepared test specimen for 5 days at 85°C (185°F), long-term oven aging (LTOA); or 2) subjecting the specimen to low pressure oxygen for 5 days at 60°C (140°F). The LTOA procedure is recommended for use for dense-graded mixes containing both conventional asphalts and modified binders. While the procedures have been developed for tests on dense-graded mixes containing comparatively stiff binders, they can also be used for other mix types as well. For example, for open-graded mixes and dense-graded mixes containing soft binders, low pressure oxygen aging is recommended since confinement is provided to the specimen during the aging process.

STOA and LTOA is recommended for all specimens evaluated for resistance to thermal cracking and are also recommended for fatigue and permanent deformation evaluation in the more comprehensive systems.

2.4.2 Water

Water sensitivity evaluation is a procedure which is performed in advance of the accelerated performance-related tests. However, hot water conditioning (and possibly freezing conditioning as well) can be accomplished prior to performing the permanent deformation test so that the response characteristics of the "conditioned" mix are available to the designer as well.

The procedure for hot water conditioning could also be used for the beam specimens in fatigue testing if the engineer considers this to be a problem. It may also be desirable to consider beam conditioning when new binders or nonconventional mixes are evaluated.

2.5 Related Factors

2.5.1 Compaction

In the mix design and analysis process the compaction method used to prepare specimens is of paramount importance. The purpose of any laboratory compaction process is to simulate the actual compaction produced in the field. Factors such as particle orientation and aggregate interlock, void content and structure, and the number of interconnected voids should be considered in the selection of a compaction device. During the past five years a number of researchers including the A-003A team, have investigated the relationship between compaction (laboratory and field) and expected performance; highlights of some of these studies are reported in subsequent paragraphs to provide the basis for selection of compaction method recommended for use with this mix design and analysis system.

As a part of NCHRP Project 9-6(1) (von Quintus et al., 1991) various methods of compaction were investigated. While the 9-6(1) researchers indicated a preference for the Texas gyratory compactor (although the data were limited and confounded by variation in void content and laboratory methods used to simulate aging), they noted that rolling wheel compaction gave comparable results to those obtained using the gyratory compactor.

It should be noted that the steel wheel simulator used by NCHRP 9-6(1) researchers is not the same as used in the A-003A project. The NCHRP investigators considered the particular steel wheel simulator to be "relatively unsophisticated in comparison to the typical European type compactors...."

In a subsequent study for SHRP, Button et al. (1992) prepared specimens in the laboratory using three different compaction methods including the Texas gyratory, the Exxon rolling wheel, and the rotating base Marshall hammer. While the researchers concluded that the Texas gyratory compaction most often produced specimens similar to field cores, the results must be qualified because of significant differences in air void contents of compacted specimens.

As a part of the A-003A program, gyratory (Texas type), kneading, and rolling wheel compaction procedures were investigated (Sousa et al., 1991b).

The gyratory compactor was found to place excessive emphasis on the asphalt binder and to inaccurately portray the role of asphalt-aggregate interaction in the performance of properly constructed pavements. Furthermore, the shapes and dimensions of specimens produced by gyratory compactors are limited. Although the kneading compactor is more adaptable for producing a larger variety of sizes and shapes, it may create a more stable aggregate structure than is commonly developed by conventional construction practice, thereby failing to capture the role of the asphalt binder in properly performing pavements. Because the response of rolling-wheel specimens to test loads is typically between that of gyratory and kneading specimens, rolling-wheel compaction is best suited for preparing laboratory specimens. Among the methods investigated, it appears to duplicate field-compacted mixes quite well.

In another study comparing the same three methods of compaction (Harvey 1992) it was concluded:

> Gyratory, rolling, and kneading compaction produce specimens that are significantly different with respect to resistance to repetitive shear permanent deformation test results, with average results differing by more than an order of magnitude between each method for conventional asphalts. This indicates that selection of laboratory compaction method will have at least as much effect on mix performance as aggregate type, binder type, fines content, or air-void content.

European experience has proven the practicality and superiority of rolling-wheel compaction. It is the recommended form of specimen preparation in France and is a major component of the LCPC's methodology for mix design and evaluation (Bonnot 1986). Studies in the United Kingdom (Nunn 1978, Brown and Cooper 1980) as well as the Royal Dutch Shell Laboratory, Amsterdam, (Van Dijk 1975) also demonstrated the effectiveness of rolling-wheel compaction.

Rolling-wheel compaction is intuitively appealing for its obvious similarity to field compaction processes. Moreover, extensive studies have demonstrated that it produces uniform specimens with engineering properties similar to those of cores extracted from recently constructed pavements. Rolling-wheel compaction is a comparatively easy procedure to use and enables rapid fabrication of specimens in suitable numbers and shapes for a comprehensive mix design and analysis system. Because specimens produced by rolling-wheel compaction are cored or sawed from a larger mass, all surfaces are cut. Cut surfaces are desirable because air voids can be more accurately measured, comparisons with specimens extracted from in-service pavements are more accurate, specimens are more homogenous, and test results are likely to be less variable. Rolling-wheel compaction also has the advantage that specimens containing large-size aggregate can be produced without difficulty.

Based on these studies, as well as an evaluation of international experience, it is strongly recommended that rolling-wheel compaction be used for the preparation of laboratory specimens of an asphalt-aggregate mix which are to be evaluated as a part of a comprehensive asphalt-aggregate mix design and analysis system.

2.5.2 Stiffness

Although not included as a research topic in the original work plan, it became evident that consideration must be given to stiffness determinations since they are so important in analyzing mix response in pavement structures. Accordingly, stiffness measurements were made using a number of procedures (including axial dynamic, shear dynamic, diametral resilient, and flexural stiffness) from the bending beam test used for measurement of fatigue properties. Tests have been made over a range of temperatures from 4° to 60°C (39° to 140°F) and loading frequencies from .01 to 16 Hz.

In summarizing the results of the stiffness testing in this report, the information has been included in the sections related to the five mix characteristics evaluated in the A-003A investigation. For example, the association between flexural stiffness and asphalt and aggregate properties has been included in the section on fatigue properties. A similar approach has also been applied to low-temperature cracking, aging, and water sensitivity. In the case of water sensitivity, a slightly revised version of the axial dynamic modulus (ASTM D-3497) has been developed to accommodate specimen configuration.

2.6 Summary

In summary, the comprehensive mix design and analysis system utilizing the newly developed tests, briefly described herein, consists of a series of subsystems in which the mix components, asphalt (or binder) and aggregate, and their relative proportions are selected in a step-by-step procedure to produce a mix which can then be tested and evaluated to ensure that it will attain the desired level of performance in the specific pavement section in which it is to function. The influence of environmental factors, the effects of traffic loading, and the consequence of the pavement structural section design at the selected site are also included in this evaluation.

Depending on the climatic conditions and loading factors to which the specific pavement is subjected, any or all of the distress modes may be evaluated. For example, in a hot, dry climate, it may not be necessary to examine the potential for thermal cracking; whereas, because of the potential for fatigue and rutting, it would be essential to evaluate these two modes. The degree to which this is done is dependent on the design level selected.

While satisfactory resolution of the water sensitivity problem is desirable in the initial design phase, it may not always be possible to completely preclude the deleterious influence of water and/or water vapor. Accordingly, provision is also included in the distress evaluation phase for defining the characteristics of mixes which reasonably reflect the influence of this fact. In addition, the effects of long-term mix aging must be considered. For example, as the mix ages its stiffness increases, leading, in turn, to increased propensity for thermal cracking. These considerations are shown in Figure 2.1 as input at the appropriate place in the mix-design process.

The next five chapters of the report summarize the results of studies on fatigue (Chapter 3), permanent deformation (Chapter 4), thermal cracking (Chapter 5), aging (Chapter 6), and water sensitivity (Chapter 7).

3

Accelerated Performance-Related Tests for Fatigue Properties of Asphalt-Aggregate Mixes

Development of the accelerated performance-related test (APTs) for fatigue consisted of a number of phases: 1) review of the state-of-knowledge in the fatigue area including identification of candidate tests to measure fatigue resistance which can be used for performance (fatigue cracking) prediction; 2) conduct a pilot test program to evaluate the candidate tests and select suitable equipment and a procedure or procedures to define mix fatigue response; 3) conduct an expanded test program using the selected equipment and methodology selected in Phase 2 to provide an expanded database and information for validation of the binder specification; to explore relationships between mix properties, laboratory fatigue response and anticipated pavement performance; and to develop surrogate models of fatigue behavior that, when appropriate, might substitute for laboratory testing; and 4) development of a mix design and analysis system to investigate fatigue cracking.

This chapter presents a brief summary of the results of the test selection investigation, development of a surrogate fatigue model based on the results of the expanded test program, a summary of the relationships, and the framework of a mix design and analysis procedure to consider fatigue cracking (Tayebali et al., 1993).

3.1 Test Method Selection

The primary purpose of the initial investigation was to identify a suitable laboratory test procedure for characterizing the fatigue response of asphalt-aggregate mixes. In addition, primarily as a result of the initial literature review (Tangella et al., 1989), a series of working hypotheses was developed for the fatigue behavior of asphalt-aggregate mixes. This section not only summarizes the principal findings and recommendations regarding such a procedure but also reflects on the working hypotheses that have supported the work and highlights other considerations of importance to the mix design and analysis process.

Based on the findings of the summary report (Tangella et al., 1990) and prior experience of the research team, the following test methods were identified as the most promising for possible use in measuring those mix properties which significantly affect pavement performance:

Flexural fatigue test	•third-point prismatic (beam) •trapezoidal cantilever
Tensile fatigue tests	•diametral •uniaxial tension-compression
Fracture mechanics approach	•K, J, or C*-line integral
Tensile strength and stiffness	•as surrogate for tensile fatigue effects

Table 3.1 provides an overview of the agencies involved and the test methods evaluated. Criteria for test selection included the following:

- sensitivity to mix variables, particularly asphalt properties;
- reasonable simulation of field conditions;
- prediction of fundamental properties that can be used in appropriate design or performance models;
- ease and simplicity of use;
- time requirements;
- ease of implementation, and equipment cost; and
- reliability, accuracy, and precision.

Table 3.1. Test methods evaluated for fatigue program

Agency	Test
University of California, Berkeley (UCB)	• Beam — pulsed loading (1.67 Hz) controlled-stress or -strain • Direct tension — correlation with fatigue • Notched beam — C*-line integral
SWK Pavement Engineering/University of Nottingham (SWK/UN)	• Trapezoidal — sinusoidal loading (20 Hz) controlled-stress
North Carolina State University (NCSU)	• Diametral — pulsed loading (1.67 Hz) controlled-stress or -strain

Implicit in the criteria is also the relevancy of the test method to the specific distress under investigation. The overriding consideration was, however, the ability of the test to relate to pavement performance and to be sensitive to material (asphalt and/or aggregate) properties.

Development and selection of the fatigue equipment and methodology were also based on the ability of the test results to characterize material properties which can be used in mechanistic and/or mechanistic-empirical models. Procedures such as wheel-track tests, although conceptually simple and capable of providing very useful information, are not viable candidates for the accelerated performance-related test (APTs) because fundamental properties are not measured. Test results are difficult to interpret fundamentally and may be useful for a very limited range of traffic, pavement, and environmental conditions. In developing and selecting the accelerated performance-related tests, primary emphasis was given to prior knowledge of pavement performance; past research; and the consensus opinion of researchers, the SHRP research community, and an expert task group (ETG).

The laboratory program considered several variables which are summarized in Table 3.2. Two aggregates (RB and RL) and two asphalts (AAG-1 and AAK-1) from the Materials Reference Library (MRL), were used. Specimens tested at UCB were sawed from larger beams prepared by kneading compaction, while the beam specimens tested by SWK/UN were sawed from slabs prepared by a form of rolling wheel compaction; cylindrical specimens tested by NCSU were prepared by gyratory compaction for testing in the diametral mode. Mixes were prepared at two asphalt contents using a conventional dense aggregate gradation. Table 3.3 lists the asphalt contents utilized.

The experimental design used in this study was the smallest fractional factorial design which permits the estimation of all two-factor interactions, in addition to the main effects of the variables being used. In this case it was determined that one-half of the full factorial (i.e. 32 cells) would be necessary to estimate the main effects and interactions. To obtain estimates of purely experimental error, this 32-factorial combination was fully replicated for a total of 64 tests for the fatigue tests.

Table 3.2. Significant mix and test variables for fatigue study

Variable	Level of Treatment			No. of Levels
	1	2	3	
Aggregate:				
Stripping Potential[a]	Low		High	2
Gradation		Medium		1
Asphalt:				
Temperature	Low		High	2
Susceptibility[a]				2
Content		Optimum	High	
Compaction:				
Air Voids (percent)	4 ± 1		8 ± 1	2
Test Conditions:				
Temperature	0°		20°C	2
Stress and/or Strain Level	Low[b]		High[a]	2

[a]Based on the information from Materials Reference Library (MRL)
[b]Varies with temperature

Table 3.3. Asphalt content used for various mixes

Aggregate Stripping Potential[a]	Temperature Susceptibility[a] Low (AAK, PI=0.5) Asphalt Content[b]		High (AAG, PI=1.5) Asphalt Content[b]	
	Optimum	High	Optimum	High
Low (RB)	5.1	5.7	4.9	5.5
High (RL)	4.3	5.0	4.1	4.8

[a]Based on the information from Materials Reference Library (MRL)
[b]For beam specimens the asphalt content used was by weight of aggregate. For trapezoidal specimens the asphalt content used was by weight of mix

3.2 Laboratory Test Methods

This evaluation highlighted many of the advantages and disadvantages of the candidate accelerated performance-related test. Two of the candidate tests, uniaxial tension tests and fracture mechanics tests, were quickly eliminated after preliminary testing. Gripping the specimen is a difficulty in pure tension testing, and end-cap failure due to stress concentrations was a persistent problem in the limited testing that was completed. Testing for fracture mechanics analysis is thought to be too extensive for routine mix design and analysis; repetitive fatigue tests are necessary both to evaluate the crack initiation process as well as the crack growth rate and notched-beam strength tests are necessary to evaluate the C*-line integral.

Among the remaining three candidate procedures, the diametral (indirect tension) test is obviously very appealing because of its ability to evaluate briquette-shaped specimens. The testing program demonstrated that, although it was reasonably reliable, diametral fatigue was generally inferior to flexural fatigue in the sensitivity of its measurements to mix composition. Measured stiffnesses were comparatively large—perhaps excessively so—and cycles to failure were unreasonably small. With the exception of the effect of aggregate type on stiffness, other mix and loading effects in the diametral testing were found to be reasonable.

In the final analysis, diametral testing is not suitable for routine mix design and analysis because of the following: 1) the high incidence of unacceptable fracture patterns, 2) stress concentrations at the loading platens, and 3) its limitation to controlled-stress loading conditions. Moreover, its variable biaxial stress state, its inability to reverse stress fields, and the confounding influence of permanent deformation within test specimens on their resistance to repetitive tensile loading raised serious concern.

The testing program revealed no striking differences between the flexural beam and the trapezoidal cantilever testing. However, beam measurements were convincingly more sensitive to mix variables than cantilever measurements. With the exception of beam testing's failure to reasonably demonstrate the effect of asphalt content on cycles to failure and questionable stiffness-temperature effects from the cantilever testing, the results of both tests were judged to be reasonable.

Although beam tests are advantageous because of their uniform stress distribution and because gluing is unnecessary, the beam and cantilever tests are considered as equivalent means for assessing the fatigue behavior of asphalt-aggregate mixes. Nevertheless, the authors prefer the beam test because of their familiarity with it and because of the sophistication of the current design of the test equipment and its software interface.

3.2.1 Hypotheses

The investigations reported herein were influenced by a series of working hypotheses about the fatigue behavior of asphalt-aggregate mixes. Further insights regarding these hypotheses, developed as the investigation progressed, are summarized as follows:

Hypothesis 1: *Fatigue cracking is caused by the repetitive application of traffic loads. For typical heavy-duty pavements, fatigue is a result of tensile stresses and/or strains at the underside of the asphalt-aggregate layer(s). The maximum principal tensile strain is considered to be the primary determinant of fatigue cracking.*

Although it has not been subjected to rigorous evaluation, the maximum principal tensile strain is a convenient indication of expected fatigue damage, both for laboratory testing and pavement analysis. However, the dissipated energy during each loading cycle is also an excellent indicator of fatigue response. Furthermore, dissipated energy has greater conceptual appeal than a simple strain indicator because it captures both elastic and viscous effects.

Hypothesis 2: *For purposes of fatigue analysis, the critical stress and/or strain state in the pavement structure can be estimated with acceptable accuracy by the theory of linear elasticity in which the mechanical behavior of the asphalt-aggregate mix is characterized by its modulus of elasticity and Poisson's ratio.*

Because fatigue distress accumulates most rapidly under moderate to cool temperatures and rapid traffic loading, the theory of linear elasticity provides a reasonable indication of the response of the pavement—particularly the asphalt-bound component—to traffic loads. Although the increased accuracy that can be achieved by a linear viscoelastic approach may be unnecessary, it appears that linear viscoelastic modeling may produce useful estimates of the energy that is dissipated during each loading cycle and, thus, might be the preferred approach to structural analysis.

Hypothesis 3: *Testing to destruction under cyclic loading is necessary in order to accurately measure the fatigue response of asphalt-aggregate mixes.*

The primary alternatives to destructive fatigue testing include tensile or flexural strength and stiffness measurements. Fatigue behavior is correlated with these properties, and regression models—calibrated using fatigue test results for a broad range of mixes—are useful both for mix as well as structural design. Fatigue testing is necessary, however, when high accuracy is required, when the candidate mix is only marginally suitable, and when the behavior of unconventional mixes and/or modified materials are being assessed.

Hypothesis 4: *In laboratory fatigue testing, pulsed loading is preferred to sinusoidal loading because the rest period permits stress relaxation similar to that permitted under in-service traffic loading.*

Although both pulsed and sinusoidal loading were used in the study, the experiments were designed neither to investigate possible effects of the different wave forms nor to document possible effects of rest periods. Test results confirmed, however, that mix effects on fatigue response were similar in either pulsed or sinusoidal loading. As a practical matter, accelerated performance-related testing in fatigue requires a loading frequency more rapid than the 1 to 2 Hz frequency which is characteristic of pulsed loading.

Hypothesis 5: *Although pavements become fatigued in response to repeated flexure, fatigue is basically a tensile phenomenon, and test specimens can be evaluated equally well under either tensile or flexural loading.*

Both flexural and tensile testing methods were evaluated herein. The tensile methods proved unacceptable in part because failure patterns frequently indicated undesirable end-cap or loading-platen influences. Fatigue response measured by indirect tension (diametral) loading differed significantly from that measured under flexural loading. Specimens failed much sooner in diametral testing because stresses are not reversed and because permanent deformations are allowed to accumulate. Diametral testing was ultimately judged to be unsuitable for routine use. At the same time, when testing difficulties are overcome, it seems likely that direct uniaxial tension testing will yield accurate measurements of fatigue response.

Hypothesis 6: *Mode of loading is a critical concern in mix design systems because mix effects are quite different between controlled-stress and controlled-strain loading systems. The mode-of-loading effect is more likely due to differences in the rates of crack propagation than in differences in the times to crack initiation.*

The general pattern that stiffer mixes perform better under controlled-stress loading but worse under controlled-strain loading was confirmed by the testing reported herein. Although the importance of mode of loading to informed mix design systems cannot be overstated, proper interpretation of laboratory test results is expected to permit either controlled-stress or controlled-strain testing in the laboratory environment.

Hypothesis 7: *Fatigue tests, accelerated by the application of large stress and/or strain levels, are satisfactory for the purposes of mix design and analysis. That is, for practical purposes, mixes are ranked in essentially the same way regardless of stress and/or strain levels.*

Mixes may be ranked differently at one loading level than at another, that is, the ϵ-N or w_o-N curves for different mixes are not always parallel. Thus, performance at a less destructive loading level cannot always be accurately inferred from testing at a more destructive level. Nevertheless, testing at *two or more* higher levels is sufficient to indicate the behavior at the lower levels to which typical paving mixes are subjected in situ.

Hypothesis 8: *Under simple loading, crack initiation in a given mix is related to strain or stress level as follows:*

$$N_f = a\,(1/\epsilon)^b \quad \text{or} \quad N_f = c\,(1/\sigma)^d$$

where:

N_f = number of load applications to crack initiation;
ϵ,σ = tensile strain and stress, respectively; and
a,b,c,d = experimentally-determined coefficients dependent on test temperature.

These relationships have been consistently confirmed for the ranges of stresses and strains to which laboratory specimens have been subjected. Replacing the strain or stress with the energy dissipated during an initial loading cycle, w_o, yields an equally reliable and accurate expression as follows:

$$N_f = e\,(1/w_o)^f$$

where:

w_0 = initial dissipated energy; and
e,f = experimentally determined coefficients.

There has been no evidence of a fatigue limit, a stress or strain below which repeated stressing does not eventually induce fatigue failure.

Hypothesis 9: *Under compound or mixed loading—due, for example, to multiple temperatures and/or stress or strain levels—cracking in a given mix is initiated when the linear summation of cycle ratios equals one as shown below:*

$$\Sigma\,(n_i/N_i) = 1$$

where:

n_i = number of applications of stress σ_i or strain ϵ_i and
N_i = number of applications to failure at stress σ_i or strain ϵ_i.

The linear-summation-of-cycle-ratios hypothesis was not examined in the current study and remains a viable technique with which to account for the effects of exposure to multiple temperature and/or stress or strain levels. Although cumulative dissipated energy initially seemed to be a promising replacement for the linear-summation-of-cycle-ratios procedure, its sensitivities both to temperature and to load level suggest that it is not a direct replacement for the linear summation of cycle ratios, that is, at the critical location:

$$\Sigma W_i \neq W_D$$

where:

W_i = cumulative dissipated energy under temperature or load i; and
W_D = cumulative dissipated energy at failure.

Nevertheless it seems reasonable that a relationship of the following type might be applicable to compound-loading situations:

$$\Sigma (W_i/W_{Di}) = 1$$

where:

W_i = cumulative dissipated energy under temperature or load i; and
W_{Di} = cumulative dissipated energy to failure under temperature or load i.

Hypothesis 10. *The principles of fracture mechanics represent the most feasible mechanistic approach for estimating rates of crack propagation in pavement structures.*

SHRP A-003A investigations of fracture-mechanics principles focused on laboratory testing requirements instead of pavement analyses. Although the required laboratory testing was deemed unsuitable for routine use, fracture mechanics remains attractive as a mechanistic approach for examining the rate of crack propagation in pavement structures. Fracture mechanics does not offer the potential to deal with crack initiation.

3.3 Considerations of Dissipated Energy

Early literature (van Dijk, 1975) had advanced the notion that a unique relationship might exist between the number of cycles to failure and the cumulative energy that had been dissipated to failure. If so, laboratory testing could be abbreviated, surrogate testing would be more promising, and compound loading could be handled directly. Because of these advantages, considerable effort was made to investigate possible relationships between cycles to failure and cumulative dissipated energy. These efforts related cumulative dissipated energy to failure as follows:

$$W_N = A\,(N_f)^z$$

where:

N_f = number of cycles to failure;
W_N = cumulative dissipated energy to failure; and
A,z = experimentally determined coefficients.

Unfortunately, the uniqueness of this relationship for different types and conditions of testing could not be substantiated. In fact, detailed investigation revealed that the relationships were different for different mixes and were affected by both test temperature and mode of loading.

Despite this disappointment, dissipated energy remains a very useful concept in fatigue investigations. The initial energy that is dissipated during each loading cycle—capturing effects not only of the imposed strain level but also of the dynamic mix properties—is a good predictor of cycles to failure and, thus, is a key component of surrogate models. Furthermore, dissipated energy is highly correlated with stiffness decrements during fatigue testing and helps to explain the effects of mode of loading on mix behavior.

3.4 Extended Test Program and Validation Studies

The purpose of this phase of the investigation was to improve the fatigue test procedure and equipment which had been selected for use; validate the accelerated performance-related test for fatigue; validate the fatigue parameters incorporated in the SHRP binder specifications; and develop a surrogate fatigue model which might be used in lieu of actual fatigue testing. Validation of SHRP binder specification requirements for fatigue was reported in *Relationship Between Asphalt Properties and Asphalt-Aggregate Mix Performance — Stage I Validation* (UCB et al., 1994) and will not be discussed herein.

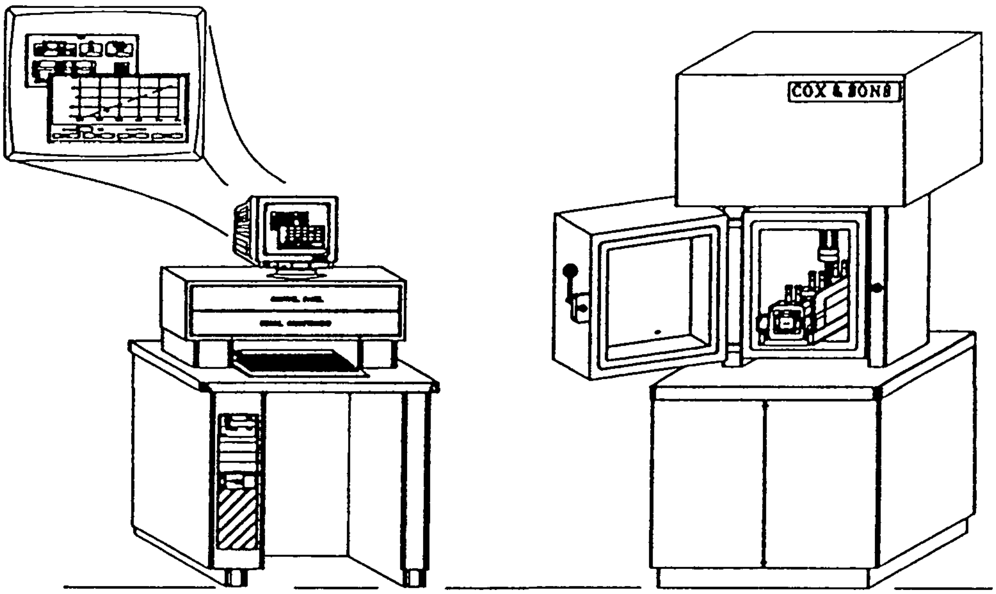
3.4.1 Test Equipment and Procedure

As noted earlier, the flexural beam (third-point loading) fatigue test method conducted in the controlled-strain mode of loading was selected. This mode of loading was considered to be more compatible with the crack propagation concept and pavement fatigue cracking models that were being developed as a part of the SHRP A-005 contract.[3] The resulting equipment is illustrated in Figure 3.1. Beams 63.5 mm × 50.0 mm × 381 mm (2.5 in. × 2.0 in. × 15.0 in.) are utilized. Sinusoidal loads up to 25 Hz frequency can be applied with or without rest periods and temperatures up to 30°C (86°F) can be utilized.

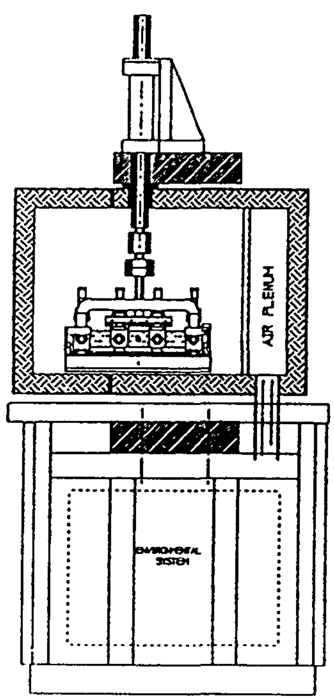
The new equipment has been developed to reduce set-up time to less than 5 minutes per specimen; with this equipment the fatigue response of an asphalt-aggregate mix can be characterized in 24 hours. The procedure includes testing four specimens, each at different strain levels, in the controlled-strain mode of loading at 10 Hz frequency.

Significant improvements in fatigue data resulted from the use of the new test equipment and procedure. The coefficient of variation (CV) for fatigue life has been reduced from almost 90 percent (for the pilot test program) to nearly 40 percent. This is most likely due to improvement in control of the repeated strain, as well as the use of larger size beam specimens fabricated by rolling wheel compaction. The use of rolling wheel compaction virtually eliminated fracturing of the aggregate which was observed in the pilot test program

[3]The equipment which has been developed can be performed in other modes of loading, e.g. controlled-stress; modifications to the software are required and time constraints did not permit these modifications to be made during the course of the project.



a. overall view with computer control unit and controlled temperature chamber



b. side view

Figure 3.1. Schematics of flexural beam fatigue test apparatus

specimens prepared using kneading compaction. These results are used subsequently in the section on mix design and analysis as a part of the design reliability considerations.

3.4.2 Surrogate Model Development

One goal of the expanded fatigue test program was the development of surrogate fatigue models that, when appropriate, might substitute for laboratory fatigue testing. The program included eight MRL asphalts and two aggregates, RD and RH. One asphalt content was utilized for each aggregate (corresponding to Hveem stability of 35).

The program included a study of the influence of air void content (2 levels) and strain magnitude (2 levels). Specimens were only short-term aged and the tests were performed at 20°C (68°F) and at a frequency of loading of 10 Hz. A full-factorial design of 32 mixes was used, which resulted in a total of 128 individual controlled-strain beam fatigue tests with replication.

While the surrogate model was initially developed using the data obtained from the 128 tests referred to above, it was eventually calibrated using the following studies: 1) 8 × 2 expanded test program; 2) mix design experiment; 3) temperature equivalency experiment; 4) Laboratoire Central Ponts et Chaussees (LCPC)-Nantes, France, validation study; and 5) Federal Highway Administration—Accelerated Load Facility (FHWA-ALF) validation study. The calibration included the results from a total of 196 specimens from 44 mixes.

The strain- and energy-dependent models recommended for use in the surrogate mix analysis are the following:

$$N_f = 2.738*10^5 \exp^{0.077\ VFB} (\epsilon_0)^{-3.624} (S''_0)^{-2.720} \tag{3.1}$$

$$N_f = 2.365 \exp^{0.069\ VFB} (W_0)^{-1.882} \tag{3.2}$$

where:

N_f = fatigue life;
ϵ_0 = initial strain, in/in;
S''_0 = initial loss-stiffness, psi;
W_0 = initial dissipated energy per cycle, psi; and
VFB = percent voids filled with bitumen.

The coefficient of determination for the strain based model was 0.79 with coefficient of variation of 90 percent. For the energy-based model, the coefficient of determination was 0.76 with a coefficient of variation of 99 percent.

These equations are based on flexural stiffness measurements. The SHRP materials testing methodology specifies the use of the shear frequency sweep test for Level 1 of the abridged procedure. Regression calibrations are required for estimating flexural stiffness and phase angle from the shear stiffness and shear phase angle at 20°C (68°F) and 10 Hz frequency using the combined data are, therefore, required. Thus, shear stiffness tests were conducted on prismatic specimens with the dimensions — 6.4 cm wide, 5.1 cm tall, and 15.2 cm long (2.5 in. × 2 in. × 6.0 in.). A combined data set of 70 observations was used for model calibrations yielding the following relationships:

$$S_o = 8.560\ (G_o)^{0.913} \qquad R^2 = 0.712 \tag{3.3}$$

$$\sin\phi_{So} = 1.040\ (\sin\phi_{Go})^{0.817} \qquad R^2 = 0.810 \tag{3.4}$$

$$S''_o = 81.125\ (G''_o)^{0.725} \qquad R^2 = 0.512 \tag{3.5}$$

where:

S_o = initial flexural stiffness, psi;
S''_o = initial flexural loss stiffness, psi;
G_o = initial shear stiffness, psi;
G''_o = initial shear loss stiffness, psi;
$\sin\phi_{So}$ = initial sine of phase angle in flexural test; and
$\sin\phi_{Go}$ = initial sine of phase angle in shear test.

Presently, the use of Equation 3.1 developed from the strain-based surrogate model is recommended. The shear stiffness can be employed in the surrogate procedure by the following:

1. Convert the shear loss-stiffness (G''_o) at 20°C (68°F) and 10 Hz frequency to a flexural loss-stiffness (S''_o) at the same temperature and frequency using Equation 3.5.

2. Estimate the fatigue resistance from Equation 3.1.

3.5 Validation Studies

Validation studies were conducted to compare results and ranking of asphalt mixes from specific, accelerated, wheel-track test facilities to those obtained from the A-003A laboratory accelerated performance-related test—the controlled-strain, flexural beam, fatigue test. The specific wheel-track facilities included the SWK laboratory wheel-track device, a full-scale, LCPC, circular, test track at Nantes, France; and the FHWA, accelerated-load-test facility located at McLean, Virginia.

In addition, a limited study was conducted of shift factors recommended by Finn et al. (1986), factors which bring fatigue life estimates from mechanistic analysis into line with measurements of fatigue cracking in selected sections of the AASHO Road Test Pavements. Recommended shift factors included 13.0 for 10 percent cracking (Class 2) in the wheel path and 18.4 for 45 percent cracking. The authors (Finn et al.) suggested further studies to refine these recommendations.

A limited study was conducted as a part of the A-003A fatigue program to validate this recommendation. Validation was not conducted in terms of comparing the results of controlled-strain fatigue tests in the laboratory directly with the performance of full-scale pavement sections. However, results of this investigation do provide an assessment of the reasonableness of the fatigue test program and recommendations and can therefore be considered as a part of the validation effort for fatigue.

Results of the laboratory wheel-track tests conducted at SWK/UN on asphalt mixes containing six core MRL asphalts and one MRL aggregate are summarized as follows:

1. For mix stiffness, ranking of core MRL asphalts based on indirect tensile stiffness at 20°C (68°F) was similar to the ranking obtained based on the flexural stiffness determined at UCB.

2. For fatigue life, ranking of core MRL asphalts based on fatigue life (N_1) from wheel-track testing was similar to the ranking based on fatigue life obtained from flexural beam fatigue tests performed at UCB.

Controlled-strain flexural beam fatigue test results on LCPC-Nantes materials are in good agreement with test results obtained by SHELL-KSLA as well as by the LCPC. The rankings of the performance of asphalt mixes observed in the circular wheel-track test section, however, are not in agreement with the fatigue lives estimated using any of the laboratory test methods or the surrogate fatigue model. Findings from the LCPC-Nantes validations study are summarized as follows:

1. Flexural beam fatigue testing in the controlled-strain and controlled-stress mode of loadings at UCB and SHELL-KSLA, respectively, and trapezoidal cantilever fatigue testing in controlled-strain and controlled-stress mode of loadings at LCPC and SWK/UN, predicted that mixes containing asphalt B (5.4 percent) would show better fatigue performance compared to mixes containing asphalt A (5.4 percent). Similarly, fatigue tests indicated that the high modulus mix would also show better fatigue performance compared to mixes containing asphalt A (4.6 percent).

2. Fatigue life estimated for the in-situ structures containing these different mixes based on both the laboratory fatigue tests and the surrogate fatigue model also confirmed the above ranking of the asphalt mixes.

3. Wheel-track test results did not support the ranking of the mixes obtained from the laboratory fatigue results. Due to the concern that the asphalt wearing course may have affected the test results in the first wheel-track experiment, a second wheel-track experiment was conducted without the asphalt wearing course. Results of the second wheel-track test were generally identical to those of the first experiment. The pavement section containing asphalt B (5.4 percent) exhibited more surface cracking than did the section containing asphalt A (5.4 percent) at the same number of wheel load passes; and the section containing the high modulus mix had more surface cracking than the

section containing asphalt A (4.6 percent) at the same number of wheel load passes.

For the FHWA-ALF validation study, comparison of the actual and estimated fatigue lives was limited due to unavailability of the details of the experimental results from FHWA. The variations in temperature and material properties of the various structural layers were unavailable. Preliminary results from the pavement testing reported by FHWA indicate fatigue lives to surface crack initiation of approximately 55,000 and 100,000 load repetitions for the single and dual tire configurations, respectively. This is in good agreement with the laboratory-based estimates of fatigue lives of approximately 45,000 load repetitions for the single tire, and 80,000 for dual tire configurations.

Relative to the shift factor analysis, the laboratory measurements produced reasonable results in terms of suitable in situ performance for the 44 mixes tested in this phase (90 percent reliability level).

This determination of mix suitability was accomplished using layered-elastic analysis to determine strains in the asphalt concrete based on individual mix stiffnesses at 20°C (68°F). Fatigue lives at the calculated strain levels were then ascertained for each of the 44 mixes. These numbers of repetitions were compared with the estimated ESALs based on the AASHTO design procedure (AASHTO 1986) which had been converted to their equivalent at 20°C (68°F) by a temperature conversion factor. Tables 3.4 and 3.5 contain summaries of these computations. NCHRP Report 291 estimate is based on the fatigue model developed by Finn et al. (1986) which is based on controlled-stress tests.

While many of the mixes were considered acceptable at the smallest traffic loading (1,000,000 ESALs), the percentage of suitable mixes decreased with increases in traffic loading. Although this seems to indicate that AASHTO design procedures are more conservative, vis-à-vis fatigue cracking, at smaller traffic levels than at larger ones, it also suggests that requirements for mix quality increase with increases in traffic level (despite the AASHTO requirements for thicker pavement sections with increased loading). The analysis also suggests that the loading environment is more severe in the southwestern region of the United States than in the northeast. Although such differences might be reduced or even eliminated by region-specific structural designs, it is not unreasonable to expect that mixes that are suitable in one region of the country might not be suitable in another.

Considering the fact that the mixes tested in this research intentionally spanned a wide range of likely mix performance, a shift factor of 13 is certainly within an acceptable range for use with the mix design and analysis procedures recommended herein. However, a factor of 10 would be somewhat more discriminating and is recommended initially for design applications. For 45 percent cracking, a shift factor of about 14 is consistent with NCHRP Report 291 findings.

Because AASHTO structural design procedures are based on overall pavement serviceability rather than on specific distress mechanisms, analyses such as the above cannot yield accurate shift-factor estimates. At the same time, the above analysis has confirmed that the shift

Table 3.4. A-003A mix suitability for Northeastern United States (Region I-A)

Variable		Pavement Structure: 4 in. Surface 12 in. Base	4 in. Surface 17 in. Base	8 in. Surface No Base	8 in. Surface 6 in. Base	8 in. Surface 12 in. Base
Design ESALs		1,000,000	4,000,000	1,000,000	4,000,000	16,000,000
Temperature Conversion Factor		0.614	0.614	0.920	0.920	0.920
$ESALs_{20°C}$		614,000	2,456,000	920,000	3,680,000	14,720,000
Shift Factor		13.0	13.0	13.0	13.0	13.0
N_{design}		47,000	189,000	71,000	283,000	1,132,000
Trial Reliability Multiplier		3	3	3	3	3
Trial Minimum N_{supply}		141,000	567,000	213,000	849,000	3,396,000
$Var\{Ln(N_{supply})\}$		0.177	0.250	0.196	0.276	0.387
Reliability Multiplier ($Var\{Ln(N_{demand})\} = 0.3$ and 90% Reliability)		2.42	2.58	2.46	2.64	2.89
Minimum N_{supply} ($M \cdot N_{demand}$)		114,000	488,000	175,000	747,000	3,271,000
Percentage of Suitable A-003A Mixes	NCHRP 291	0	0	55	11	0
	A-003A	95	73	100	95	75

Table 3.5. A-003A mix suitability for Southwestern United States (Region III-B)

Variable		Pavement Structure				
		4 in. Surface 12 in. Base	4 in. Surface 17 in. Base	8 in. Surface No Base	8 in. Surface 6 in. Base	8 in. Surface 12 in. Base
Design ESALs		1,000,000	4,000,000	1,000,000	4,000,000	16,000,000
Temperature Conversion Factor		0.838	0.838	1.839	1.839	1.839
$ESALs_{20°C}$		838,000	3,352,000	1,839,000	7,356,000	29,424,000
Shift Factor		13.0	13.0	13.0	13.0	13.0
N_{design}		64,000	258,000	141,000	566,000	2,263,000
Trial Reliability Multiplier		3	3	3	3	3
Trial Minimum N_{supply}		192,000	774,000	423,000	1,698,000	6,789,000
$Var\{Ln(N_{supply})\}$		0.191	0.269	0.232	0.327	0.459
Reliability Multiplier ($Var\{Ln(N_{demand})\} = 0.3$ and 90% Reliability)		2.45	2.63	2.54	2.76	3.05
Minimum N_{supply} ($M \cdot N_{demand}$)		157,000	678,000	358,000	1,562,000	6,902,000
Percentage of Suitable A-003A Mixes	NCHRP 291	0	0	25	0	0
	A-003A	91	66	98	91	61

factors recommended by Finn et al. (1986) generally allow reasonable modeling and, following adjustments to reflect a different mode of loading and to accommodate reliability analysis, reasonable judgments about the adequacy of specific mixes to resist fatigue cracking in service. Such shift factors certainly provide an effective beginning, and design agencies are encouraged to start with them and to make refinements based on local experiences.

3.6 Mix Design and Analysis

The proposed mix design and analysis system recognizes that mix performance in situ may depend on critical interactions between mix properties and in situ conditions (the pavement structure, traffic loading, and environmental conditions). It thus provides not only for sensitivity to mix behavior but also for sensitivity to the in situ traffic, climatic, and structural environment as well. Because a hierarchial approach has been adopted, the analysis system is relatively simple for routine purposes but permits more exhaustive investigation when necessary: reliability is a key ingredient at all levels and for all applications. The structure of the analysis system provides the flexibility necessary to accommodate future refinements and extensions.

The methodology assumes that a trial mix has been proportioned, that traffic and environmental conditions have been determined, and that the pavement cross section has been designed. The analysis system seeks to determine, with predetermined reliability, whether the trial mix would perform satisfactorily in service. If it would not, the designer can choose to redesign the mix, strengthen the pavement section, or repeat the analysis using more refined measurements and/or estimates. The several steps in the system are identified as follows:

1. Determine design requirements for reliability (probability of avoiding the acceptance of a deficient mix) and performance (extent of permissible fatigue cracking).

2. Determine the expected distribution of in situ pavement temperatures.

3. Estimate design traffic demand (ESALs).

4. Select trial mix.

5. Prepare test specimens and condition as required.

6. Measure stiffness of trial mix.

7. Design pavement structural section.

8. Determine design strain under standard axle load.

9. Determine the resistance of the trial mix to fatigue (N_{supply}) in the laboratory or by regression estimate.

10. Apply a shift factor to the travel demand (ESALs) to account for differences between laboratory and in situ conditions (such as traffic wander and crack propagation).

11. Compare traffic demand (N_{demand}) with mix resistance (N_{supply}).

12. If N_{demand} exceeds N_{supply}, reanalyze current trial mix with procedures that yield greater accuracy or alter trial mix and/or structural section and perform another analysis iteration.

Key features of the mix design and analysis system are briefly described in immediately following sections.

3.6.1 *Levels of Analysis*

Although mix designs must consider not only material properties but also in situ traffic, climatic, and structural conditions, testing and analysis need not be extensive for most routine applications. However, simplistic systems do not yield the greatest possible accuracy, nor are they capable of reliably treating unconventional mixes or uncommon design applications. As a result, testing and analysis details must vary depending upon design requirements. For routine use, surrogate or accelerated fatigue testing at a single temperature is recommended. For complex designs, on the other hand, the necessary testing is extensive, and the full range of in situ temperatures must be investigated.

The analysis process described herein is thus a hierarchal one. The Level 1 requires only stiffness testing and uses a previously calibrated regression model to estimate fatigue life. The Level 2 replaces regression estimates with fatigue test measurements but limits the testing and analysis to a single temperature. Level 3, the most complex, requires a complete battery of fatigue tests at multiple temperatures.

3.6.2 *Traffic Loading and Temperature Considerations*

For purposes of structural design, traffic loading is typically expressed as the number of ESALs per lane expected during the pavement's design life. The analysis system uses this convention for mix design purposes as well. Although distress-dependent load equivalency factors may eventually be developed, AASHTO load equivalency factors are recommended for initial use. Thus, the load equivalency factors that are used for pavement structural design will also be used for mix design.

Mix testing and analysis over a range of temperatures is considered to be both unnecessary and unacceptable for most routine mix designs. For mixes of typical temperature sensitivity, testing at a single test temperature is recommended. This requires conversion of the design ESALs to its equivalent at the test temperature. Predetermined temperature frequency distributions (by climatic region) and predetermined temperature equivalency factors should suffice for most purposes. For mixes of atypical temperature sensitivity, testing over a range of temperatures representative of in situ conditions is considered necessary.

The most desirable temperature for testing normal mixes would be at or near the *critical* temperature anticipated at highly stressed locations within the pavement structure[4]. The

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[4]Testing at an effective temperature is also a possible alternative. The effective temperature is defined to be the one at which single temperature testing and analysis would yield results identical to multi-temperature testing with analytical accumulations of distress over the range of temperatures anticipated in situ. Identifying the effective temperature is similar in many respects to the process of developing temperature equivalency factors. The latter process, however, is more transparent to the mix designer and is expected to permit fatigue testing at a common temperature for applications covering much of the continental United States.

other. More damage occurs at this temperature because of both the frequency of its occurrence and the sensitivity of the mix to damage at this temperature. Any imprecision in temperature equivalency factors that are referenced to the critical temperature is likely to have negligible effect on damage estimates since most of the damage accumulates at temperatures at and near the critical temperature. At these locations temperature equivalency factors have the smallest possible error. However, a major disadvantage of testing at the critical temperature is that, this temperature is location- and pavement structure-specific, test results are not generally transferable.

Accordingly, a temperature of 20°C (68°F) is recommended for testing typical mixes in fatigue. This is a convenient temperature for production testing in the laboratory, and is near the critical temperature level at many locations within the continental United States. The advantage of single-temperature testing in production laboratories outweighs the possible loss in accuracy from testing at a temperature different from (although near) the critical temperature.

3.6.3 Reliability

Decisions about anticipated mix performance cannot be made with absolute certainty. Although large safety factors can reduce the likelihood of error, their cost consequences can be considerable. Reliability analysis offers the potential for assuring an acceptable level of risk in mix analysis without the costs of excessive safety factors.

The analysis system requires that mix resistance (N_{supply}, the laboratory fatigue life) exceed traffic demand (N_{demand}, the adjusted field ESALs estimate) by an amount which is carefully chosen to meet reliability requirements. This is accomplished by applying a reliability multiplier (M) to N_{demand} before it is compared to N_{supply}. The reliability multiplier (M) increases with increases in design reliability level as well as with increases in the variabilities of mix-resistance and traffic-demand estimates. A mix initially judged to be marginal may ultimately be judged to be acceptable by more accurate estimates of mix resistance (increasing sample size in the laboratory testing) or, if possible, by relaxing requirements for the acceptable level of risk.

3.6.4 Mechanistic Analysis

The maximum principal tensile strain at the underside of the asphalt layer governs the initiation of fatigue cracking in situ. Mixes will perform adequately only if they can sustain the necessary repetitions of this strain level without cracking. For mix-analysis purposes, multilayer elastic theory provides a convenient and sufficiently accurate means for estimating the maximum strain anticipated in situ at 20°C (68°F) under the standard axle load. Laboratory testing or regression estimation is then used to establish the fatigue resistance at this critical strain level.

3.6.5 *Overview of Analysis System*

Distinguishing characteristics of the fatigue analysis system are displayed in Table 3.6. The three levels of the analysis hierarchy are differentiated primarily by the extent of required testing, the treatment of temperature, and analytical requirements. Mixes of typical temperature sensitivity can be analyzed at a single temperature (Level 1 or 2). Level 1, based on shear frequency sweeps instead of fatigue testing, is applicable to conventional dense graded mixes. Unconventional mixes require fatigue testing and analysis of the type characteristic of Level 2. Finally, the multiple temperature testing and analysis of Level 3 is necessary for mixes of atypical temperature sensitivity. Table 3.7 summarizes the recommended level of fatigue testing and analysis for mixes of varying types.

For all levels, the design traffic is expressed in terms of the number of AASHTO ESALs in the critical lane during the pavement's design life, adjusted to its equivalent at 20°C (68°F). A shift factor must be applied to this traffic estimate to enable direct comparisons between the design traffic estimate and laboratory measurements. The shift factor accounts for such effects as crack progression, traffic wander, construction variability, differences between field and laboratory modes of loading, etc. The end result of the traffic analysis is an estimate of traffic demand (N_{demand}) that is commensurate with laboratory fatigue measurements.

Mix resistance to fatigue distress (N_{supply}) is ascertained from laboratory measurements using either surrogate testing and a regression model (Level 1) or direct fatigue testing (Levels 2 and 3). In either case, the mix is characterized as a linearly elastic material, and the appropriate strain level is determined by simulating the pavement response to the standard axle load at a temperature of 20°C (68°F).

Conceptual development of the mix analysis system has been completed as part of SHRP contract A-003A, and considerable progress has been made toward establishing a readily implementable package for use by material engineers nationwide. In addition to completing the calibration process, one of the key remaining tasks is to validate the process by demonstrating its ability to reliably discriminate between suitable and unsuitable mixes.

3.7 Abridged Analysis System

The abridged analysis system, which includes both Levels 1 and 2, is generally applicable to mixes having binders of typical temperature sensitivity. The evaluation of conventional mixes relies initially on Level 1 analysis. Fatigue testing and Level 3 analysis would be employed only when added accuracy is desired or for the evaluation of unconventional mixes.

Table 3.6. Distinguishing characteristics of fatigue analysis systems

Variables		Level 1: Abbreviated analysis with surrogate testing	Level 2: Abbreviated analysis with limited fatigue testing	Level 3: Comprehensive analysis with full fatigue testing
Testing	Type	Dynamic properties from shear frequency sweeps	Flexural beam fatigue	Flexural beam fatigue
	Temperature	20°C	20°C	Multiple
In Situ Conditions	Traffic	Equivalent ESALs at 20°C	Equivalent ESALs at 20°C	Equivalent ESALs at 20°C
	Structure	Tensile strain under "standard" load at 20°C	Tensile strain under "standard" load at 20°C	Tensile strain under "standard" load at 20°C
	Temperature	Frequency distribution at bottom of surface layer	Frequency distribution at bottom of surface layer	Frequency distribution at bottom of surface layer
Analysis	Mechanistic	Multilayer elastic	Multilayer elastic	Multilayer elastic
	Damage	Preanalysis (temperature equivalency factors for design ESALs)	Preanalysis (temperature equivalency factors for design ESALs)	Development of unique temperature equivalency factors for design ESALs

Table 3.7. Recommended level of fatigue testing and analysis

Mix Characteristics	Level 1: Abbreviated analysis with surrogate testing	Level 2: Abbreviated analysis with limited fatigue testing	Level 3: Comprehensive analysis with full fatigue testing
Dense graded mixes with conventional binders of typical temperature sensitivity	Recommended	Optional for increased accuracy	Optional for increased accuracy or complete mix cataloging
Unconventional mixes with binders of typical temperature sensitivity	Not applicable	Recommended	Optional for increased accuracy, complete mix cataloging, or investigative analyses
Mixes with binders of atypical temperature sensitivity	Not applicable	Not applicable	Required

1. Determine Design Requirements for Reliability and Performance

Design reliability and performance requirements are set by the individual design agency. Presumably, they reflect the importance of the paving project as evidenced by such factors as highway functional classification, traffic volume and the tradeoffs between benefits and costs. The analysis system proposed herein enables the designer to select any level of reliability, the probability that an asphalt mix will provide satisfactory performance throughout the design period. However, because of the highly variable nature both of asphalt mixes and of conditions encountered in situ, the costs associated with designs of very high reliability are likely to be quite large.

Performance requirements in fatigue generally specify the extent of permissible fatigue cracking expressed as a percentage of the pavement or wheel-track surface area. Unfortunately, the analysis system proposed herein has not yet been calibrated to the extent that would permit the designer to evaluate possible effects of varying performance levels. The recommended level targeted by the current procedure limits cracking to approximately 5 percent of the pavement surface area within the design lane or approximately 10 percent within the wheel-tracks.

2. Determine Expected Distribution of In Situ Temperatures

Pavement analysis in the abridged procedure assumes a uniform temperature of 20°C (68°F) throughout the asphalt layer. However, to effectively treat the destructive effects of traffic under other temperature conditions, it is necessary to know the expected frequency distribution of in situ temperatures at the underside of the asphalt layer. The FHWA's Integrated Model provides a relatively convenient way to determine this distribution at any location within the continental United States. Computation time can be reduced without seriously jeopardizing accuracy by limiting the analysis to one-half of the annual total of 8760 hours. Table 3.8 illustrates that regional estimates may eventually prove sufficient for most applications.

Each design agency is required to determine temperature distributions within its geographical jurisdiction only when initially setting up its mix design and analysis system. The process need not be repeated each time a new mix is analyzed. For mixes with typical temperature sensitivities, the design and analysis computations are shortened by the use of temperature conversion factors (see Step 3, below).

3. Estimate Design Traffic Demand (ESALs)

The starting point for estimating the traffic demand (ESALs) for mix design is the number of ESALs anticipated within the design lane during the design period. Adjusting this estimate to yield the equivalent number of ESALs *at a pavement temperature of 20°C (68°F)* requires the use of the temperature frequency distribution, temperature equivalency factors, and the assumption, in the absence of other information, that the accumulation of ESALs is

Table 3.8. Frequency distribution (percent) of pavement temperature

	Northeastern United States (Region I-A)		Southwestern United States (Region III-B)	
Mid-range Temperature at Bottom of Asphalt Layer (°C)	4 in. Asphalt Layer	8 in. Asphalt Layer	4 in. Asphalt Layer	8 in. Asphalt Layer
-2.5	7.6	2.1	...	...
0.0	17.8	20.8	...	...
2.5	2.9	3.4	0.8	...
5.0	3.9	3.9	2.4	0.8
7.5	4.3	4.6	3.6	3.5
10.0	4.6	5.4	4.6	6.4
12.5	5.2	5.8	5.8	7.4
15.0	5.8	6.2	6.8	6.9
17.5	6.9	6.8	7.1	6.9
20.0	9.0	7.6	7.1	6.9
22.5	7.0	10.8	7.8	7.4
25.0	5.9	9.4	10.6	8.3
27.5	5.2	7.1	8.0	10.4
30.0	4.6	5.6	7.8	12.6
32.5	3.8	0.3	5.3	9.0
35.0	3.4	...	6.0	8.0
37.5	2.0	...	4.5	5.2
40.0	...	...	4.0	0.1
42.5	...	...	4.2	...
45.0	...	...	3.1	...
47.5	...	...	0.5	...

uniformly distributed through the hours of the year (Tayebali et al., 1993). Table 3.9 illustrates the computations that are required. If detailed traffic forecasts are available, nonuniform ESAL accumulations can easily be handled as well.

Although the process of calibrating temperature equivalency factors is rather tedious (Tayebali et al., 1993), it is a one-time process that need not be repeated for other mixes which employ binders of normal temperature sensitivity. Ultimately, when temperature conversion factors[5] such as illustrated by Table 3.10 are developed for a particular jurisdiction, the detailed computations of Table 3.9 will be unnecessary. A single factor that is sensitive to asphalt layer thickness and geographical area would then be available to convert design ESALs to their equivalent at 20°C (68°F).

4. Select Trial Mix

Using preselected asphalt, additives, and aggregate, the trial mix is initially designed either by the design agency's conventional practice or by SHRP's volumetric proportioning procedure (SHRP 1994). Subsequent redesigns are evaluated as desired at the discretion of the materials engineer.

5. Prepare Test Specimens and Condition as Required

Cylindrical specimens 5 cm in diameter and 15 cm in height (2 in. × 6 in.) for shear frequency sweep testing and beams [50 mm × 63 mm × 38 mm (2 in. × 2.5 in. × 15 in.)] for flexural fatigue testing are prepared by rolling-wheel compaction in accordance with SHRP Test Method M-008. Before testing, all specimens are subjected to short-term oven aging in accordance with SHRP Test Method M-007. Level 2 analysis requires flexural fatigue testing while Level 1 analysis uses shear frequency sweep testing as a surrogate for fatigue testing.

6. Measure Stiffness of Trial Mix

The abridged procedure requires an estimate of the flexural stiffness modulus of the asphalt-aggregate mix at 20°C (68°F). This estimate is used in the multilayer elastic analysis to determine the critical level of strain to which the mix is subjected under the standard traffic load.

The SHRP materials testing protocol is expected to specify shear frequency sweep tests for all conditions (all distress modes, mixes, and levels of analysis). For measurements at 20°C

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[5]Developed from temperature equivalency factors computed according to the procedure illustrated in Table 3.9 (Deacon et al. 1993).

Table 3.9. Illustrative computation of equivalent ESALs at 20°C

Midrange Temperature (°C)	Frequency	ESALs	Temperature Equivalency Factor	Equivalent Design ESALs
-2.5	f_1	$ESAL_d \times f_1$	TEF_1	$ESAL_1 \times TEF_1$
0.0	f_2	$ESAL_d \times f_2$	TEF_2	$ESAL_2 \times TEF_2$
2.5	f_3	$ESAL_d \times f_3$	TEF_3	$ESAL_3 \times TEF_3$
...	...	...	...	...
42.5	f_{n-2}	$ESAL_d \times f_{n-2}$	TEF_{n-2}	$ESAL_{n-2} \times TEF_{n-2}$
45.0	f_{n-1}	$ESAL_d \times f_{n-1}$	TEF_{n-1}	$ESAL_{n-1} \times TEF_{n-1}$
47.5	f_n	$ESAL_d \times f_n$	TEF_n	$ESAL_n \times TEF_n$
Equivalent ESALs at 20°C ($ESAL_{20°C}$)				$\Sigma(ESAL_i \times TEF_i)$

Table 3.10. Temperature conversion factor for design ESALs

Region	4 in. Pavement	8 in. Pavement
I-A	0.614	0.920
I-B	0.760	1.422
I-C	0.826	1.130
II-A	0.531	0.848
II-B	0.740	1.473
II-C	0.859	1.816
III-A	0.564	0.849
III-B	0.838	1.839
III-C	0.934	1.922

(68°F) and 10 Hz, flexural properties can be estimated from shear properties through the following regression equations:

$$S_o = 8.560(G_o)^{0.913} \qquad R^2 = 0.712 \tag{3.3}$$

$$S''_o = 81.125(G''_o)^{0.725} \qquad R^2 = 0.512 \tag{3.5}$$

where:

S_o = initial flexural stiffness at 50th loading cycle in psi;
G_o = shear stiffness at 10 Hz in psi;
S''_o = initial flexural loss stiffness at 50th loading cycle in psi; and
G''_o = shear loss stiffness at 10 Hz in psi.

For Level 1 analysis, estimates of flexural stiffness and flexural loss stiffness are determined using the above equations. Shear frequency sweep tests, conducted in accordance with SHRP Test Method M-003, allow sufficiently accurate estimates of G_o and G''_o from measurements on a single briquette specimen. For Level 2 analysis, fatigue testing at 20°C (68°F) and 10 Hz yields direct estimates of all necessary flexural properties.

7. Design Structural Section

Because mix performance in fatigue depends on the pavement's structure, the pavement cross section must be known or assumed as a precondition to mix evaluation. Structural design is accomplished according to the design agency's normal procedures.

8. Determine Design Strain Under Standard Axle Load

Multilayer elastic analysis is used to determine the design strain, the maximum principal tensile strain at the bottom of the asphalt layer, under the "standard" AASHTO axle load. The standard load is a 80,000 kN (18,000 lb), single-axle, dual-tire load. A uniform contact pressure of 590 kPa (85 psi) and a tire spacing of 30.5 cm (12 in.) are assumed. The analysis is based on a temperature of 20°C (68°F) distributed uniformly throughout the pavement section. The flexural stiffness modulus of the asphalt-aggregate layer is measured directly (Level 2) or estimated using Equation 3.6 from shear stiffness measurements (Level 1), and its Poisson's ratio is assumed to be 0.35. Moduli and Poisson's ratios of other layers, representing annual average conditions, are determined in accordance with standard AASHTO procedures. Laboratory testing of substrata materials is considered unnecessary for designing the asphalt-aggregate mix.

9. Determine the Resistance of the Trial Mix to Fatigue

For Level 1 analysis, fatigue resistance is estimated from the following, previously calibrated, regression model:

$$N_{supply} = 2.738\ 10^5\ e^{0.077 \cdot VFB}\ \epsilon_o^{-3.624}\ S_o''^{-2.720} \qquad R^2 = 0.79 \tag{3.6}$$

where:

N_{supply} = the number of load repetitions to 50-percent reduction in stiffness (crack initiation);
e = base of the natural logarithms;
ϵ = the flexural strain in in/in;
S_o'' = the initial flexural loss stiffness at 50th loading cycle in psi (estimated by Equation 3.5); and
VFB = the voids filled with bitumen in percent as measured using the frequency-sweep specimens or as determined from the volumetric proportioning process.

For Level 2 analysis, fatigue resistance is measured in the laboratory by subjecting beam specimens to repeated flexure [20°C (68°F) at 10 Hz frequency] in accordance with SHRP Test Method M-009. The minimum testing program, which can usually be completed within 24 h, involves four specimens subjected to strain levels expected to induce failure at approximately 10,000, 35,000, 100,000, and 350,000 load cycles (or 20 minutes, 1 h, 3 h, and 10 h, respectively). If the required accuracy cannot be achieved by testing four specimens, additional specimens must be tested. At the completion of testing, a model of the following form is fit to the data:

$$N_f = K_1 \epsilon^{K_2} \tag{3.7}$$

The fatigue life (N_{supply}) corresponding to the design strain is then computed using Equation 3.6.

10. Apply a Shift Factor to the Travel Demand (ESALs)

Laboratory estimates of fatigue life (N_{supply}) can be compared with service requirements ($ESAL_{20°C}$) only after the application of a suitable shift factor. The shift factor is applied as follows:

$$N_{demand} = \frac{ESAL_{20°C}}{SF} \tag{3.8}$$

where:

N_{demand} = design traffic demand (laboratory-equivalent repetitions of standard load);
$ESAL_{20°C}$ = design ESALs adjusted to a constant temperature of 20°C; and

SF = empirically-determined shift factor.

Shift factors recommended for application initially depend upon the amount of cracking that is permissible as follows:

10.0 for 10 percent cracking in the wheel path and
14.0 for 45 percent cracking in the wheel path.

Because experience with these recommendations is limited, design agencies are encouraged to consider adjustments that reflect their historical experiences with mix performance. Future research should eventually help to guide these efforts.

11. Compare Traffic Demand (N_{demand}) with Mix Resistance (N_{supply})

Satisfactory mix performance demands that the mix resistance (N_{supply}) equals or exceeds the traffic demand (N_{demand}). A multiplicative safety factor is applied to N_{demand} to account for the fact that neither N_{supply} nor N_{demand} is known with certainty and to accommodate the desired level of design reliability. Thus, for a satisfactory mix, the following must be met:

$$N_{supply} \geq M \cdot D_{demand} \qquad (3.9)$$

where:

M = a multiplier whose value depends on the design reliability and on the variabilities of the estimates of N_{supply} and N_{demand}.

The reliability multiplier can be estimated from Table 3.11 or calculated from the following equation:

$$\mathrm{Ln}(M) = Z_R\,[\mathrm{Var}\{\mathrm{Ln}(N_{supply})\} + \mathrm{Var}\{\mathrm{Ln}\,(N_{demand})\}]^{0.5} \qquad (3.10)$$

where:

Z_R = a function of the reliability level which assumes values of 0.253, 0.841, 1.28, and 1.64 for reliability levels of 60, 80, 90, and 95 percent, respectively,
$\mathrm{Var}\{\mathrm{Ln}(N_{supply})\}$ = variance of the natural logarithm of N_{supply}, and
$\mathrm{Var}\{\mathrm{Ln}(N_{demand})\}$ = variance of the natural logarithm of N_{demand}.

For Level 1 analysis (surrogate testing and regression model), $\mathrm{Var}\{\mathrm{Ln}(N_{supply})\}$ depends upon the extent of extrapolation. Disregarding variability associated with the surrogate stiffness testing, $\mathrm{Var}\{\mathrm{Ln}(N_{supply})\}$ is approximated as follows:

Table 3.11. **Reliability multipliers**

Level of Analysis	Variance of $Ln(N_{supply})$	Variance of $Ln(N_{demand})$	Reliability Multiplier: 60-Percent Reliability (Z_R = 0.253)	80-Percent Reliability (Z_R = 0.841)	90-Percent Reliability (Z_R = 1.28)	95-Percent Reliability (Z_R = 1.64)
Level 1 (Surrogate Testing with Regression Model)	0.6	0.2	1.254	2.122	3.142	4.336
		0.4	1.288	2.319	3.597	5.155
		0.6	1.319	2.512	4.064	6.029
		1.0	1.377	2.897	5.048	7.960
	0.7	0.2	1.271	2.221	3.368	4.739
		0.4	1.304	2.416	3.829	5.585
		0.6	1.334	2.609	4.303	6.488
		1.0	1.391	2.994	5.306	8.485
Levels 2 and 3 (Fatigue Testing)	0.2	0.2	1.174	1.702	2.247	2.821
		0.4	1.216	1.918	2.695	3.562
		0.6	1.254	2.122	3.142	4.336
		1.0	1.319	2.512	4.064	6.029
	0.4	0.2	1.216	1.918	2.695	3.562
		0.4	1.254	2.122	3.142	4.336
		0.6	1.288	2.319	3.597	5.155
		1.0	1.349	2.705	4.547	6.962
	0.6	0.2	1.254	2.122	3.142	4.336
		0.4	1.288	2.319	3.597	5.155
		0.6	1.319	2.512	4.064	6.029
		1.0	1.377	2.897	5.048	7.960
	1.0	0.2	1.319	2.512	4.064	6.029
		0.4	1.349	2.705	4.547	6.962
		0.6	1.377	2.897	5.048	7.960
		1.0	1.430	3.285	6.112	10.169

Predicted N_{supply}	Variance of $Ln(N_{supply})$ Surrogate Model
1,000,000	0.651
3,500,000	0.666
10,000,000	0.684
35,000,000	0.709

For Levels 2 and 3 analysis (laboratory fatigue testing), $Var\{Ln(N_{supply})\}$, which depends on the nature of the testing program, can be determined from Equation 3.11 or estimated from Table 3.12.

$$Var\{Ln(N_{supply})\} = \sigma^2 \left[1 + \frac{1}{n} + \frac{(X-\bar{x})^2}{q \sum (x_p-\bar{x})^2} \right] \qquad (3.10)$$

where:

$VAR\{Ln(N_{supply})\}$ = variance of the extrapolated fatigue life (Ln),
σ^2 = variance of laboratory fatigue life (Ln) (use 0.1521 for A-003A testing procedures and equipment),
n = total number of test specimens,
X = strain (Ln) at which extrapolated fatigue life (Ln) is required,
$\bar{x}$ = average test strain (Ln),
q = number of specimens tested at each strain level, and
x_p = strain (Ln) at *p*th strain level.

For both fatigue testing and the surrogate regression model, the variance can also be estimated using the approximate equations of Table 3.13.

$Var\{Ln(N_{demand})\}$ is a function primarily of the accuracy of the traffic estimates and, as a consequence, will vary from agency to agency.

12. If Inadequate, Alter Trial Mix and/or Structural Section and Iterate

If a particular mix is judged to be inadequate for a specific application, several options are available to the designer including the following:

- Repeat the analysis with a less demanding level of design reliability.
- Reduce $Var\{Ln(N_{supply})\}$ by adding laboratory testing or by expanding its scope.
- Redesign the pavement structure to reduce tensile strain levels within the asphalt mix.
- Modify the mix design to improve its fatigue resistance.

Table 3.12. Variance of $Ln(N_{supply})$

Extrapolated Fatigue Life (N_{supply})	Number of Replicate Specimens					
	1	2	3	4	5	6
Laboratory Testing at Four Levels of Strain (Corresponding to 10,000, 35,000, 100,000, and 350,000 Load Cycles)						
1,000,000	0.367	0.260	0.224	0.206	0.195	0.188
1,500,000	0.421	0.287	0.242	0.219	0.206	0.197
2,000,000	0.464	0.308	0.256	0.230	0.214	0.204
3,000,000	0.531	0.342	0.278	0.247	0.228	0.215
4,000,000	0.583	0.368	0.296	0.260	0.238	0.223
6,000,000	0.662	0.407	0.322	0.280	0.254	0.237
8,000,000	0.723	0.438	0.342	0.295	0.266	0.247
12,000,000	0.817	0.483	0.373	0.318	0.285	0.262
16,000,000	0.884	0.518	0.396	0.335	0.298	0.274
24,000,000	0.988	0.570	0.431	0.361	0.319	0.291
32,000,000	1.067	0.609	0.457	0.381	0.335	0.304
Laboratory Testing at Two Levels of Strain (Corresponding to 10,000 and 350,000 Load Cycles)						
1,000,000	0.420	0.286	0.242	0.219	0.206	0.197
1,500,000	0.480	0.316	0.261	0.234	0.218	0.207
2,000,000	0.526	0.339	0.277	0.246	0.227	0.214
3,000,000	0.599	0.376	0.301	0.264	0.242	0.227
4,000,000	0.655	0.404	0.320	0.278	0.253	0.236
6,000,000	0.742	0.447	0.347	0.299	0.270	0.250
8,000,000	0.808	0.480	0.371	0.316	0.283	0.261
12,000,000	0.907	0.530	0.404	0.341	0.303	0.278
16,000,000	0.983	0.567	0.429	0.360	0.318	0.290
24,000,000	1.096	0.624	0.467	0.388	0.341	0.309
32,000,000	1.181	0.667	0.495	0.409	0.358	0.324

Table 3.13. Regression Equations for Computing Variance of $Ln(N_{supply})$

Estimation Procedure	Number of Replicate Specimens	$Var\{Ln(N_{supply})\}$
Laboratory Testing at Four Levels of Strain (Corresponding to 10,000, 35,000, 100,000, and 350,000 Load Cycles)	1	$0.005283\ (N_{supply})^{0.3085}$
	2	$0.008558\ (N_{supply})^{0.2472}$
	3	$0.01263\ (N_{supply})^{0.2076}$
	4	$0.01713\ (N_{supply})^{0.1792}$
	5	$0.02186\ (N_{supply})^{0.1575}$
	6	$0.02673\ (N_{supply})^{0.1402}$
Laboratory Testing at Two Levels of Strain (Corresponding to 10,000 and 350,000 Load Cycles)	1	$0.006903\ (N_{supply})^{0.2988}$
	2	$0.009653\ (N_{supply})^{0.2455}$
	3	$0.01345\ (N_{supply})^{0.2086}$
	4	$0.01764\ (N_{supply})^{0.1817}$
	5	$0.02213\ (N_{supply})^{0.1607}$
	6	$0.02643\ (N_{supply})^{0.1444}$
Regression Model		$0.4656\ (N_{supply})^{0.02401}$

3.8 General (Unabridged) Analysis System

The general analysis system (Level 3), used primarily for evaluation of mixes having binders of atypical temperature sensitivity, requires fatigue testing over a range of temperatures. The analysis process becomes quite complex as a result of the necessity to simulate the broad range of in situ temperature conditions. The accumulation of fatigue damage over the range of temperature levels is usually estimated using the linear-summation-of-cycle-ratios principle.

The recommended approach, which applies similar tools and is based on principles similar to those of those of more conventional approaches, is designed to produce mix-specific (and possibly site-specific as well) temperature equivalency factors which can subsequently be used in a single-temperature analysis. This approach is recommended so that both abridged and unabridged analysis systems are similar in structure and in application[6]. Although the development of specific temperature equivalency factors is a relatively detailed process (Deacon et al., 1993), it is no more complex than other approaches of comparable accuracy.

[6]Knowing the temperature equivalency factors for new or unconventional mixes may also eliminate the need for comprehensive analysis when their use is being evaluated for other, subsequent applications.

Once the temperature equivalency factors have been developed, the process of mix evaluation parallels that of Level 2 analysis.

To support the general analysis system, fatigue testing is generally recommended at four temperature levels [10°, 15°, 20°, and 25°C (50°, 59°, 68°, and 77°F)]. Much of the expected in situ damage occurs within this temperature range, and laboratory testing is facilitated by avoiding more extreme conditions. It may sometimes be desirable to test a larger number of specimens at 20°C (68°F)—the temperature at which the basic analysis is performed—to reduce the variability of the estimated fatigue life.

3.9 Summary

An improved procedure has been developed for defining the fatigue resistance of asphalt- and/or binder-aggregate mixes. This procedure permits the determination of the fatigue response of a mix in 24 hours at one temperature level with a reliability of at least 80 percent. Also provided, based on extensive fatigue testing, is a surrogate model utilizing the results of stiffness measurements on the mix.

The results of the program have been incorporated in an innovative design and analysis system for evaluating the fatigue resistance of asphalt-aggregate mixes. This system provides an effective mechanism for the interpretation of laboratory fatigue measurements and for determining the impact of asphalt-aggregate interactions on expected pavement performance. It combines mix testing with traffic loading (repetitions, wheel loads, and tire pressures), environmental conditions (temperature), and the pavement cross-section to assure that, with preselected reliability, fatigue cracking in the asphalt-bound layer will not exceed acceptable limits.

The analysis system assumes that a trial mix has been identified, that traffic and environmental conditions have been determined, and that the pavement cross section has been designed. It then seeks to judge, with predetermined reliability, whether the trial mix would perform satisfactorily in service. If it would not, the designer can opt for redesigning the mix, strengthening the pavement section, or repeating the analysis using more refined measurements and/or estimates.

For routine mix designs, the testing and analysis system has been simplified to the maximum possible extent. Laboratory testing is limited to stiffness measurements, and the primary analysis requires only a single estimate of in situ strains using traditional assumptions of linear elasticity. Unconventional mixes or uncommon applications, on the other hand, require more extensive testing and analysis for reliable decision making. Multiple-temperature fatigue testing must be performed, and analysis must address the complex thermal environment anticipated in situ.

Key features of the mix analysis system include the use of temperature conversion factors and quantitative reliability concepts. Temperature conversion factors—used to convert design ESALs to their equivalent at a common reference temperature of 20°C (68°F)—have been found to be an effective and simple way of treating environmental temperature effects and of reducing the necessity for extensive multiple temperature testing. Reliability concepts provide a quantitative means for comparatively judging the adequacy of surrogate testing-regression models vis-à-vis laboratory fatigue testing: they thus permit and encourage a hierarchical approach to mix design which routinely simplifies the process yet permits detailed analysis where necessary.

Conceptual development of the mix analysis system has been completed as part of SHRP Project A-003A, and considerable progress has been made toward establishing a readily implementable package for use by material engineers nationwide. In addition to completing the calibration process, one of the key remaining tasks is to validate the analysis system by demonstrating its ability to reliably discriminate among suitable and unsuitable mixes.

4

Accelerated Performance-Related Tests For Permanent Deformation of Asphalt-Aggregate Mixes

Development of the accelerated performance-related tests (APTs) for permanent deformation (rutting) evaluation consisted of a number of phases: 1) review of the state-of-knowledge in the permanent deformation area including the identification of candidate tests to measure the propensity of a mix to permanent deformation and which can be used for performance (rutting) prediction; 2) formulation of working hypotheses for permanent deformation development in asphalt-aggregate mixes and pavement structures based on the state-of-knowledge evaluation; 3) evaluation of a limited number of mixes to define key mix parameters and the suitability of the candidate tests to measure all of these parameters as well as their ability to predict permanent deformation (N.B., the evaluation process led to the conclusion that none of tests which had been initially selected for evaluation were suitable and that a new test or set of tests should be developed); 4) development of a suitable laboratory test or tests to define permanent deformation response guided by analytical developments for a constitutive relationship which could be used to predict mix response in representative pavement structures; 5) performance of a test program with the selected tests to develop a database of mix properties for use in the constitutive relations to permit the estimation of rutting propensity in representative pavement structures; and 6) validation of the approach by means of wheel-tracking tests. Also included is a description of how the results of the permanent deformation test program can be used in a mix design and analysis system.

4.1 Literature Evaluation and Hypotheses

From an extensive literature review and evaluation, SHRP-A/IR-91-104 (Sousa et al., 1991a) was prepared. The following summarizes the key findings and recommendations from that study.

1. Rutting is caused by a combination of densification (decrease in volume and, hence, increase in density) and shear deformation; however, shear deformation

rather than densification is considered to be the primary cause of rutting in properly constructed pavements.

2. Mix characteristics and test or field conditions that have important effects on rutting of asphalt concrete pavements include the following:

   - Dense aggregate gradations mitigate the effects of rutting in asphalt concrete layers.

   - An aggregate with a rough surface texture and angular shape tends to mitigate the occurrence of permanent deformation.

   - Lower viscosity asphalts make the mix less stiff and, therefore, more susceptible to rutting; harder (more viscous) asphalt should be used in thicker pavements in hotter climates.

   - Laboratory specimens should be compacted to densities comparable to those reached in situ as a result of both construction and subsequent traffic loading.

   - When compacted in the field with heavy rollers, mixes of relatively poor workability can be effective in minimizing rutting propensity. Such mixes have a stable arrangement of the mineral skeleton and, thereby, large internal friction. (To an extent, poor workability derives from other factors including rough, angular, and densely graded aggregate; stiff asphalt; and low asphalt content.)

   - Temperature has a significant effect on rutting.

   - Large proportions of heavy trucks will likely increase the rate of rutting.

   - Higher tire inflation pressures will likely increase the amount of rutting.

3. Both layer-strain and viscoelastic methods are presently used for predicting permanent deformations in pavements.

4. Procedures for rutting prediction require that suitable techniques be developed not only for calculating the response of the pavement to load but also for realistically characterizing material properties. The overall objective of materials testing should be to reproduce as closely as practical in situ pavement conditions including the general stress state, temperature, moisture, and general conditions of the affected materials.

5. All proposed methods for estimating rutting require field and test-track validation. A complete mechanistic validation must assure that the profile of

permanent strain be accurately estimated. Test sections on existing roads have been used to study pavement response: they are advantageous because they capture the effects of real traffic patterns and real environmental conditions.

6. Comparisons of predicted permanent-strain profiles with measurements in test tracks indicate that, in many instances, layer-strain analysis overestimates permanent strains in the tensile zone. Test-track results suggest that the permanent strains are quite small near the bottom of thick bituminous layers. The above indicates the apparent discrepancy between the actual distribution of permanent strain with depth, and the theoretically calculated distribution.

7. Although many researchers recognize that shear stress is the primary mechanism of rutting, suitable laboratory test methods and theoretical models are not yet available for properly treating shear-induced permanent deformation.

8. Current analytically-based models predict only the rut depth under the center line of loading and do not consider the contribution to rutting caused by the development of shear stresses at the edges of the tires.

The findings suggested that significant new developments would be required before sufficiently reliable test procedures, analytical models, and design systems could be made available. They provided the basis for development of a theoretically sound analysis procedure and related test methodology to permit more reliable mix designs and more appropriate modeling of the real conditions under which rutting occurs. Thus the following guided the test selection program:

1. A prediction model should be developed that directly accounts for the shear stresses developed within the entire zone of permanent deformation, extending outward from the centerline of loading to at least the tire edges. This can be achieved using a finite element technique.

2. The states of stress under which permanent deformation characteristics of materials are obtained in the laboratory should be extended to the entire zone of permanent deformation. Laboratory tests should duplicate the states of stress that are encountered within the entire rutting zone, in particular where the shear stress is greater than the normal stress. Accordingly, the rutting propensity of mixes should be measured using equipment capable of directly applying shear stresses.

3. The development of a generalized permanent deformation law for asphalt concrete should continue. This law should consider the effects of temperature as well as states of stress and/or strain encountered in sections of the pavement away from the centerline of loading.

4. To validate the test methods and analytical system, laboratory-scale wheel-track devices should be developed and full-scale field tests should be conducted

on pavements composed of different structures and subjected to different patterns and characteristics of traffic (e.g. tire pressures). Emphasis should be placed on measuring the states of stress and strain in the asphalt-concrete layer including shear deformations.

4.2 Test Method Selection

In the test method selection process three separate agencies with laboratory test capabilities were utilized including the University of California at Berkeley (UCB), North Carolina State University (NCSU), and SWK Pavement Engineering/University of Nottingham (SWK/UN).

The UCB program included a series of tests conducted in the axial and shear loading modes in creep and repeated loading. These results were used to lay the groundwork for the development of a permanent deformation constitutive relationship and an analysis procedure to predict permanent deformation under repetitive traffic loading. It was planned to compare this mix testing and analysis system with the VESYS procedure, an alternative system that incorporates a unique test series and utilizes linear viscoelastic modeling. This program was conducted at NCSU. The results from wheel-track tests conducted by SWK/UN were planned to be used to evaluate the results of the UCB and NCSU programs.

As the test program evolved at UCB, a series of important mix response characteristics were identified which were judged to be of practical significance. These characteristics included the following:

- dilation under shear loading;
- increase in stiffness with increase in hydrostatic pressure;
- negligible volumetric creep;
- residual permanent deformation on removal of load;
- temperature and rate of loading dependence; and
- difference in response in creep and repeated loading.

Equipment development to reflect these characteristics progressed at UCB so that by the time the test method selection phase was completed, equipment for the next phase was developed and ready for use. Results of the test program at NCSU reinforced this effort since the parameters developed from the VESYS type test (α and μ) were not sensitive to mix parameters.

The resulting equipment is illustrated schematically in Figure 4.1. It permits simultaneous application of shear and axial loads to cylindrical specimens 15 cm or 20 cm (6 in. or 8 in.) in diameter and 5 cm to 9 cm (2.0 in. to 3.5 in.) in height respectively. Confining pressures of 100 psi (690 kPa) can be used over a temperature range of -10° to +70°C (14° to 158°F). Both shear and axial loads can be applied sinusoidally, repetitively, or sustained (creep loading). For sinusoidal loading, frequencies in the range 20 to 0.01 Hz (approximately three decades) are feasible. Repeated loads using a haversine pattern can be

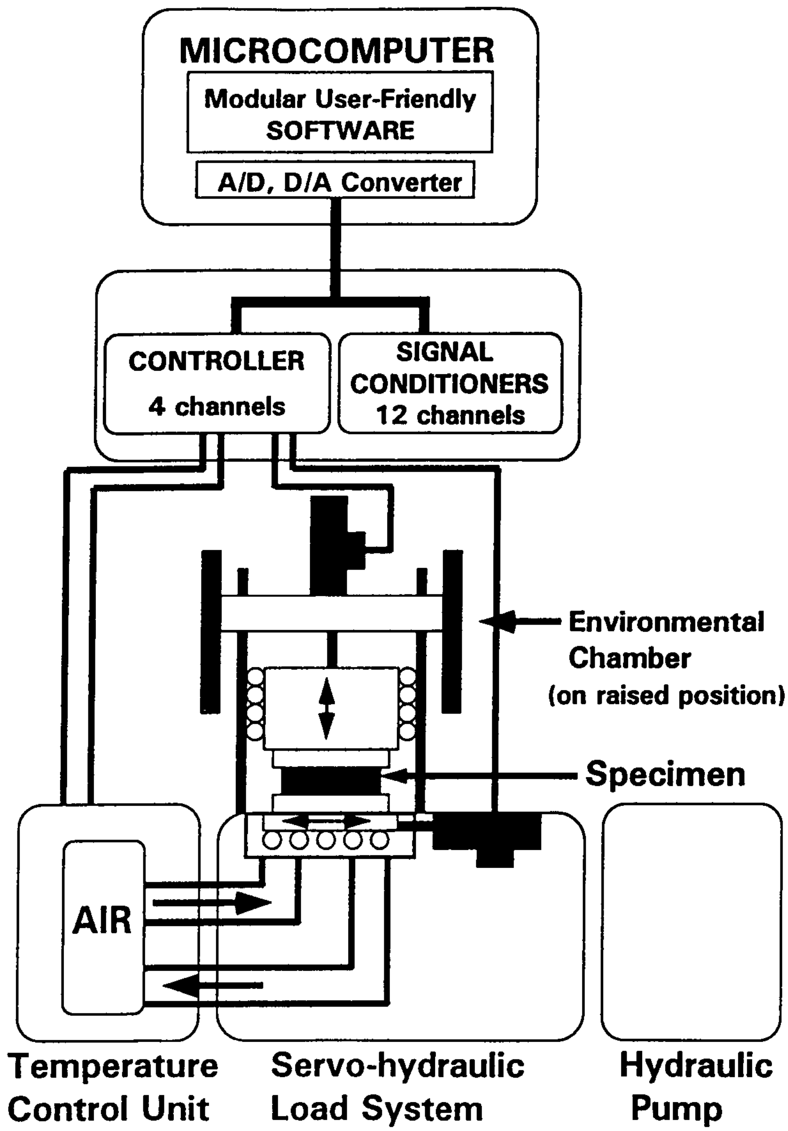


Figure 4.1. Permanent deformation test equipment

applied with a range in times of loading and rest periods. Shear stresses are transmitted to the specimen through end caps which have been bonded to it using an epoxy resin. Shear, axial, and radial deformations are measured using LVDTs in direct contact with the test specimen.

Because the shear stresses are applied only to the end surfaces and not to the sides of the cylindrical specimens, the lack of complementary shear stresses on the vertical faces results in stresses and deformations which could confound the material response. Results of finite element analyses indicate that for the specimen sizes utilized, these influences are relatively small (Sousa et al., 1993).

Test methodologies have been developed which permit the definition of the important characteristics of a mix necessary to define its propensity for permanent deformation including 1) dilation under shear loading, 2) increase in stiffness with increase in hydrostatic pressure at higher temperatures, 3) temperature and rate of loading dependence, and 4) residual permanent-deformation development with unloading.

While one might argue that many of these parameters could be evaluated in a conventional axial loading test, the ability to independently control the shear and axial stresses and to have a direct measure of the dilation characteristics of a mix (i.e., the normal force generated as the specimen tends to dilate under shear stress application can be directly measured) favor this type of test.

Specimen size and configuration also influenced the decision to select the shear test. For conventional mixes [nominal 25 mm (1 in.) maximum size aggregate] it is desirable to test specimens which are at least 100 mm (4 in.) and preferably 150 mm (6 in.) in diameter. With a 150 mm (6 in.) diameter specimen, an axial loading test would require a specimen height of at least 150 mm (6 in.) (with polished end surfaces) and preferably 304 mm (12 in.) in height, i.e. a height to diameter ratio of 2 to 1 to minimize end effects on material response. On the other hand, a 150 mm (6 in.) diameter specimen for the simple shear test could have a height in the range of 50 mm to 76 mm (2 in. to 3 in.) and provide reasonable results based on finite element analyses of the test configuration. This configuration lends itself to testing pavement (field) cores as well.

With large stone mixes, specimens of the order of 200 mm (8 in.) in diameter are desirable. Considering the requirements stated above, this makes the simple shear test even more appealing from a standpoint of specimen size.

It was also concluded that none of the existing analysis procedures for pavement response to load would capture the type of behavior described above. Thus it was deemed necessary to develop a constitutive relationship for asphalt-aggregate mixes reflecting the above-noted characteristics and which would be compatible with a three-dimensional finite element representation of typical pavement structures permitting, in turn, the estimation of the accumulation of permanent deformation under repetitive traffic loading. At the same time, it was also necessary to develop procedures along with test equipment which define the parameters contributing to the permanent deformation response of asphalt-aggregate mixes.

4.3 Analytical and Test Developments

The model developed to incorporate the characteristics of materials response listed in the previous section is nonlinear viscoelastic. This model is shown schematically in one-dimensional form in Figure 4.2. It must be emphasized, however, that the model incorporates the coupling of volumetric and deviatoric behavior to provide a constitutive relationship which is three-dimensional.

The *nonlinear elastic response characteristics* (related to modulus E_o) are determined by the following:

1. **simple shear constant height test** — measurements of stress and deformation under fast application of a shear stress while maintaining the specimen height constant permits determination of the initial shear stiffness and a measure of the dilation characteristics. Dilation is strain-related and is associated with the granular structure of dense mixes. Its influence can normally be neglected at low temperature because the asphalt-concrete mix is so stiff that conventional traffic loads are not of sufficient magnitude to generate strains large enough to mobilize the dilational component. At higher temperatures, however, strains sufficient to permit dilation become an important aspect in determining permanent deformation response.

2. **uniaxial strain test** — rapid application of an axial stress while maintaining the specimen perimeter constant provides additional information on the nonlinear elastic response.

3. **volumetric test** — rapid application of a hydrostatic stress permits determination of a measure of bulk modulus from measurements of the hydrostatic stress, and radial and axial strains.

Complex modulus determinations, over a range in temperatures from 4° to 60°C (39° to 140°F) and at frequencies ranging from 0.02 Hz to 10 Hz, using sinusoidally applied shear stresses and resulting in small strains (approximately 0.1 percent), permit determination of the *rate and temperature dependencies* of the mix. Results of these determinations permit determination of the values of E_n and η_n (from $n=1$ to n).

Assumption of *thermo-rheologically simple response,* as seen earlier, appears reasonable for small deformation. *Plasticity* is associated with a yield stress and an associative flow rule and both isotropic and kinematic hardening (which are subjected to the Kuhn-Tucker complementary conditions). The plastic-deformation response characteristics (represented by the slider in Figure 4.2) are determined from the simple shear, constant height test since the test is performed at three stress levels and the recovery of deformation is observed after each stress application for a period of time sufficient to permit equilibrium to be obtained.

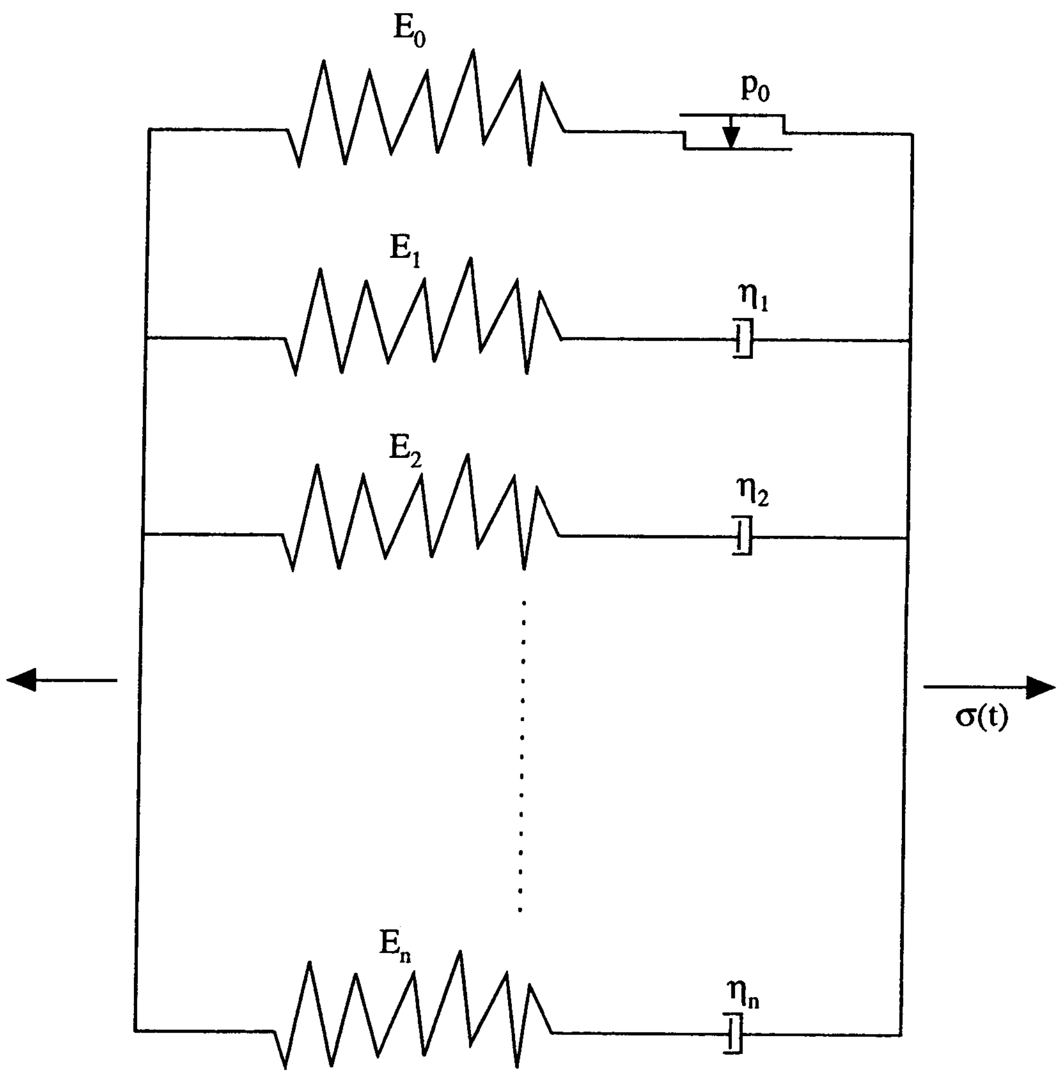


Figure 4.2. Schematic representation of nonlinear viscoelastic model (with slider)

While damage (rapid increase in strain beyond some threshold value) has been considered, it has not been incorporated in the analytical solutions developed in this investigation. Provision has been made to allow the dashpots to change as a function of either strain level or strain rate magnitude.

The constitutive relationship described above can be used to predict the development of permanent deformation in a mix in a specific pavement structure, using a finite element idealization such as that shown in Figure 4.3.

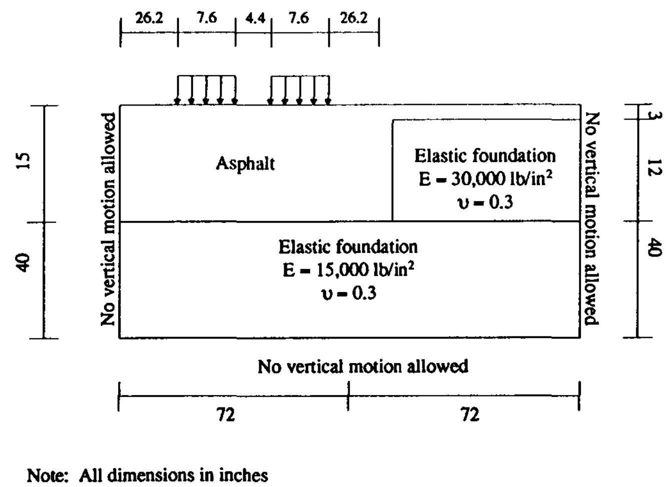


Figure 4.3. Pavement structure used to estimate permanent deformations and maximum permanent shear strains (dimensions are in inches, 1 in. = 2.54 cm)

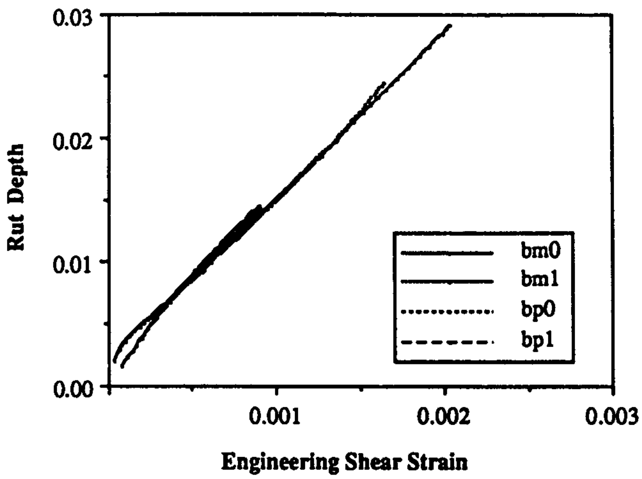
Analyses have been performed for 16 mixes (4 MRL asphalts and aggregate RD and RH) tested according to procedures described earlier. Relationships between maximum permanent shear strain and rut depth have been developed for those 16 mixes, for the pavement structure shown in Figure 4.3 (Sousa et al., 1993). The resulting relationships for the mixes containing each binder are shown in Figure 4.4. These results suggest that the following relationship is appropriate:

$$\text{Rut depth} = 10 \text{ or } 11\ (\gamma_p)_{max} \tag{4.1}$$

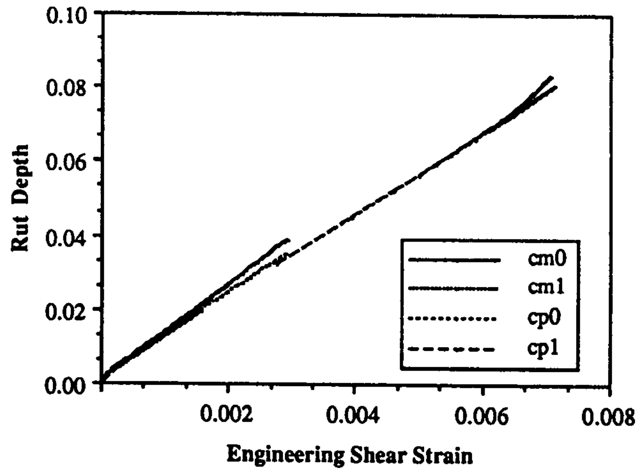
where: $(\gamma_p)_{max}$ = maximum permanent shear strain

The rut depth is expressed in inches in Equation 4.1. If rutting in millimeters is desired, the coefficient of the equation is about 275.

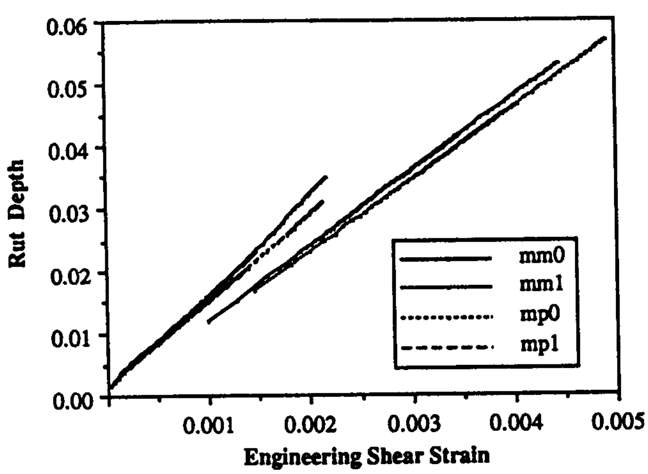
While the relationships shown in Figure 4.1 were obtained for a tire pressure of 690 kPa (100 psi), preliminary computations suggest that Equation 4.1 is independent of contact pressure. These computations were performed on a similar pavement structure with contact stresses of 1380 kPa and 3450 kPa (200 psi and 500 psi).



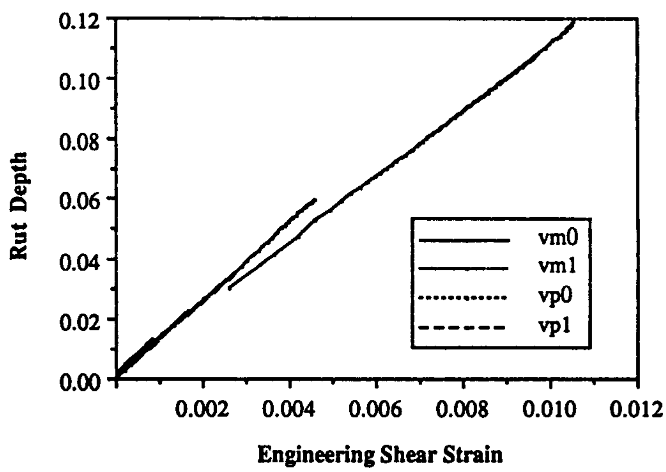
a. Asphalt AAK, aggregates RD(m) and RH(p), low(0) and high(1) void contents



b. Asphalt AAC, aggregates RD(m) and RH(p), low(0) and high(1) void contents



c. Asphalt AAM, aggregates RD(m) and RH(p), low(0) and high(1) void contents



d. Asphalt AAG, aggregates RD(m) and RH(p), low(0) and high(1) void contents

Figure 4.4. Relationships between maximum permanent shear strain (engineering shear strain) and rut depth obtained for pavement section of Figure 4.3

It must be emphasized that this relationship is dependent on the thickness of the asphalt layer. In the example presented herein an asphalt-concrete layer of 381 mm (15 in.) was used. As the layer thickness decreases, the coefficient of Equation 4.1 will decrease, e.g. for asphalt-concrete layers used as overlays on portland cement concrete pavements in the range of 100 mm to 150 mm (4 in. to 6 in.) (Weissman 1993).

The resulting test methodologies using the simple shear test equipment have been divided into levels which are associated with two recommended levels of mix design and analysis. These are described in the next section.

4.4 Mix Design and Analysis

The analysis system proposed herein is based on the premises that a trial mix has been identified, that traffic and environmental conditions have been determined, and that the pavement structural (cross) section has been designed. The analysis system permits determination of whether the trial mix will perform satisfactorily in service. Two levels are considered: 1) Level 1 — an estimate is made of the number of repetitions which the mix can sustain to a fixed level of rutting, or 2) Level 2 — a rut depth in the asphalt-bound layer is estimated for the prescribed conditions and compared to the tolerable level established for the site.

If the mix does not meet the requirements, repeating the analysis using more refined measurements or redesigning the mix are two alternatives available to the engineer. Mix redesign could include 1) increasing the amount of crushed (rough textured) aggregate in the mix, 2) using a more viscous grade of asphalt, or 3) using a modified binder rather than a conventional asphalt cement. The recommended approach can be briefly summarized as follows:

1. Determine design requirements for reliability and performance (permissible rut depth).

2. Determine the expected distribution of in situ pavement temperatures.

3. Estimate design traffic demand (ESALs).

4. Select trial pavement structural section.

5. Select trial mix.

6. Prepare test specimens and condition as required.

7. Determine the resistance of the trial mix to permanent deformation using the cyclic shear test or the suite of tests required for rutting prediction.

8. Apply factors to the traffic demand to account for differences between laboratory and in situ conditions and, as appropriate, to convert traffic loading to its equivalent at the laboratory test temperature.

9. As appropriate, determine the amount of rutting associated with N_{demand}.

10. If N_{demand} exceeds N_{supply}, or the rut depth corresponding to N_{demand} exceeds the permissible rut depth, redesign the mix. (N.B., it may be possible to improve the reliability of the traffic estimate to a specific level of rutting by testing more specimens, or to determine a more reliable estimate of the amount of rutting for the specific traffic conditions by using the suite of tests.)

The procedure can also be used for mix-design purposes, that is, for selection of binder content. Within this framework, mixes are tested over a range in binder contents and the design binder content would be the maximum amount which could be used so that the estimated rut depth will not exceed the level selected for the anticipated traffic.

Levels of Analysis. The proposed system recognizes that a range in testing requirements is desirable. For routine applications the testing need not be extensive. However, when unconventional mixes are utilized or more complex design applications are encountered, then the extent of testing and analysis is increased.

Two levels have been stipulated. Level 1 requires repeated simple shear testing at constant height, at a single stress condition and a single temperature, and uses previously developed analyses (e.g. like those shown in Figure 4.4) to insure that the level of rutting will not exceed some prescribed value. Level 2, the more comprehensive, requires a suite of tests using the simple shear test equipment, including shear stiffness tests performed at multiple temperatures.

Traffic Loading and Temperature Considerations. If the traffic is expressed in terms of ESALs per lane for structural pavement design purposes, this same measure can be used for mix-design purposes as well. Development of permanent deformation in mixes is significantly affected by the magnitude of the tire contact pressure. Accordingly, an estimate of the range in tire pressures associated with the traffic must be known so that a representative tire contact pressure can be utilized to establish the conditions for mix evaluation.

For routine mix designs, shear testing at a single temperature, termed the critical temperature, T_c, is recommended. Conversion of the design traffic level (expressed in terms of ESALs) to its equivalent at the critical temperature is required. Predetermined temperature factors (by climatic region) will likely suffice for this situation.

The critical temperature is the temperature at highly stressed locations within the pavement structure; it has been defined as the temperature for a prescribed temporal distribution of traffic[7] at which more rutting occurs than at other temperatures. For rutting, the highly stressed locations occur in the upper portion of the asphalt- and/or binder-bound layer, i.e. in the upper 8 cm to 10 cm (3 in. to 4 in.) near the surface. More rutting occurs at the critical temperature because of both the frequency of its occurrence relative to traffic and the sensitivity of the mix to rutting at this temperature.

Reliability. The analysis system for Level 1 requires that the mix resistance to permanent deformation, termed N_{supply} (associated with a prescribed limiting rut depth), equal or exceeds the traffic demand, N_{demand} (adjusted traffic estimate), which has been increased by an amount determined by the designer on the basis of a pre-selected level of reliability. The value of N_{demand} is increased by a reliability multiplier (M), the value of which increases with increasing reliability selected for the design and with increasing variability of mix response and traffic demand. Although reliability remains an important design consideration for Level 2 analysis, specific recommendations for its treatment in such analyses have not yet been developed.

Mechanistic Analysis. In the procedure used for the Level 1 analysis, the amount of rutting has been related to the maximum shear strain occurring in the upper part of the pavement layer; both parameters have been determined by a finite element analysis of a representative pavement structure, using conventional asphalt-aggregate mixes and selected values of tire contact pressure. Level 2 makes direct use of the methodology of finite element analysis, and the measurement of mix characteristics, using the suite of tests to *predict* the amount of rutting for the site-specific traffic and environmental conditions.

4.4.1 Overview of Analysis Systems

Distinguishing characteristics of the permanent-deformation analysis system are shown in Table 4.1. Simplified testing with the cyclic shear test distinguishes Level 1 from Level 2[8], which requires the complete characterization of mix response using the suite of tests. Figures 4.5 and 4.6 provide schematic frameworks for both systems.

Level 1 is expected to suffice for mixes of typical temperature sensitivity. Level 2 provides an optional procedure for investigative analyses and for calibrating models used in Level 1 analysis. Table 4.2 summarizes the recommended level of permanent-deformation testing and analysis for different types of mixes.

For most permanent deformation analyses (Level 1), the design traffic is expressed in terms of the number of ESALs (AASHTO) in the critical lane during the design life, adjusted to its equivalent at the critical temperature, T_c. A shift factor must be applied to the traffic

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[7]A uniform traffic distribution has been assumed through the day and the year for the information presented herein.

[8]While three levels have been shown for the other design systems, only two are recommended for permanent deformation. Essentially Level 1 and Level 3 are the same.

Table 4.1 Distinguishing characteristics of permanent deformation analysis system

Variables		Level 1	Level 2
		Abbreviated analysis with limited cyclic shear testing	Comprehensive analysis with full testing
Testing	Type	Cyclic shear	Constant height simple, shear, uniaxial strain, volumetric, shear frequency sweep
	Temperature	Critical temperature, T_c	40°C with frequency sweeps at 4°, 20°, 40°, and 60°C
In Situ Conditions	Traffic	Equivalent ESALs at T_c, 85th percentile tire pressure	ESALs by temperature class, 85th percentile tire pressure
	Structure	Critical shear stress under "standard" load at T_c	Complete stress and/or strain pattern from finite element analysis
	Temperature	Frequency distribution at 5-cm (2-in.) depth	Frequency distribution throughout surface layer
Analysis	Mechanistic	Finite element analysis with nonlinear viscoelastic surface properties[a]	Finite element analysis with nonlinear viscoelastic surface properties
	Damage	Preanalysis (temperature equivalency factors for design ESALs)	Integral part of finite element analysis

[a]It is possible that sufficiently accurate results for shear stress may be determined using multi-layer elastic analysis as experience is developed.

estimate and laboratory measurements. The shift factor attempts to account for differences in stress states, loading conditions, traffic wander, etc. The end result of the traffic analysis is an estimate of traffic demand (N_{demand}) that is commensurate with laboratory permanent-deformation measurements.

Mix resistance to permanent-deformation distress (N_{supply}) is determined from measurements with the simple shear equipment using the cyclic shear test for Level 1. For Level 2, the amount of rutting associated with N_{demand} is calculated based on the results of the suite of tests and compared with the tolerable level selected for the rut depth for the specific pavement site.

4.4.2 Temperature Equivalency Factors

To ensure productivity, simplify and reduce costs, permanent deformation testing for the Level 1 analysis is limited to a single temperature. The destructive effects of anticipated traffic in the field are expressed as equivalent ESALs at the single temperature.

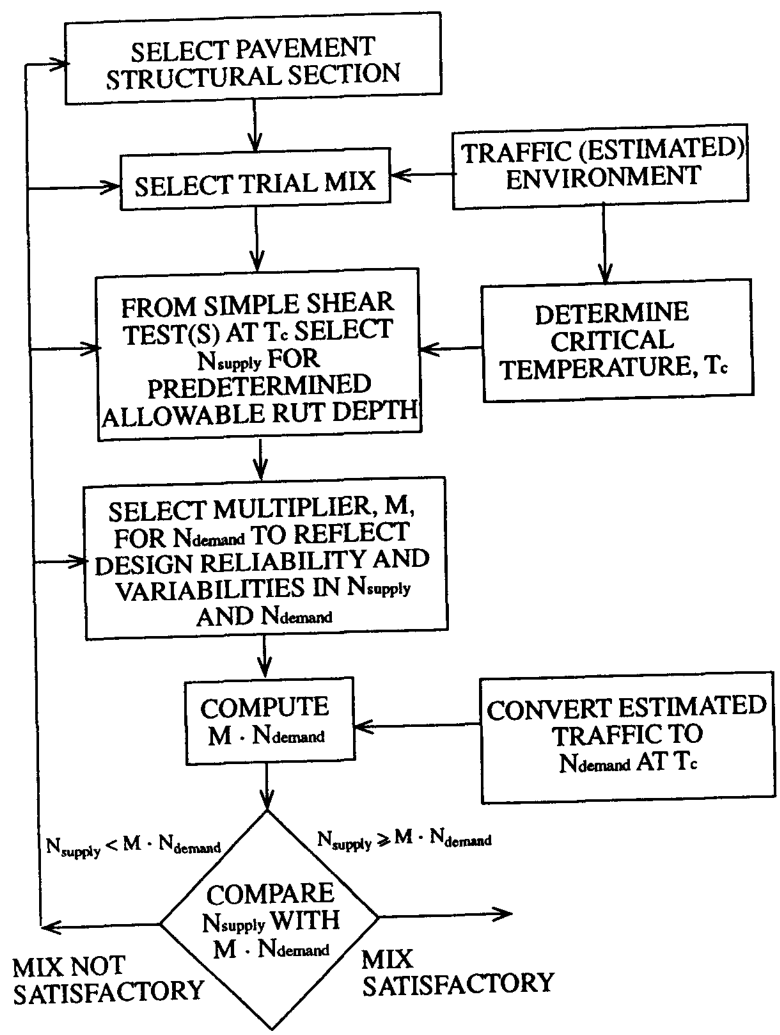

Figure 4.5. Routine mix design and analysis system

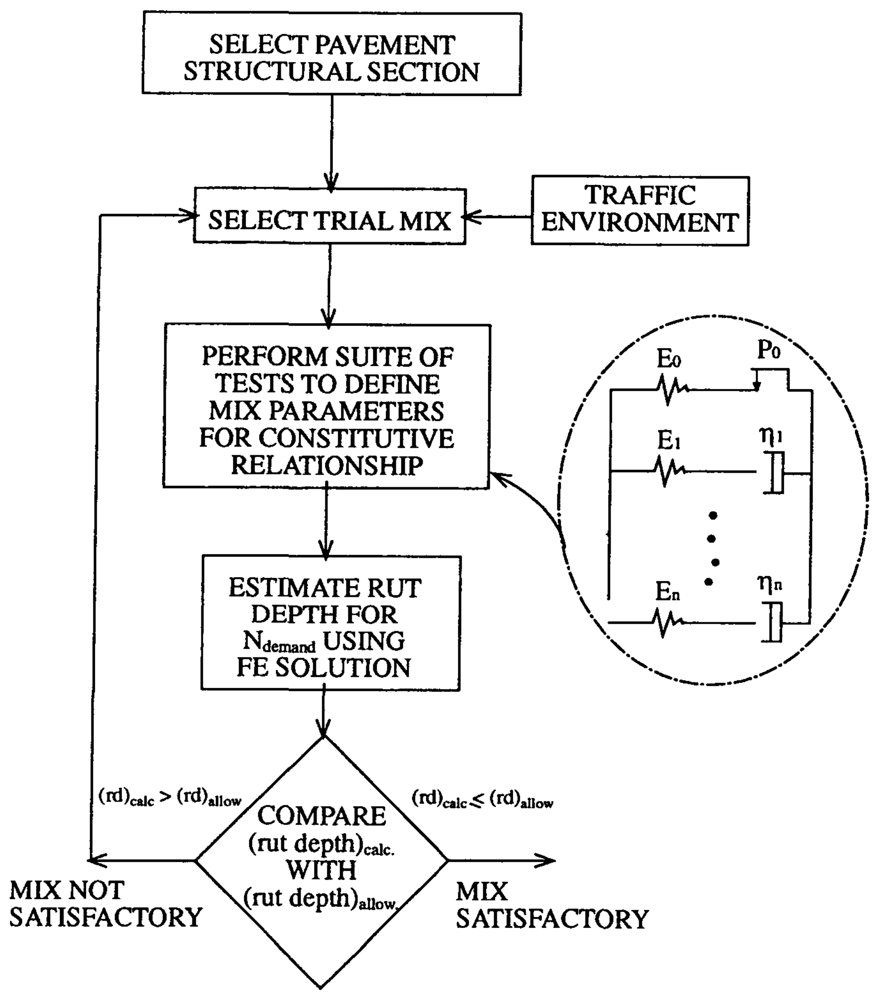

Figure 4.6. Comprehensive mix design and analysis system

Table 4.2. Recommended levels of permanent deformation testing and analysis

Mix Characteristics	Level 1	Level 2
	Abbreviated analysis with limited cyclic shear testing	Comprehensive analysis with full testing
Dense graded mixes with conventional binders of typical temperature sensitivity	Recommended	Optional for investigative analyses and model calibrations
Unconventional mixes with binders of typical temperature sensitivity	Recommended	Optional for investigative analyses and model calibrations
Mixes with binders of atypical temperature sensitivity	Not applicable	Optional for investigative analyses and model calibrations

The temperature equivalency factors (TEF) build upon the AASHTO load-equivalency concept. The temperature-equivalency factor, TEF_i, is defined as the number of ESALs at the common temperature, T_c, that is equivalent in destructive effect to one ESAL applied at some other temperature, T_i. If $ESAL_i$ represents the number of ESALs anticipated when the temperature is T_i, then the product, $ESAL_i \times TEF_i$, would represent the equivalent effect of the loading at the common temperature, T_c. Therefore:

$$TEF_i \times ESAL_i = \text{Equivalent } ESAL_c \tag{4.2}$$

alternatively,

$$TEF_i = [\text{Damage of } ESAL_c \text{ at } T_c]/[\text{Damage of } ESAL_i \text{ at } T_i] \tag{4.3}$$

These factors have been developed for the nine climatic regions throughout the United States incorporated in the FHWA program (Lytton et al., 1990) to determine environmental parameters for use in pavement analyses. Table 4.3 contains a listing of these factors for Region IIIB (Southwestern United States).

Temperature-conversion factors, which convert repetitions of traffic (ESALs) to the equivalent number at the critical temperature are shown in Table 4.4. Their use is recommended in lieu of computations of combined frequency-and-temperature-equivalency factors because they yield identical results and are easier to apply.

Table 4.3. Temperature equivalency factors as a function of reference temperature, Region IIIB

Mid-range Temp. at 5 cm (2 in.) (°C)	Frequency (Percent)	Reference Temperature (°C)				
		39.0	41.0	43.0	45.0	47.0
25.0	3.42	0.0378017	0.0257838	0.0141293	0.0067692	0.0059000
26.0	4.22	0.0543119	0.0370450	0.0203003	0.0097257	0.0084769
27.0	3.61	0.0698432	0.0475385	0.0261055	0.0125069	0.0109009
28.0	4.59	0.0971893	0.0662907	0.0363268	0.0174038	0.0151690
29.0	3.11	0.1046651	0.0713898	0.0391210	0.0187425	0.0163358
30.0	3.45	0.1484723	0.1012698	0.0554950	0.0265871	0.0231731
31.0	2.49	0.1649630	0.1125177	0.0616587	0.0295401	0.0257469
32.0	2.85	0.2221129	0.1514984	0.0830198	0.0397740	0.0346667
33.0	2.69	0.2814753	0.1919883	0.1052079	0.0504041	0.0439319
34.0	2.92	0.3146476	0.2146144	0.1176068	0.0563443	0.0491093
35.0	2.51	0.5258816	0.3586925	0.1965604	0.0941702	0.0820781
36.0	1.37	0.4307887	0.2938317	0.1610172	0.0771418	0.0672362
37.0	1.48	0.5729328	0.3907851	0.2141469	0.1025957	0.0894217
38.0	1.74	0.7515400	0.5126093	0.2809055	0.1345791	0.1172982
39.0	1.92	1.0000000	0.6820784	0.3737732	0.1790711	0.1560771
40.0	2.79	1.4395924	0.9819150	0.5380810	0.2577894	0.2246874
41.0	0.98	1.4661070	1.0000000	0.5479915	0.2625374	0.2288257
42.0	1.21	1.9534844	1.3324296	0.7301601	0.3498126	0.3048942
43.0	1.30	2.6754194	1.8248459	1.0000000	0.4790903	0.4175717
44.0	2.01	3.1738901	2.1648420	1.1863150	0.5683520	0.4953715
45.0	1.62	5.5843740	3.8089811	2.0872892	1.0000000	0.8715928
46.0	1.14	5.6062218	3.8238830	2.0954553	1.0039123	0.8750028
47.0	0.21	6.4070903	4.3701381	2.3947984	1.2473247	1.0000000

°C = 5/9 (°F−32)

Table 4.4 also indicates the percentage of rutting expected to occur within 2°C of the critical temperature. Selecting a test temperature which corresponds to the largest possible percentage is desirable in order to minimize the error that may occur when mix temperature sensitivity is atypical.

4.4.3 Reliability

Considerations of reliability offer the potential for assuming an acceptable level of risk in mix design and analysis without expensive over-design. As used herein, reliability refers to the probability that the mix will provide satisfactory performance during the design period; i.e. the amount of rutting will not exceed some prescribed value such as 1 cm (0.5 in.). Reliability levels can be specified in the range of 60 percent to 95 percent corresponding to risk levels of 40 percent to 5 percent respectively. Generally a lower level of risk means a higher mix cost or a reduction in the number of acceptable mixes available (including both binders and aggregates).

Table 4.4. **Temperature factors — permanent deformation**

Region	Critical Temperature (°C)	Temperature Conversion Factor (to T_c)	Percent Rutting with 2°C of Critical Temperature
I-A	36	0.1025	55.1
I-B	41	0.0809	53.4
I-C	36	0.1082	56.0
II-A	38	0.0604	46.1
II-B	42	0.0869	57.2
II-C	44	0.0897	56.2
III-A	38	0.0668	49.4
III-B	45	0.0812	58.7
III-C	42	0.1317	58.7

°C = 5/9 (°F−32)

This analysis system requires that the mix resistance (N_{supply}) exceed the traffic demand (N_{demand}) by some factor which is determined based on reliability requirements. This condition can be expressed in the following form:

$$N_{supply} \geq M \cdot N_{demand} \tag{4.4}$$

where: M = a reliability multiplier (greater than 1) whose magnitude is dependent upon the variabilities of the estimated number of repetitions in the laboratory associated with a certain level of rutting in the pavement and the estimated traffic, and is also dependent upon the desired reliability of the design.

Equation 4.4 can also be written in logarithmic form:

$$\text{Ln}\,(N_{supply}) \geq \text{Ln}\,(N_{demand}) + \delta \tag{4.5}$$

where: δ = an increment (greater than 0) whose value is equal to Ln(M).

Limited data from a series of constant height, repeated load, simple shear tests yielded a mean square error of 0.602 for a range in binder contents. This permitted a determination of the standard deviation of Ln(N_{supply}) as shown in Table 4.5. Using these determinations, it was then possible to determine the reliability multiplier, M. The results are shown in Table 4.6.

Table 4.5. Standard deviation of prediction Ln(N_{supply})

Number of Specimens Tested	Standard Deviation of Predicted Ln(N_{supply})
1	1.095
2	0.949
4	0.866
8	0.822

Table 4.6. Reliability multipliers

Sample Size	Variance of $Ln(N_{demand})$	Reliability Multiplier			
		60-Percent Reliability (Z_R = 0.253)	80-Percent Reliability (Z_R = 0.841)	90-Percent Reliability (Z_R = 1.28)	95-Percent Reliability (Z_R = 1.64)
1	0.2	1.349	2.704	4.545	6.957
	0.4	1.377	2.986	5.046	7.955
	0.6	1.404	3.090	5.567	9.022
	1.0	1.455	3.480	6.673	11.381
2	0.2	1.304	2.416	3.830	5.587
	0.4	1.334	2.609	4.305	6.490
	0.6	1.363	2.802	4.797	7.456
	1.0	1.417	3.188	5.839	9.592
4	0.2	1.280	2.270	3.482	4.945
	0.4	1.312	2.464	2.946	5.805
	0.6	1.342	2.657	4.425	6.723
	1.0	1.397	3.042	5.437	8.754
8	0.2	1.267	2.197	3.313	4.640
	0.4	1.300	2.392	3.772	5.479
	0.6	1.331	2.585	4.245	6.375
	1.0	1.388	2.970	5.243	8.356

4.4.4 Shift Factor

A shift factor must be applied to the traffic forecast to enable direct comparison to be made between field- and laboratory-traffic estimates. This shift factor will account for traffic wander, construction variability, and differences between laboratory and actual (field) states of stress, as well as other factors.

To this time relatively little research has been directed toward the definition of shift factors for estimating permanent-deformation. The Shell researchers (van der Loo 1976), based on an analysis of the influence of traffic wander on permanent deformation, have suggested that, when the effects of single and dual tires are considered together, the amount of rutting with wander is about the same as would be obtained if all vehicles traveled in one path.

In other studies where the creep test has been used within the framework of the layer-strain procedure (van der Loo 1978, Monismith et al., 1987), coefficients in the range 1 to 2 have been used to multiply the laboratory-estimated rutting values to permit them to correspond to field-measured values. For example, in a reasonably controlled study in Saudi Arabia, a factor of 1.5 was used to establish correspondence between laboratory-predicted (using the layer-strain procedure and creep test results) and field-measured values (Monismith et al., 1987).

Unfortunately, time limitations have not permitted the development of well documented shift factors in terms of load applications. Ultimately, such factors will necessarily have to depend on the results of controlled tests [e.g., laboratory wheel-tracking, FHWA-ALF, and special pavement studies (SPS-9) investigations].

Nevertheless, repeated load testing of pavement cores extracted from a limited number of general pavement studies (GPS) sites has provided an opportunity for developing first-generation estimates of appropriate shift factors (Sousa et al., 1993). These first-generation estimates are a starting point from which mix designers can make adjustments to reflect local experiences with mixes known to be either good or poor performers.

Analysis was limited to the seven GPS sections shown in Table 4.7. In each case, the pavement had been in service for fewer than nine years and the asphalt layer exceeded 12.7 cm (5 in.) in thickness. Laboratory test results are summarized in Table 4.8.

For this analysis, ESALs were adjusted to their equivalents at the 7-day average maximum pavement temperature at a 5.0 cm (2 in.) depth by applying a constant factor of 0.0725. This average appears reasonable in light of information available for the nine FHWA regions. Analysis of the data resulted in the following equation:

$$N_{demand} = 0.0562\ ESAL_{T'}^{0.924} \tag{4.6}$$

where:

N_{demand} = number of repetitions required in constant height, repeated load, simple shear test

$ESAL_{T'}$ = number of design-lane ESALs after conversion to its equivalent at the average maximum pavement temperature at a 5-cm (2-in.) depth, T′.

Because of the relative weakness in the regression of this equation and in the database from which it was derived and because the exponent of $ESAL_{T'}$ is so close to one, a simple shift factor relating N_{demand} to $ESAL_{T'}$ should *initially* suffice for mix-design purposes. It appears that a factor on the order of 0.04 might be an appropriate beginning point when testing and analysis is conducted at the critical instead of the maximum pavement temperature, Figure 4.7.

Thus, to determine N_{demand} for mix-design purposes, the traffic estimate (ESAL) is first converted to its equivalent at the critical temperature through use of the appropriate temperature conversion factor (Table 4.4). Then N_{demand} is simply the product of 0.04 and the ESALs at the critical temperature.

Table 4.7. GPS sections included in calibration

UCB Identification (SHRP Site #)	Surface Thickness (inches)	Age (years)	Average Maximum Daily Temp. (°F)	Rut Depth (inches)	Traffic Loading (ESALs)
GX64-1	16.7	1	52	0.14	637,550
(053071)		2	52	0.16	1,275,000
GX32-1	12	3	52	0.12	139,986
(171003)		4	52	0.17	179,982
GX29-1	10	4	54	0.20	284,935
(201009)		5	54	0.23	404,268
GX14-1	13	5	49	0.18	1,418,454
(211014)		6	49	0.19	2,051,845
GX44-1	9.5	4	43	0.23	980,000
(231012)		5	43	0.25	1,190,000
GX62-1 (351022)	6.6	6	49	0.15	724,306
GX71-1	7.7	7	54	0.16	1,637,481
(481029)		8	54	0.23	1,993,484

°C = 5/9 (°F−32)   1 in. = 2.54 cm

Table 4.8. Laboratory test results

UCB Identification (SHRP Site #)	Age (Years)	Test Temperature (°C)	Test Results	
			Permanent Shear Strain (%)	Cycles to Permanent Shear Strain
GX64-1 (053071)	1	52	1.27	6,872
	2	52	1.45	9,694
GX32-1 (171003)	3	52	1.09	185
	4	52	1.54	395
GX29-1 (201009)	4	54	1.82	451
	5	54	2.09	582
GX14-1 (211014)	5	49	1.64	3,089
	6	49	3.36	3,827
GX44-1 (231012)	4	43	2.09	756
	5	43	1.86	931
GX62-1 (351022)	6	49	1.36	1,304
GX71-1 (481039)	7	54	1.45	1,105
	8	54	2.09	2,170

°C = 5/9 (°F−32)

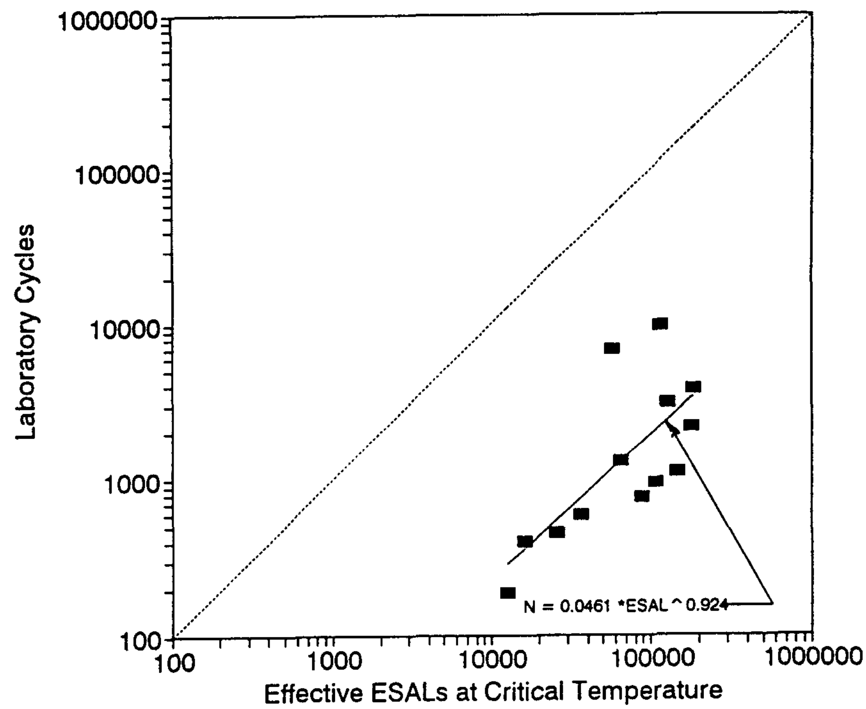

Figure 4.7. Relationship between laboratory cycles, N, and effective ESALs at the critical pavement temperature

4.4.5 Level 1 Analysis System

The Level 1 analysis system[9] includes the following steps:

1. **Determine design requirements for reliability and performance.** The analysis system outlined herein permits the designer to select a specific level of reliability commensurate with the pavement site for which the mix will be utilized.

   Performance requirements for permanent deformation generally call for the amount of rutting to not exceed some level, e.g. 10 mm to 13 mm (0.4 in. to 0.5 in.) in order to minimize the potential for hydroplaning.

2. **Determine expected distribution of in situ temperature.** Pavement analysis in the abridged procedure requires that the mix be evaluated at the critical temperature, T_c, the procedure for which has been briefly described earlier, and summary results are shown in Table 4.4.

[9]This system has been developed using information determined from the Level 2 procedure which makes use of the nonlinear viscoelastic, three-dimensional, constitutive relationship and a finite element analysis as described earlier.

For this mode of distress it is important to emphasize that temperatures in the upper part of the temperature range have a significant influence on the development of permanent deformation. Thus, if temperature data are available for a specific site in a region for which the critical temperature has been calculated using the FHWA model and if these data differ from the average for the region, then the site-specific critical temperature can be calculated following the procedure suggested herein so that the mix can be evaluated at this temperature rather than at the average for the region.

It is also important to emphasize that these computations need only be performed once for a specific region or subregion thereof.

3. **Estimate design traffic demand (ESALs).** For this procedure it is necessary to estimate the number of design-lane ESALs at the critical temperature. Accordingly, temperature equivalency factors, like those shown in Table 4.3, or more simple temperature conversion factors, like those shown in Table 4.4, must be available to convert the actual number of ESALs to the equivalent number at T_c. Conversion factors like those described earlier for Region III-B need only be computed once for specific regions. As noted earlier, it is likely that these conversion factors will be somewhat dependent on the structural pavement section; i.e., there may be a different conversion factor for a 10-cm (4-in.) asphalt-concrete overlay on a portland cement concrete pavement as compared to a comparatively thick asphalt-concrete layer for which the factor shown earlier had been determined.

4. **Select trial mix.** For a given binder and aggregate, a trial mix is selected. This might be done according to the SUPERPAVE methodology or by any procedure which the responsible agency considers appropriate. Changes or redesigns are evaluated at the discretion of the design (materials) engineer.

5. **Prepare test specimens and condition as required.** Cylindrical specimens 15 cm (6 in.) in diameter by 5 cm (2 in.) in height are obtained from slabs prepared by rolling wheel compaction in accordance with Harvey (1991). These specimens are cored and then sawed so that the end surfaces are smooth and parallel. The cut surfaces insure that the specimens are comparatively uniform throughout as compared to specimens prepared in molds which may have substantial density gradients both across their diameters and throughout their height (e.g. Eriksen 1992).

   Generally the specimens will have been subjected to the short-term oven aging (STOA) procedure (Bell et al., 1993) to simulate the mix as it exists early in its constructed life. If desired, to define longer term effects, the mix could also be subjected to long-term aging (LTOA) (Bell et al., 1993). Moreover, if the effects of water on permanent-deformation response are considered to be important, water conditioning can be accomplished using equipment and a procedure also developed as part of the A-003A contract (Terrel and Al-Swailmi, 1993a).

6. **Perform repeated load, constant height, simple shear tests.** In the Level 1 procedure, repeated load, constant height, simple shear tests are performed at the critical temperature, T_c, for the specific site. At this time, the recommended procedure is to use a shear stress of 70 kPa (10 psi) [associated with tire pressures of about 690 kPa (100 psi)] which is repeatedly applied with a duration of 0.1 sec and a time interval between loading of 0.6 sec. The repeated loading is continued for 1 hour, permitting the specimen to be subjected to a total of about 5000 stress repetitions. (N.B., for very stiff mixes extrapolation to the selected value of shear strain may be required.)

7. **Determine the resistance of the trial mix to permanent deformation.** From finite element analyses it has been determined that there exists a reasonably constant ratio between the maximum shear strain obtained in representative asphalt-bound layers and the permanent shear strain obtained in the constant height, simple shear test for the 690 kPa (100 psi) tire loading condition (Sousa et al., 1993).[10] Examples of these relationships have been shown earlier, Figure 4.4. The ratio is of the order of 10 to 11. That is, a rut depth of 1.3 cm (0.5 in.) would correspond to a shear strain of about 5 percent.

   Thus to develop N_{supply} for the given mix, the maximum shear strain is selected from the relation:

   $$\text{Rut depth} = 10 \text{ (or 11)} \cdot (\gamma_p)_{max} \quad (4.1)$$

8. **Apply a shift factor to the traffic demand (ESALs).** The design traffic volume, i.e., the laboratory-equivalent repetitions of the standard load, N_{demand}, is determined from:

   $$N_{demand} = ESALs_{Tc} \cdot SF \quad (4.7)$$

   where:

   $ESALs_{Tc}$ = design ESALs adjusted to the critical temperature, T_c; and
   SF = empirically determined shift factor.

   At this time it is recommended that a shift factor of 0.04 be used. This shift factor, as noted earlier, was determined from analyses of a limited number of GPS test sites.

9. **Compare traffic demand (N_{demand}) with mix resistance (N_{supply}).** Satisfactory performance requires that the mix resistance (N_{supply}) equal or exceed the traffic demand (N_{demand}). As with fatigue a multiplier, M, should be applied to N_{demand} since neither N_{supply} nor N_{demand} is known with

[10] Limited analyses for 1380 and 3450 kPa (200 and 500 lb/in^2) provided the ratio between rut depth and maximum shear strain.

certainty. This factor permits the incorporation of an appropriate level of design reliability as well. Thus, a satisfactory mix must meet the following:

$$N_{supply} \geq M \cdot N_{demand} \tag{4.4}$$

where:

M = multiplier whose value depends on the design reliability and on the variabilities of the estimates of N_{supply} and N_{demand}

Table 4.5 provides the variance of $Ln(N_{supply})$, as a function of the number of specimens tested and traffic level. Table 4.6 provides values for M depending on both the variance in the sample size and $Ln(N_{demand})$ for reliabilities varying from 60 percent to 95 percent.

10. **If inadequate, alter trial mix and perform another analysis iteration.** If the particular mix is determined to be inadequate a number of alternatives are available to the designer:

    - adjust the asphalt content;
    - adjust the aggregate gradation;
    - use a modified binder;
    - select another aggregate source; or
    - combinations of the above.

4.4.6 Mix Design Using Level 1 Methodology

The Level 1 methodology can be used as a mix-design procedure to select the initial binder content. Mixes can be prepared over a range in binder contents by rolling wheel compaction to an air-void content of about 3 percent. For each mix the procedure described in the previous section would be followed to select N_{supply}. The mix with the highest binder content which satisfies $M \cdot N_{demand}$ (adjusted traffic) would be that selected for further evaluation.

4.4.7 Level 2 Analysis

For the Level 2 analysis the following series of tests would be performed on a selected binder-aggregate mix at a temperature of 40°C (104°F) in the simple shear equipment.

1. **Uniaxial strain test.** A test in which the axial load is rapidly applied and the confining pressure necessary to maintain a constant diameter of the specimen is measured.

2. **Volumetric test.** A test in which the specimen is subjected to a hydrostatic stress state and the associated volume change is determined.

3. **Simple shear constant height test.** A test in which a shear stress is rapidly applied while maintaining the specimen at constant height and the corresponding shear strain is measured.

In addition, frequency sweeps in shear over a range in frequencies from 0.01 Hz to 10 Hz are performed at 4°, 20°, 40°, and possibly 60°C (39°, 68°, 104°, and 140°F). The shear stress is adjusted to provide a shear strain of about 0.0001 inches per inch, and an axial stress is applied to maintain constant height.

The data obtained from these tests are used to define the nonlinear elastic, viscous, and plastic parameters for the constitutive relationship for the mix (depicted schematically in the model illustrated in Figure 4.2). Details on the permanent deformation modeling effort are found elsewhere (Sousa et al., 1994).

With this constitutive relationship, it is then possible to estimate the rutting occurring in this mix within the specific pavement structure in which it is to be used (Sousa et al., 1993). If the estimated rut depth exceeds some prescribed level, a different mix must be evaluated in the same manner.

It is anticipated that this procedure would be used primarily for mix evaluation for major projects, for investigative analyses, and for model calibrations.

4.5 Summary

Results of the SHRP A-003A investigation in the permanent-deformation area provide a new test methodology and equipment to define the propensity of a mix for permanent deformation. The equipment permits the simultaneous application of shear and axial and/or normal stresses to cylindrical specimens as large as 20 cm (8 in.) in diameter and 8.9 cm (3.5 in.) in height. Temperatures of up to 70°C (158°F) can be used and confining pressures to 690 kPa (100 psi) can be applied.

Two levels are recommended for use of the equipment in permanent-deformation evaluation. For the first level, the simple shear test is performed in the unconfined condition with a single shear stress, e.g. 69 kPa (10 psi), at the critical temperature, T_c, and the load is repeatedly applied for one hour (0.1 second time of loading and 0.6 second time interval between load applications) to permit the definition of a relationship between shear strain and stress repetitions.

The second level encompasses the suite of tests including constant height shear creep, uniaxial strain, volumetric, and frequency sweep. The first three tests are performed at 40°C (104°F) while the frequency sweep is conducted at temperatures ranging from 4° to 60°C (39° to 10°F).

The Level 1 methodology permits determination of the suitability of a mix to carry the anticipated traffic in a specific environment. In the procedure, the number of repetitions in the test is an estimate of the traffic which can be carried to some prescribed level of rutting. The Level 2 procedure permits the estimation of rut depth for some prescribed traffic volume.

The repeated load, constant height, shear test can also be used for mix-design purposes. For a specific asphalt- and/or binder-aggregate mix, specimens are prepared over a range of asphalt contents at air void content of about 3 percent. A mix is considered suitable in this procedure if it can carry the prescribed number of repetitions in the simple shear test (associated with a specific rut depth) at this 3 percent air void content when tested at the critical temperature for the site.

The test appears to capture the important mix characteristics which define its propensity for permanent deformation and include the following:

- dilation under shear loading;
- increasing stiffness with increasing confinement at elevated temperatures;
- negligible volumetric creep;
- residual permanent deformation on removal of load; and
- temperature and rate of loading dependence.

Moreover, by performing the test in repeated loading rather than creep (Level 1 procedure), important differences in the accumulation of permanent deformation are obtained which may be particularly important when modified binders are used.

5

Accelerated Performance-Related tests for Low-Temperature Cracking Properties of Asphalt-Aggregate Mixes

The development of the proposed APT for evaluating the performance of asphalt-aggregate mixes in environments with low temperatures was divided into the following phases: a) review of the state of knowledge, b) testing programs, c) evaluation of test results, and d) validation of proposed methodology. This chapter presents a brief summary of the test selection process, the field validation of the selected test, and the framework for a mix design and analysis system to consider thermal cracking.

5.1 Literature Evaluation and Hypotheses

Low-temperature cracking of asphalt concrete results from cold temperatures and is considered to be a serious problem in those regions of the United States and Canada in which temperatures drop below freezing, resulting in thermal induced stresses in this element of the pavement structure. One hypothesis for thermal cracking is that as the temperature drops to an extremely low value producing tensile stresses that exceed the tensile strength of the asphalt concrete a micro-crack develops at the surface and edge of the pavement structure. The micro-crack eventually will penetrate the full depth of the asphalt concrete and spread across or, in the case of multi-lane highways, along the pavement surface. The second hypothesis is that thermal fatigue cracking occurs when temperatures cycle above the level required for low temperature cracking even though the stress in the pavement is typically far below the asphalt concrete at that temperature. Consequently, failure does not occur immediately, but develops over a period of time similar to the time required for fatigue cracking associated with load-induced strains in the asphalt concrete pavement layer. If thermal cracking occurs it may range from regularly spaced intervals of 30 m (100 ft) for new pavements to less than 3 m (10 ft) for older pavements. Material summarized herein is described elsewhere (Vinson et al., 1989).

5.1.1 Factors Affecting Low-Temperature Cracking

There are three factors which can affect thermal behavior of asphalt-aggregate mixes; namely, materials, environment, and pavement geometry. Material factors include the following: asphalt type, aggregate type and gradation, asphalt content, and air void content. Environmental factors include temperature, rate of cooling, and pavement age. Factors of pavement geometry include pavement width, layer thickness, and friction between the asphalt layer and the base course. Among the several factors mentioned above, the single most important one is asphalt type and, in particular, its temperature-stiffness relationship.

5.1.2 Review of Test Methods Associated with Low-Temperature Cracking

A number of test methods have been used to evaluate thermal cracking in asphalt-concrete mixes. The test methods which have been most widely employed include the 1) indirect diametral tension test, 2) direct tension test, 3) tensile creep test, 4) flexural bending test, 5) thermal stress restrained specimen test (TSRST), and 6) coefficient of thermal contraction test. In the review by Vinson et al. (1989) the methods were evaluated based on the following criteria:

- simulation of field conditions;
- application of test results to mechanistics models;
- suitability for aging and moisture conditioning;
- potential to accommodate large stone mixes;
- ease of conduct; and
- cost of equipment.

Of all of the tests evaluated, the TSRST appeared to come closest to satisfying the criteria. Moreover, it has been used successfully by other investigators; accordingly, it was selected for detailed evaluation.

5.2 Evaluation of Thermal Stress Restrained Specimen Test

The TSRST system developed in the A-003A contract is an automated, closed loop system which has been specifically designed to measure the tensile stress in an asphalt-concrete specimen as it is cooled at a constant rate while restrained from contracting. The test system consists of a load system, data acquisition and temperature control system, and a specimen alignment stand. The load, data acquisition, and temperature control systems are controlled with a personal computer. Figure 5.1 is a schematic of the TSRST apparatus developed at Oregon State University (Jung and Vinson, 1993).

Cooling rates for the low-temperature cracking test reported in the literature range from 3° to 30°C/h. The majority of investigators have conducted their tests at a cooling rate of 10°C/h. Field cooling rates, however, are much slower than 10°C/h (e.g. 0.5° to 2.7°C/h). Therefore, it would appear that tests should be conducted at a cooling rate slower than 2°C/h in the laboratory if field cooling rates are to be simulated. However, this slow rate of cooling results in an extremely long test program. Thus, most investigators have conducted

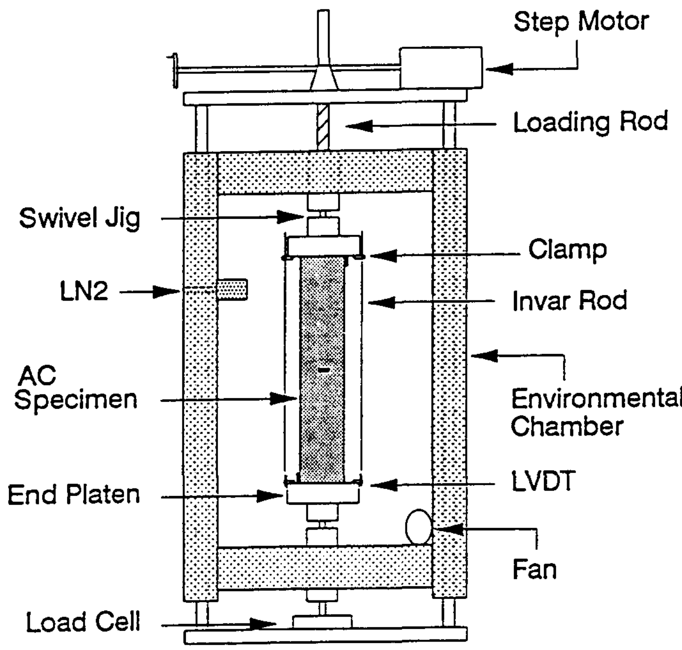

Figure 5.1. Schematic of TSRST apparatus

tests at a cooling rate of 10°C/h (or greater) and have used their results to provide a relative assessment of the propensity of asphalt concrete mixes to low-temperature cracking.

In this study, a cooling rate of 10°C/h and two different specimen sizes of 38 mm × 38 mm × 200 mm and 50 mm × 50 mm × 250 mm (1.5 in. × 1.5 in. × 8 in. and 2 in. × 2 in. × 10 in.) were selected to investigate low-temperature cracking in asphalt-concrete mixes.

5.2.1 Experiment Design

Five variables were included in the experiment design: 1) binder type and stiffness; 2) aggregate type; 3) specimen size; 4) void content; 5) cooling rate (10°C/h and 1°, 2°, and 5°C/h); and 5) long-term oven aging (4 days at 110° or 135°C [230° or 275°F]).

After mixing and curing (15 hours at 60°C [140°F]) specimens were compacted into beams using the Cox kneading compactor. The two sizes of test specimens were then sawed from the beams. Some specimens were subjected to additional oven aging at 110° or 135°C (230° or 275°F) to simulate longer term aging.

Typical results from the TSRST test are illustrated in Figure 5.2. The thermally induced stress gradually increases as the temperature is lowered until the specimen breaks. The stress at the break point is referred to as the fracture strength. The slope of the stress-temperature curve, dS/dT, increases gradually until the temperature reaches a certain value and dS/dT reaches its maximum at this temperature. Beyond this temperature, dS/dT becomes constant and the stress-temperature curve is linear. The slope tends to decrease again when the specimen is close to the break point. This may be due to the formation of micro-cracks.

Figure 5.3 illustrates typical stress-temperature curves observed for specimens prepared with two asphalts (AAG-1, AAK-2), one aggregate (RB), and at two void contents, 4 and 8 percent. The maximum induced thermal stress corresponds to the fracture temperature in this figure. The thermally induced stresses develop more rapidly in the specimens with the stiffer asphalt AAG in this temperature range. In addition, the slope of the dS/dT portion of the curve tends to be greater for specimens with lower air voids; however, lower air voids are associated with higher induced stresses and tensile strength at fracture. Summary data are shown in Table 5.1 and were obtained using the smaller sized specimens and a monotonic cooling rate of 10°C (18°F).

5.2.2 Discussion

The repeatability of the TSRST was evaluated based on the coefficient of variation for fracture temperature, transition temperature, fracture strength, and slope. The coefficients of variation for fracture temperature were less than 10 percent. For fracture strength the coefficients of variation were generally less than 20 percent, while for transition temperature and slope they were less than 15 percent and 25 percent respectively.

From the statistical analysis of TSRST results over a range of conditions, fracture temperatures were most affected by asphalt type. Fracture temperatures also were affected to a lesser degree by aggregate type, specimen size, degree of aging, and cooling rate. Fracture strength is most affected by air void content and aggregate type and to a lesser degree by asphalt type, stress relaxation, and cooling rate.

The resistance to low-temperature cracking of the asphalts used in this study, based on fracture temperature, is AAK-2 > AAK-1 > AAG-2 > AAG-1 for all variables considered. This ranking is in excellent agreement with the ranking based on the physical properties of the asphalt cement measured by researchers at Pennsylvania State University.

The RB aggregate showed better resistance to low-temperature cracking than did the RL aggregate, i.e. colder fracture temperatures. Fracture strengths of specimens with RB aggregate were greater than those for specimens with RL aggregate. This may be attributed to aggregates RB's rough surface texture and angular shape which may provide better bonding and interlocking.

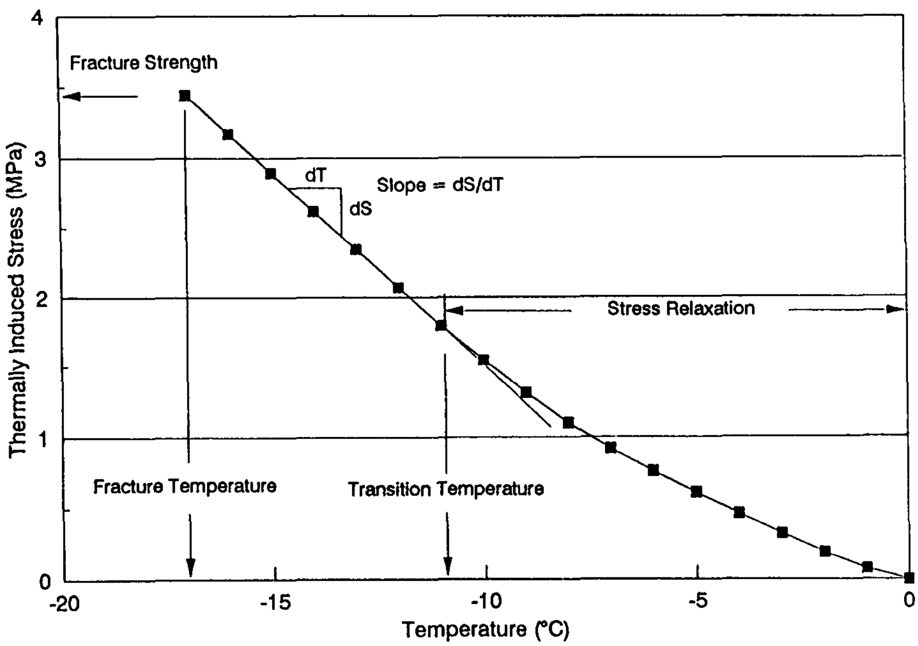

Figure 5.2. Typical result from TSRST

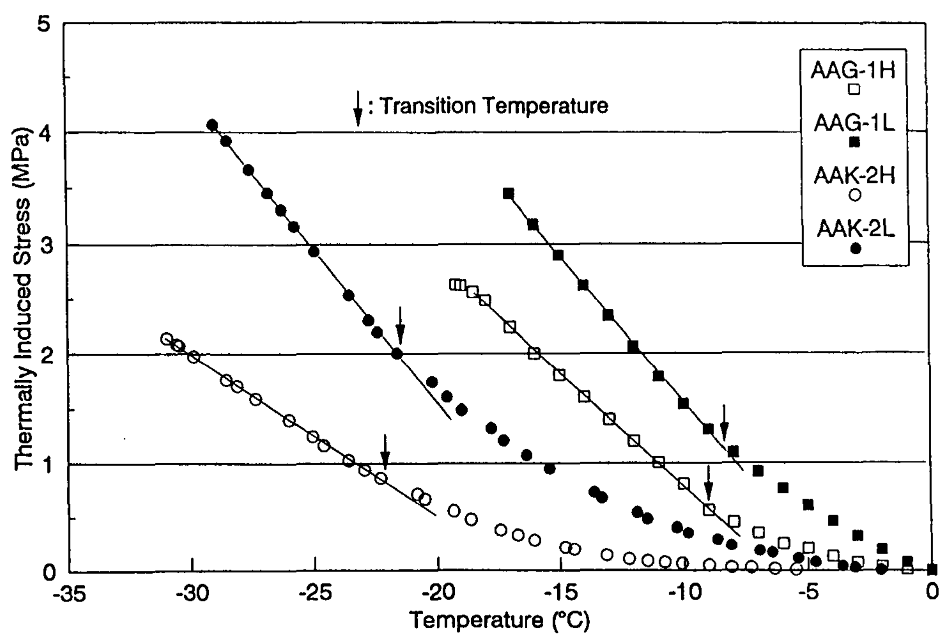

Figure 5.3. Typical stress-temperature curves

Table 5.1. TSRST test results

Asphalt	Target Air Voids Content (%)	Air Voids Content (%) Mean	σ	Fracture Temperature (°C) Mean	σ	Fracture Strength (MPa) Mean	σ	Slope (dS/dT) (MPa/°C) Mean	σ	Transition Temperature (°C) Mean	σ
AAG-1	8	8.2	0.56	-17.8	0.15	2.472	0.713	0.218	0.014	-10.0	0.55
	4	4.3	0.74	-16.6	1.23	3.146	0.193	0.264	0.022	-9.1	0.10
AAG-2	3	8.4	1.34	-18.8	0.89	2.481	0.267	0.226	0.013	-12.1	0.62
	4	5.2	0.76	-17.6	0.43	3.012	0.406	0.274	0.013	-11.1	0.10
AAK-1	8	7.9	1.29	-25.2	1.72	2.270	0.400	0.151	0.033	-16.0	0.69
	4	3.5	0.66	-23.7	0.95	3.021	0.465	0.190	0.020	-13.8	0.37
AAK-2	3	7.6	0.86	-30.9	0.29	2.389	0.167	0.145	0.013	-21.4	0.67
	4	3.9	0.31	-29.7	0.61	4.039	0.102	0.269	0.012	-20.8	0.61

Note: Asphalt contents were as follows:
AAG and RB, 4.9%; AAG and RL, 4.1%; AAK and RB, 5.1%; and AAK and RL, 4.3%.
All asphalt contents are by weight of aggregate.

Fracture temperature was also influenced by specimen size, as it was lower for larger size specimens. Stress relaxation appeared to influence the fracture temperature of mixes containing asphalt AAG but not AAK, with the fracture temperature being lower if stress relaxation were permitted. Cooling rate also influenced the fracture temperature, being lower as the cooling rate was decreased.

5.3 Validation Studies

Five test roads were selected for field validation of the TSRST. These included two in Fairbanks, Alaska, one in Elk County, Pennsylvania, and two in Finland. In addition, a validation program was conducted at the Frost Effects Research Facility (FERF) of the U.S. Army Cold Regions Research and Engineering Laboratory (USA CCREL) at which several full-scale test sections were constructed. In the FERF the environmental conditions can be precisely controlled and extensive instrumentation systems are available for temperature measurements and crack detection.

5.3.1 Field Studies

Test sites were selected in areas subjected to cold temperatures which could lead to thermal cracking. Actual sites were chosen where the materials used in the initial construction (asphalt and aggregate) were available to permit fabrication of laboratory test specimens for evaluation in the TSRST.

Alaska. Two pavement sections constructed with the same materials in Fairbanks in 1988 were utilized. Both contained a crushed gravel and AC-5 asphalt cement. One section contained a 50 mm (2 in.) layer of asphalt-concrete, the other a 75 mm (3 in.) layer. Laboratory test specimens were prepared using the materials. In addition, since AC-2.5 is normally used for paving in Fairbanks, specimens were also prepared using this grade from the same asphalt supplier. (No low-temperature cracking was observed in pavements containing the AC-2.5.) Slabs were also obtained to permit testing of the actual pavements.

During the most severe temperature period when pavement temperatures were in the range -35° to -40°C (-31° to -40°F) the cooling rate was about 0.7° (1.3°F) per hour. In the laboratory specimens were therefore tested at a cooling rate of 1.0°C (33.8°F) per hour as well as the usual 10°C (50°F) per hour.

Results of the testing are summarized in Table 5.2. These results are considered inconclusive as regards validation of the TSRST procedure. Using a field cooling rate of 1.0°C (1.8°F) per hour, the fracture temperature of the AC-5 mix is less than 1°C warmer than the fracture temperature of the control AC-2.5 mix, and yet portions of the AC-5 pavements are considered severely cracked with no cracking in the control sections.

The mean fracture temperature of the field specimens from the cracked sections was approximately 3.5°C (6.5°F) warmer than the mean for the laboratory specimens. However, the fracture temperature for the uncracked mix was close to the fracture temperature of the AC-5 mix prepared in the laboratory. The fracture temperature for the uncracked section was 2.7°C (4.7°F) colder than that of the cracked sections. Hence, the TSRST ranked the mixes in the correct order. The TSRST fracture temperatures were approximately 10°C (50°F) warmer than the minimum field pavement surface temperature, thus cracking could be expected for both sections as was generally observed in the field.

Pennsylvania. Six test sections were constructed in Elk County, Pennsylvania, in September 1976, using AC-20 asphalts from different sources as a cooperative durability project between Penn DOT and the FHWA. During the first winter (1977) rapid cooling occurred with a minimum pavement temperature reaching -23°C (-10°F). When condition surveys were conducted, two sections, T-1 and T-5, had developed severe low-temperature cracking. Subsequently (after 5 years), sections T-2, T-4, and T-6 had developed different degrees of cracking while section T-3 exhibited none.

Figure 5.4 contains the fracture temperatures of the sections determined with the TSRST at a cooling rate of 5°C/hr (9°F/hr) for laboratory prepared specimens which had been STOA. It can be noted that fracture temperatures of section T-1 and T-5 are higher than the minimum temperature indicating that the results of TSRST provide a suitable explanation for the observed cracking after the first winter.

Table 5.2. Summary of TSRST results for Alaska sections

a. laboratory specimens

Asphalt Cement	Mean Air Void Content (%)	Mean Fracture Temperature		Number of Observations
		(°C)	(°F)	
Cooling Rate 10°C/h (18°F/h)				
AC-5	2.2	-25.8	-14.4	4
AC-2.5	3.0	-28.2	-18.8	4
Cooling Rate 1.0°C/h (1.8°F/h)				
AC-5	2.7	-30.4	-22.7	2
AC-2.5	2.6	-31.1	-24.0	2

b. field samples

Asphalt Cement	Air Void Content (%)	Mean Fracture Temperature		Number of Observations
		(°C)	(°F)	
Cooling Rate 1.0°C/h (1.8°F/h)				
23rd South	4.5	-26.7	-16.1	2
23rd North	5.4	-29.3	-20.7	2
Peger Tr.	2.3	-27.2	-17.0	4
Perger Par.	3.2	-27.4	-17.3	4

Tr. = samples were taken transverse to the direction of traffic.
Par. = samples were taken parallel to the direction of traffic.

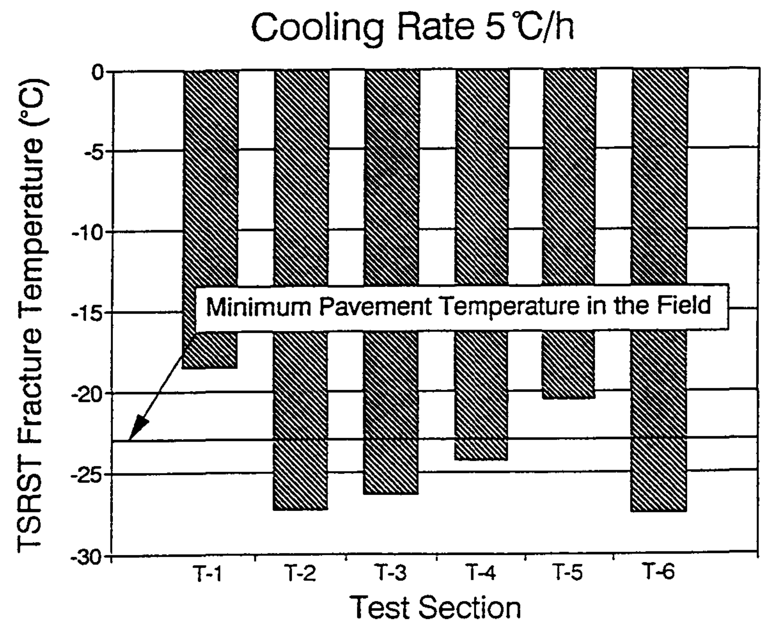

Figure 5.4. TSRST fracture temperatures and minimum pavement temperatures for Pennsylvania test sections

To investigate the relationship between Cracking Index[11] for pavement ages from 1 to 4 years and the TSRST fracture temperatures, a regression analysis was performed. Results of this analysis are shown in Figure 5.5. The values for the fracture temperature represent the average of two cooling rates used in the TSRST, 5°C/hr and 10°C/hr. This analysis indicates that there is a strong relationship between fracture temperature and Cracking Index (Kanerva et al., 1992).

Finland. Two projects, part of the Asphalt Pavement Research Program funded by the government of Finland, have been utilized. Both projects made use of the same asphalts with penetration values ranging from 65 to 150.

Peraselinajoki Project. Extensive pavement temperature data were collected using a temperature data logger recording data obtained from thermocouples located at the surface and at depths of 25 mm (1 in.), and at 50 mm (2 in.) (the bottom of the asphalt concrete). The coldest recorded air temperature was -30°C (-22°F) and the coldest pavement temperature was -20°C (-4°F). The maximum cooling rate was 0.7°C/h.

[11]Cracking Index = (full + 1/2 × half cracks + 1/4 × partial cracks)/500 feet.

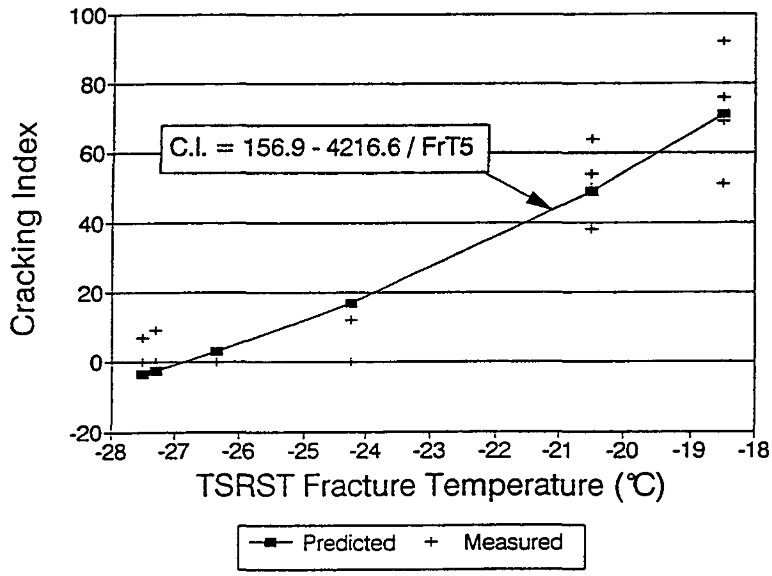

Figure 5.5. Cracking Index versus TSRST fracture temperature for Pennsylvania test sections

No low-temperature cracks were observed in any of the six test sections through the first two winters. Since no cracks were observed in the Peraselinajoki test road no specimens were prepared in the laboratory. However, because the asphalt cement is recognized as the most significant factor influencing low-temperature cracking, the laboratory test results for the Sodankyla samples could be used to represent the Peraseinajoki sections (the asphalts are the same, both test roads have a well-graded aggregate, and the asphalt contents are within 0.4 percent). A summary of the Sodankyla test results presented in the next section have been used to interpret the observed field behavior.

The mean TSRST fracture temperatures (cooling rate 2°C/h, 3.6°F/h) for the test sections and the minimum pavement temperature in the field are shown in Figure 5.6. The TSRST fracture temperatures of the field sections are all colder than the minimum pavement temperature of -20°C. Hence, the cracking behavior of the test sections can be explained by the TSRST fracture temperatures.

Sodankyla Project. This section included nine different asphalt cements with the range in penetrations noted above. The coldest pavement temperature was -24.5°C (-12.1°F) and the maximum cooling rate was 2.3°C/h (4.1°F/h). Crack measurements were obtained during the coldest months over a 300 m (984 ft) length of each section.

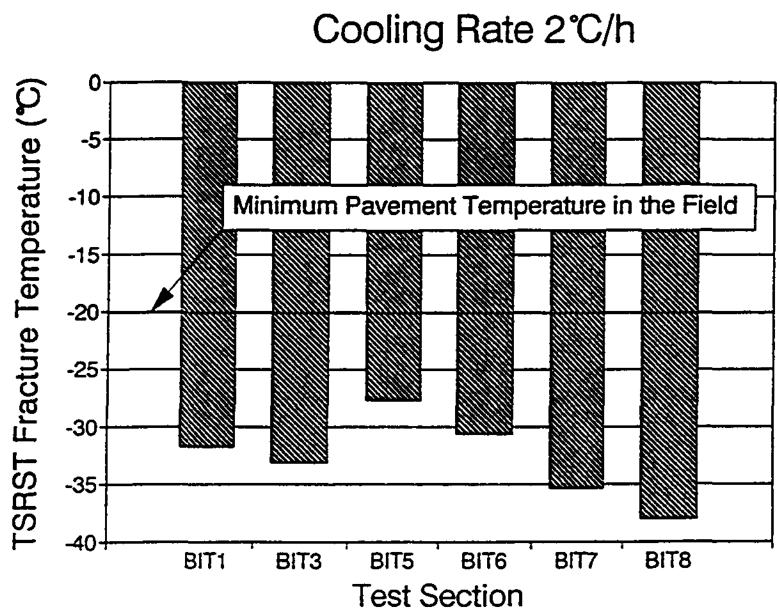

Figure 5.6. TSRST fracture temperature and minimum pavement temperature for Peraseinajoki test sections

TSRST specimens were obtained from laboratory compacted field plant mixed material obtained at the time of production. (No additional STOA was used.) Fracture temperatures, determined at a cooling rate of 2°C/h, are shown in Figure 5.7. It will be noted that the minimum pavement temperature is higher than any of the fracture temperatures. Thus, one would conclude that none of the sections should have cracked. However, as seen in Figure 5.8, all sections exhibited some cracking after the first year.

A number of conditions make the results difficult to interpret, including the following:

- The test sections were limited to one lane only and a large number of full cracks extending over the entire pavement were observed. Cracks may have occurred on the other land and advanced to the section in question.

- Only 984 ft (300 m) long segments were observed periodically. If the first crack occurred outside the segment, the moment cracking was observed would relate to the second or possibly even a third or fourth crack.

- The transverse crack pattern in the pavement before reconstruction was given in the construction documents. Approximately half of the cracks that occurred in the first winter, appeared at the same locations as the old cracks. Thus, part of the cracks may have been reflected through the base course.

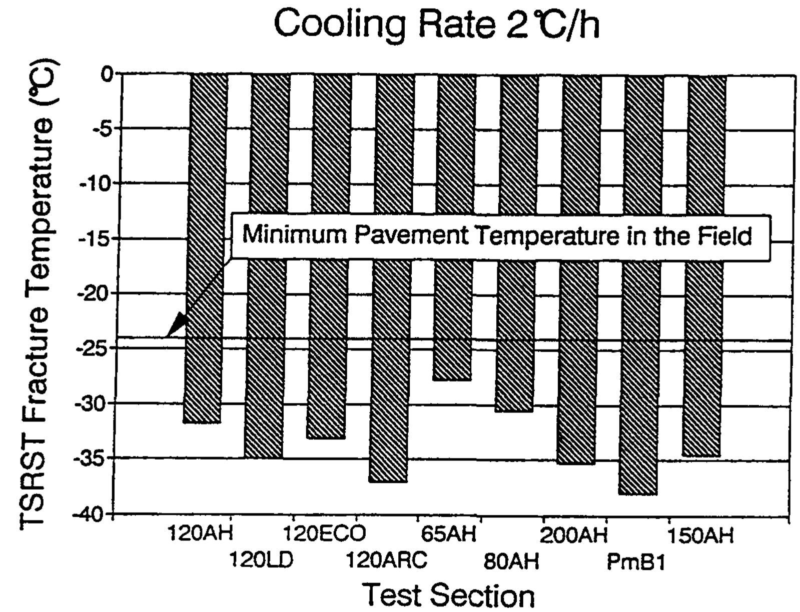

Figure 5.7. TSRST fracture temperature and minimum pavement temperature for Sodankyla test sections

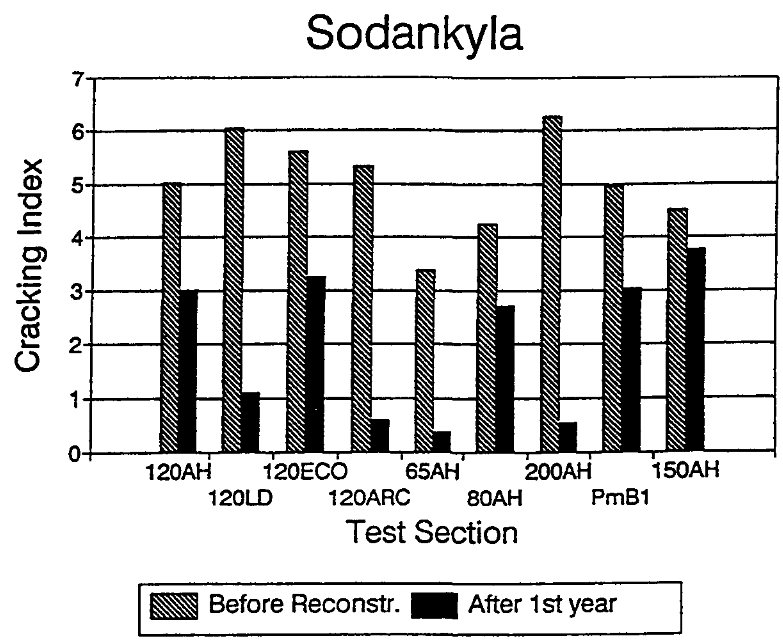

Figure 5.8. Cracking frequency before reconstruction and after first year of Sodankyla test sections

- The cracking frequency in the existing pavement was not constant (see Figure 5.8), regardless of the fact that there was no variation in the materials. This leads to the conclusion that there is a variation in the conditions of the test sections.

- The length of the sections were not constant.

- Ground thermal contraction may have caused a number of cracks in the pavement-wearing course rather than contraction in the asphalt concrete.

A multiple regression analysis of cracking frequency versus fracture temperature was also performed, Figure 5.9. Several prediction models were considered but only 28 percent of the variable "cracking air temperature" could be explained with TSRST fracture temperature and 17 percent of the variable "cracking pavement temperature."

According to the multiple regression analysis, the cracking frequency increases with decreasing TSRST fracture temperature, which is not possible. Visual inspection of the results presented in Figure 5.9, together with a knowledge of the low-temperature behavior of the PmB (SBS modified) and B1T65AH (penetration grade 65, lowest of the nine asphalts), suggest that the two data points are unreasonable. Even if these two outliers are omitted, there is not enough evidence that the TSRST fracture temperature is associated with the Cracking Index (p-value in two-sided t-test is 0.13).

In conclusion, there are apparently factors other than the mix properties, such as those noted earlier, affecting low-temperature cracking of these test sections.

5.3.2 USACRREL Study

The FERF facility consists of test basins where environmental conditions, such as temperature and moisture content, can be controlled. Four test sections were included in the performance phase of the investigation and each section used a different asphalt; i.e., different source or grade. The asphalt concrete was 50 mm (2 in.) thick and each of the sections were 61 m (200 ft) long and 1.2 m (4 ft) wide.

Temperatures in the pavement structure were measured using thermocouples. The minimum temperature observed at the surface of the pavement was -36.7°C (-34°F) and at the bottom of the pavement, -32.8°C (-27°F).

The crack detection system consisted of two types of aluminum tape and hard drawn copper wire bonded to the pavement surface with adhesive. Based on the recorded temperature profiles, cracking generally did not occur before the minimum possible temperature for the cooling system was achieved. This resulted in a constant surface temperature for a period of time before the onset of cracking. Thus, the surface temperature does not reflect

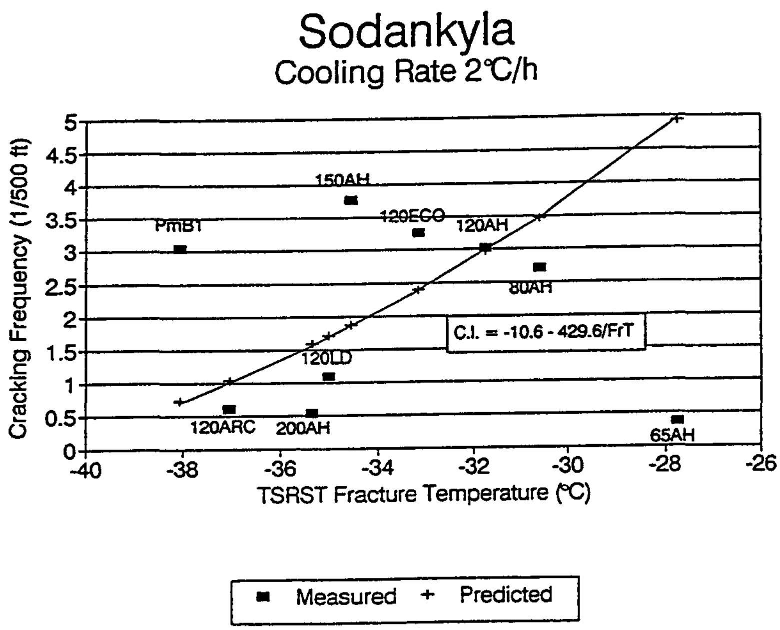

Figure 5.9. Cracking frequency versus TSRST fracture temperature for Sodankyla test sections

the cracking temperature, but the minimum temperature achieved by the cooling panels does affect the cracking temperature. However, the temperature at the bottom of the asphalt-concrete layer decreased until cracking occurred in almost all cases. The stress due to the distribution of temperature in the pavement layer initiated cracking, rather than the stress associated with the surface temperature. Consequently, the temperature at the bottom of the asphalt pavement is considered a better indicator of the cracking temperature than the surface or average temperature and has been used as an indicator of this parameter.

TSRST tests were performed on the various mixes at cooling rates of 10°C/h and 1°C/h. Multiple regression analysis was performed to investigate the relationship between the cracking temperature of the test sections and TSRST fracture temperature of the corresponding mixes. Results of the regression analysis suggest that the TSRST fracture temperature (FrT) is associated with the cracking temperatures (CrT), as seen in Figure 5.10.

A Cracking Index was also determined [CI = (Full +0.5 Half) cracks/500 ft]. To investigate the correlation between the Cracking Index in the FERF and the TSRST fracture temperature a multiple regression analysis was performed. The analysis was made with the TSRST fracture temperatures for the tests with a cooling rate of 10°C/h (18°F/h) for unaged samples. There is evidence that the TSRST fracture temperature is associated with the Cracking Index (p-value in two sided t-test 0.03). The predicted Cracking Index together with measured values versus the TSRST fracture temperatures are given in Figure 5.11.

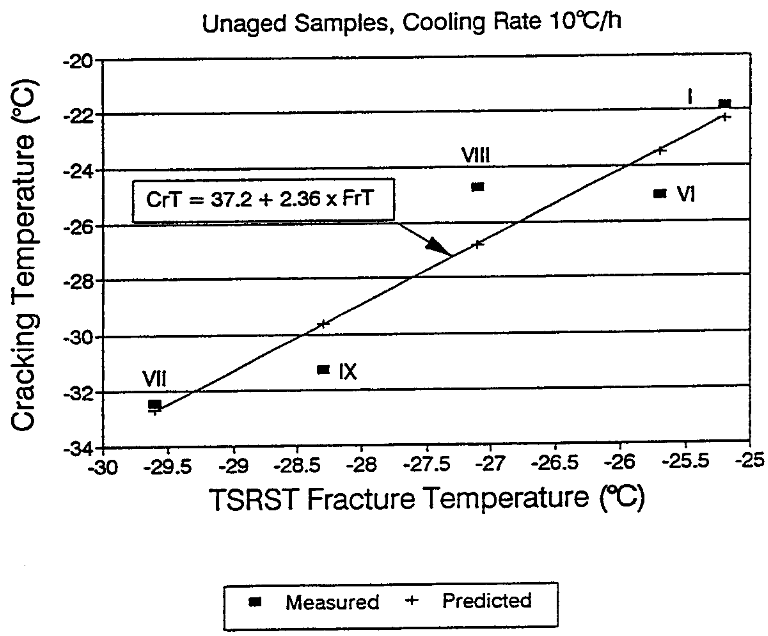

Figure 5.10. Predicted cracking temperatures for USACRREL test sections

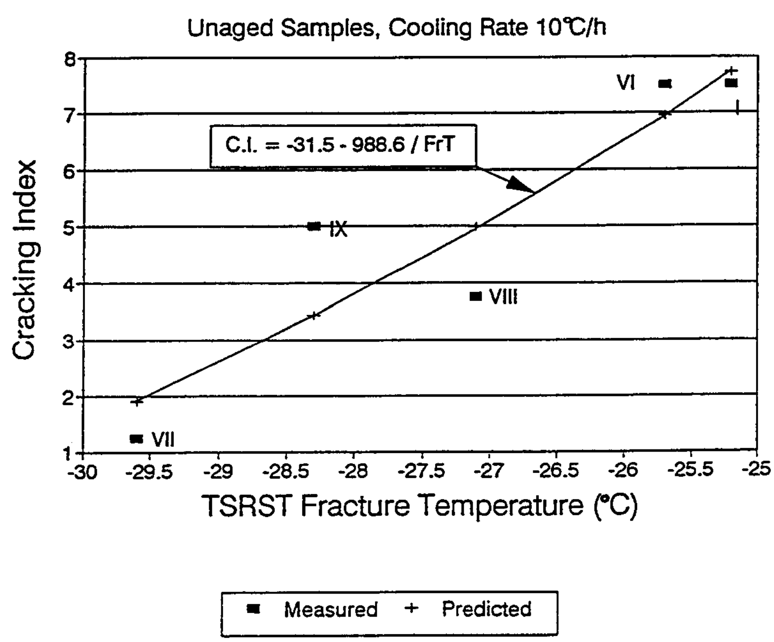

Figure 5.11. Predicted cracking index for USACRREL test sections

Use of a slow cooling rate (1°C/h rather than 10°C/h) or short-term aging of the samples does not improve the relationship between the cracking temperature in the field and TSRST fracture temperature. The fracture temperatures for laboratory samples versus field samples are shown in Figure 5.12. The test sections were not aged in the field and, accordingly, the unaged laboratory samples are closer to the actual field samples in regard to TSRST fracture temperatures, than the short-term aged samples.

Based on the USACRREL test program, where the environmental variables were closely controlled, the TSRST fracture temperature was able to predict the cracking temperature and frequency for the four mixes tested.

5.4 Mix Design and Analysis

Distinguishing characteristics of the thermal cracking system are summarized in Table 5.3. The three levels are differentiated by the amount of testing and subsequent analysis required.

Level 1 requires no testing of the mix. Testing of the PAV-aged asphalt cement provides the necessary input data. Correlations between asphalt properties and the results of TSRSTs on long-term oven aged (LTOA) specimens provide an estimate of the fracture temperature. Comparison of this temperature with the pavement surface temperature, estimated on the basis of temperature data for the coldest year in ten, provides an indication of the suitability of the mix containing that asphalt for use at the specific site. As seen in Table 5.4 this level is only recommended for dense-graded mixes containing conventional asphalt cements. For unconventional mixes with conventional binders, Levels 2 or 3 are recommended, while for mixes containing binders for atypical temperature sensitivity (e.g., modified binders) only Level 3 is recommended for use.

Level 2 requires limited testing of the mix using the TSRST. A cooling rate of 10°C per hour is recommended and the tests should be performed on long-term oven aged (LTOA) specimens. The analysis consists of using weather data (coldest year in 30) to estimate the pavement surface temperature and the fracture temperature, T_{frac}, derived from the TSRST to estimate the propensity for cracking using a regression equation developed from available performance data. By setting a maximum amount of cracking for a design's time period, e.g. 10 years, the suitability of the mix can then be judged. Details of the Level 2 design are found elsewhere (Jung et al., 1994).

Level 3[12] requires more detailed testing of the proposed mix in the TSRST using both short-term oven aged (STOA) and LTOA specimens at a cooling rate commensurate with actual site data, e.g. 1°C per hour. Results of the more extensive test program may be used in a viscoelastic analysis system which requires, in addition to the TSRST data, the mix stiffness as a function of temperature and time of loading and the thermal characteristics of the asphalt mix. The program permits an estimate to be made of the increase in crack frequency with time either deterministically or in a probablistic mode.

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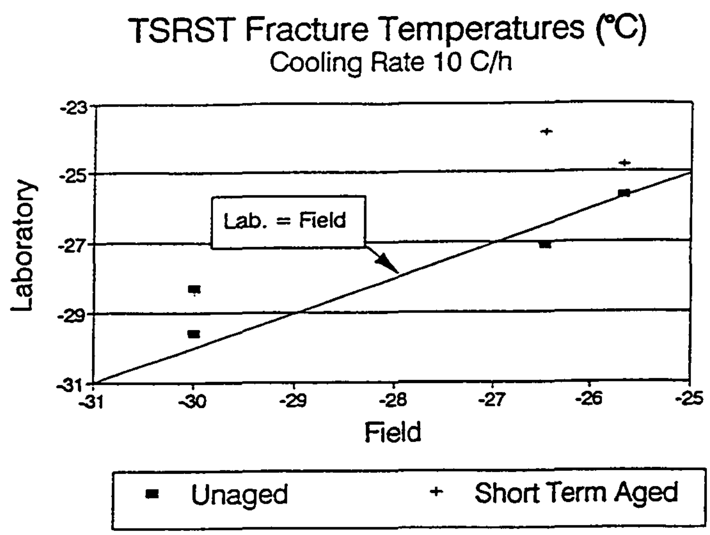
[12]Level 3 was not completed within the time framework of the A-003A program. Accordingly, this represents a recommended approach.



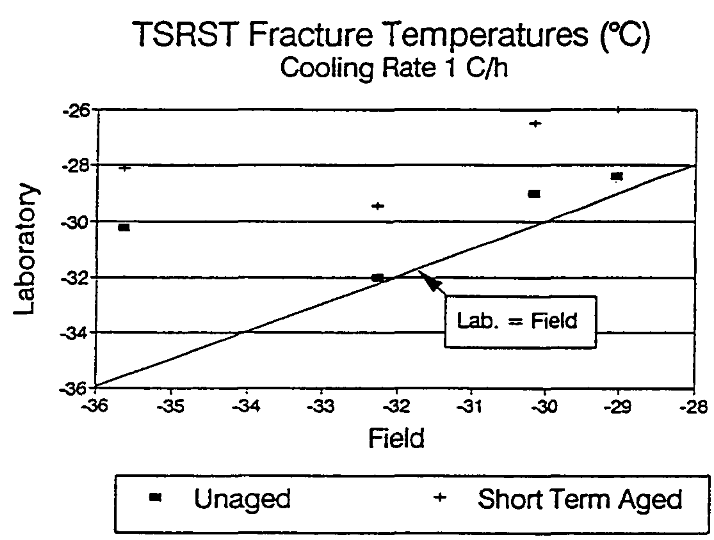


Figure 5.12. TSRST fracture temperatures of laboratory specimens versus field samples for USACRREL test sections

Table 5.3. Distinguishing characteristics of low-temperature cracking analysis systems

Variables		Level 1	Level 2	Level 3
		Abbreviated analysis with only asphalt testing	Abbreviated analysis with limited testing in thermal stress restrained specimen test	Comprehensive analysis with more extensive thermal stress restrained specimen test program
Testing	Type	Tests on PAV aged binder to estimate T_{frac} • temp at limiting stiffness • m-value • Ultimate strain at failure	Thermal stress restrained specimen T_{frac} (long-term oven aged)	Thermal stress restrained specimen test, T_{frac} (short-term and long-term oven aged)
	Temperature	Dependent on location (environment)	Cooling rate 10°C per hour	Cooling rate 1°C per hour
In situ conditions	Traffic	N.B., selection of level may be independent of traffic volume.		
	Structure	Information not required	Information not required	Thermal stress(es) resulting from temperature reduction
	Temperature	Pavement surface temperature; coldest year in 10	Pavement surface temperature; coldest year in 30	30 years of temperature records for area
Analysis	Mechanistic	No analysis	Regression equation: crack index = F(T_{frac} DPST, pavement age)	Finite element solution (e.g., ANSYS) using viscoelastic and thermal properties
	Damage	Cracking occurs if: T_{frac} < DPST	Cracking index — member of cracks per 500 ft	Crack density

Table 5.4. Recommended level of thermal fracture testing and analysis

Mix Characteristics	Level 1	Level 2	Level 3
	Abbreviated analysis with only asphalt testing	Abbreviated analysis with limited testing in thermal stress restrained specimen test	Comprehensive analysis with more extensive thermal stress restrained specimen test program
Dense-graded mixes with conventional binders of typical temperature sensitivity	Recommended	Optional for increased accuracy	Optional for increased accuracy or complete mix cataloging
Unconventional mixes with binders of typical temperature sensitivity	Not recommended at this time	recommended	Optional for increased accuracy, complete mix cataloging, or investigative analysis
Mixes with binders of atypical temperature sensitivity	Not applicable	Not applicable	Required

5.5 Summary

Based on comprehensive evaluation of existing methods to evaluate thermal properties and low-temperature cracking tendencies, the TSRST procedure was selected to achieve the project objectives.

Specific information concerning the utility of the TSRST method were confirmed through testing as follows:

1. TSRST test results provide an excellent indication of the resistance of asphalt-concrete mixes to low-temperature cracking and are in good agreement with rankings based on the physical properties of the asphalt cements used in the mixes.

2. TSRST test results are sensitive to the effects of asphalt source, aggregate type, air void content, degree of aging, and rate of change in pavement temperature. These five variables represent the major factors to be considered in the design of asphalt-aggregate mixes to mitigate low-temperature cracking.

3. TSRST test results can be correlated with specific physical properties of the asphalt cement to facilitate a simplified approach to controlling low-temperature cracking or for preparation of binder specifications.

4. Repeatability of the TSRST is considered to be very good compared to other methods of testings used to define the propensity of mixes to low-temperature cracking.

Results of the validation studies associated with three pavement sites in Alaska, Pennsylvania and Finland together with tests conducted in the FERF of the U.S. Army Corps of Engineers have provided the following:

1. Cracking behavior of the test roads could be explained with TSRST fracture temperatures for projects in Alaska, Pennsylvania, Finland (one of two investigated) and USACRREL. For the second project in Finland, there were other factors in addition to mix properties affecting low-temperature cracking. Hence, the TSRST can be utilized in the prediction of low-temperature cracking of asphalt-aggregate mixes.

2. Preliminary models to predict cracking frequency and temperature for the test roads were developed. Consequently, it appears feasible to develop a model that would predict the development of cracking in all climates as a function of age (time).

Additional research is required to further validate models to be used for predicting both fracture temperature of in-service pavements and crack frequency.

6

Accelerated Performance-related tests for Aging of Asphalt-Aggregate Mixes

The development of the tests for aging was divided into four phases: 1) review of the state of knowledge (i.e., review of the literature and identification of candidate test methods); 2) test development program to evaluate the primary test methods and to select the proposed test methods; 3) expanded test program with the selected test methods; and 4) efforts to validate the proposed test methods. This chapter presents a brief summary of the results of the literature review, the test selection process, field validation of the selected tests for both short- and long-term aging, and a framework for a mix design and analysis procedure to consider aging.

6.1 Literature Evaluation and Hypotheses

Compared to research on asphalt cement, there has been little research on the aging of asphalt mixes, and, to date, there is no standard laboratory test for use by the asphalt paving community. Pavement engineers understand the need to model the effects of short- and long-term aging of asphalt-aggregate mixes in structural design procedures, and, while some research has addressed this need, no standard procedure has emerged, as yet, to address it. While several researchers have compared laboratory aging tests and field performance, the majority have been concerned with the aging of the binder rather than of the mix. There are exceptions, however, such as the studies in California, Oregon, and a recent study for the NCHRP AAMAS project. These studies involved measurement of mix properties on both field cores and laboratory specimens.

The literature (Bell, 1989) indicates that two major factors dominate aging of asphalt-aggregate mixes:

1. Loss of volatile components and oxidation in the construction phase (short-term aging).

2. Progressive oxidation of the in-place mix with time in the field (long-term aging)

Other factors may also contribute to aging. For example, molecular structuring may occur over a period of time, resulting in steric hardening. Actinic light, primarily in the ultraviolet range, may also have an effect, particularly in desert climates.

Aging results in hardening (stiffening) of the mix which, in turn, results in a change in its performance. This may be beneficial since a stiffer mix will have improved load distribution properties and will be more resistant to permanent deformation. However, aging may also result in embrittlement (increased tendency to crack) and loss of durability in terms of wear resistance and moisture susceptibility.

Development of a laboratory method or methods to simulate aging should thus concentrate on reproducing the two dominant effects, i.e., volatilization and oxidation. Clearly, the level of temperature will play a part in the extent of volatilization, but it will also affect the amount of oxidation, and it is thought that for a particular temperature there will be a threshold level of aging. The rate at which oxidative aging occurs in laboratory specimens will be accelerated by enriching the concentration of oxygen by using a pure oxygen environment or by using compressed air or pressurized oxygen.

It is thought that asphalts will differ in their relative aging susceptibility at high and low temperatures, and that asphalts which age most under high temperature conditions may age less in subsequent low-temperature, pressure oxidation conditions. This phenomenon must be considered carefully when evaluating alternative aging procedures. Similarly, aggregate surface chemistry seems to have a significant effect on aging of an asphalt-aggregate mix and may be as significant as the asphalt properties in the aging of mixes.

It would thus seem reasonable to develop a conditioning device using a combination of temperature and/or oxygen concentration which will cause a similar level of aging to that which typically occurs in the field. It must be recognized, however, that more than one set of conditions may be necessary depending on the environment, and that the effects of structuring and actinic light must be considered.

6.2 Test Development

Based on the evaluation briefly summarized above, laboratory procedures were developed for both short-term and long-term aging of asphalt-aggregate mixes. This program consisted of two phases: 1) a preliminary program to provide the basis for test selection, and 2) a supplemental program to refine the procedures developed in the initial (preliminary) program.

6.2.1 *Aging Procedures*

In the preliminary program the following short-term aging methods were evaluated:

- short-term oven aging (STOA) of loose mix in trays in a forced draft oven (FDO) at 135° and 163°C (275° and 325°F); and,

- extended mixing of loose mix at 135° and 163°C (275° and 325°F) in a modified rolling thin film oven.

The following long-term methods were also considered in the preliminary program:

- long-term oven aging (LTOA) of compacted specimens in a forced draft oven (FDO) at 107°C (224°F);

- oxidation of compacted specimens in a pressure oxidation vessel (POV) with oxygen or air at 0.7 MPa (100 psi) or 2.1 MPa (300 psi), and 25° or 60°C (77° or 140°F); and,

- low pressure oxidation (LPO) of compacted specimens in a triaxial cell by passing oxygen or air through the specimen at 25° or 60°C (77° or 140°F).

A statistically designed experiment was used to evaluate each aging method. Compacted specimens 64 mm (2.5 in.) high by 100 mm (4 in.) diameter were prepared at target air void contents of 4 percent and 8 percent. Aging was conducted for several periods of time to develop a curve of aging versus time for each method evaluated.

Extended mixing was rejected as a short-term procedure because it was not possible to develop a low cost—high productivity approach. Pressure oxidation for long-term aging was rejected because of safety concerns and problems with sample disruption due to the high pressures used. However, this approach was reevaluated in the supplementary program, using 0.7 MPa (100 psi) oxygen pressure.

The supplementary test program evaluated short- and long-term aging combinations:

- no short-term aging, plus LTOA for 0, 2, and 7 days at 85°C (185°F);

- STOA for 4 hours at 135°C (275°F), plus LTOA for 0, 2, and 7 days at 85°C (185°F);

- STOA for 4 hours at 135°C (275°F), plus LTO for 0, 2, and 7 days at 60°C (140°F);

- STOA for 4 hours at 135°C (275°F), plus LPO for 0, 2, and 7 days at 85°C (185°F); and,

- STOA for 4 hours at 135°C (275°F), plus POV for 0, 2, and 7 days at 0.7 MPa (100 psi) and 60°C (140°F).

The same asphalt-aggregate mixes and specimen size used in the preliminary program were used for each of the aging combinations given above. Only the target air void content of 8 percent was used. Occasionally a period of five days was used instead of seven days.

6.2.2 Materials and Specimen Preparation

As shown in Table 6.1 two asphalts (AAG-1 and AAK-1) and two aggregates (RB and RL) were used in this phase. Kneading compaction was used to compact all specimens for testing after STOA described earlier.

6.2.3 Evaluation of Mix Aging

Resilient modulus and indirect tension tests were used to evaluate the effect of each aging method. The resilient modulus was determined at 25°C (77°F) in the diametral mode with a time of loading of 0.1 sec. and a frequency of 1 Hz. A constant strain level of 0.001 percent was maintained throughout the test. The indirect tension test was performed at the same temperature at a loading rate of 50 mm (2 in.) per min after the modulus test had been completed. Some tests were also conducted on the original and recovered asphalts. The resulting data are included in Bell et al., 1993a.

Table 6.1. List of materials used for all programs

Test Program	Aggregate		Asphalt	
	Code	Description	Code	Grade
Preliminary/Supplementary	RB	Granite	AAG-1	AR-4000
	RL	Chert/gravel	AAK-1	AC-30
Expanded	RB	Granite	AAA-1	150/200 pen
	RC	Limestone (high absorption)	AAB-1	AC-10
	RD	Limestone (low absorption)	AAC-1	AC-8
	RH	Greywacke	AAD-1	AR-4000
	RJ	Conglomerate	AAF-1	AC-20
	RL	Chert/gravel	AAG-1	AR-4000
			AAK-1	AC-30
			AAM-1	AC-20

6.2.4 Test Results

Results for STOA at 135°C (275°F) from the preliminary program are shown in Figure 6.1. The results show that mixes containing asphalt AAG-1 and aggregate RL underwent the

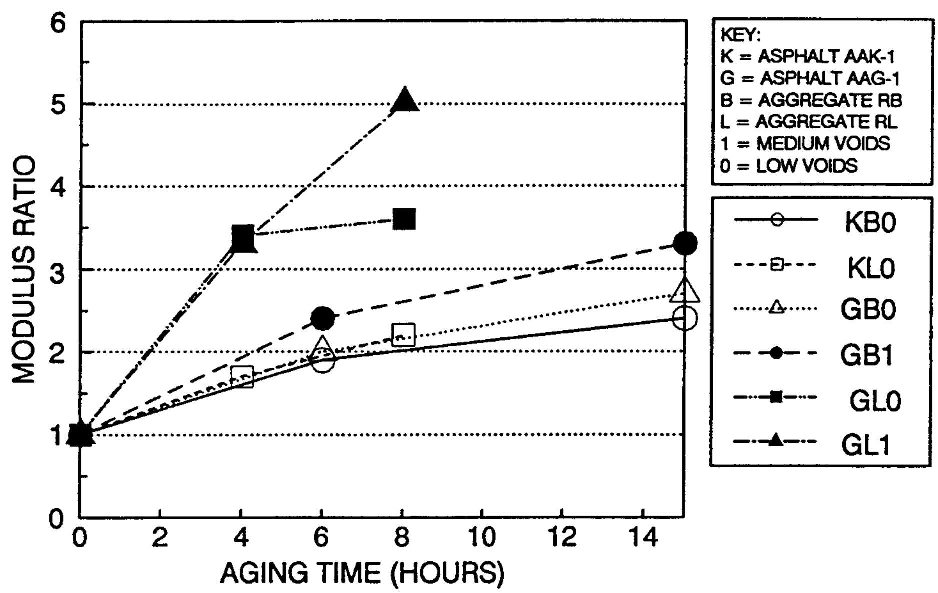


Figure 6.1. Effect of aging time on modulus for short-term aging at 135°C (275°F)

greatest changes in moduli. This result was unexpected in that the results of the Thin Film Oven Test (TFOT) indicated AAG-1 would age less than AAK-1.

Selected results of LTOA at a temperature of 107°C (224°F) are shown in Figure 6.2, with those for a temperature of 85°C (185°F) done in the supplementary program. These specimens were not subjected to short-term aging. All LTOA specimens were compacted and then subjected to a 2-day preconditioning period at 40° or 60°C (104° or 140°F) prior to the LTOA to ensure they were stiff enough to remain intact during the high temperature treatment. The data for the 107°C (224°F) series indicate that mixes using aggregate RL tended to change most. This was consistent with the 135°C (275°F) STOA. The change in modulus at seven days in the 107°C (224°F) tests was judged to be excessive and this appeared to be supported by the results of gel permeation chromatography tests on recovered asphalts.

Results of the supplementary test program are shown in Figures 6.3, 6.4, and 6.5. This test program required that six specimens be prepared in an unaged condition for each of the four mix combinations. The average modulus for each combination was then used as a datum to compare with the aged specimens such as in Figure 6.3. Also, with the exception of the first series, each of the test series generated three short-term oven aged specimens (resulting in a total of 12) for each mix combination. All of these specimens were prepared to the same target air void content of 8 percent. Although this was usually achieved within a tolerance of ±1 percent, variations in modulus occurred with "replicate" specimens because of the range of air void contents and test variability.

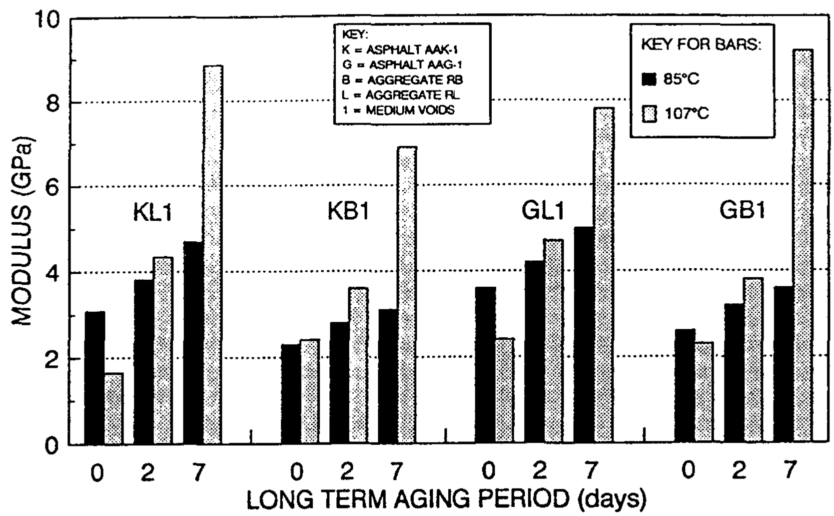


Figure 6.2. Long-term oven aging with no prior short-term aging

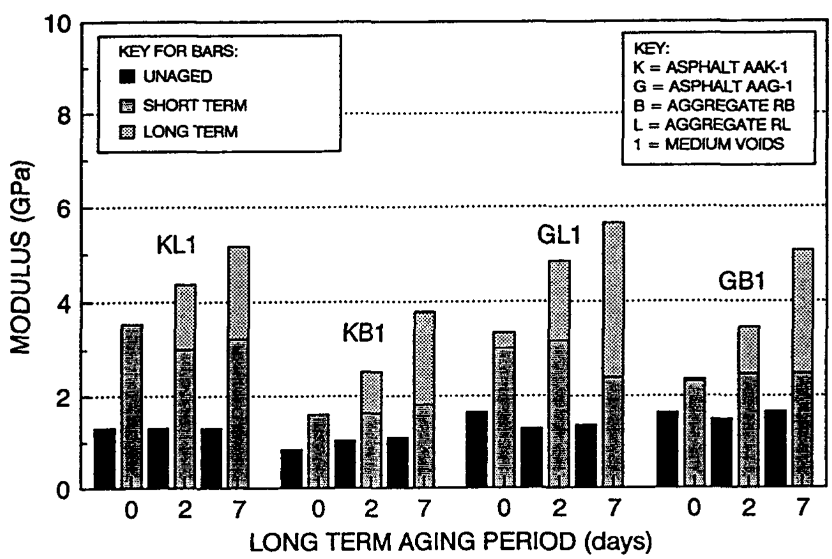


Figure 6.3. Long-term oven aging at 85°C (185°F) with prior short-term aging

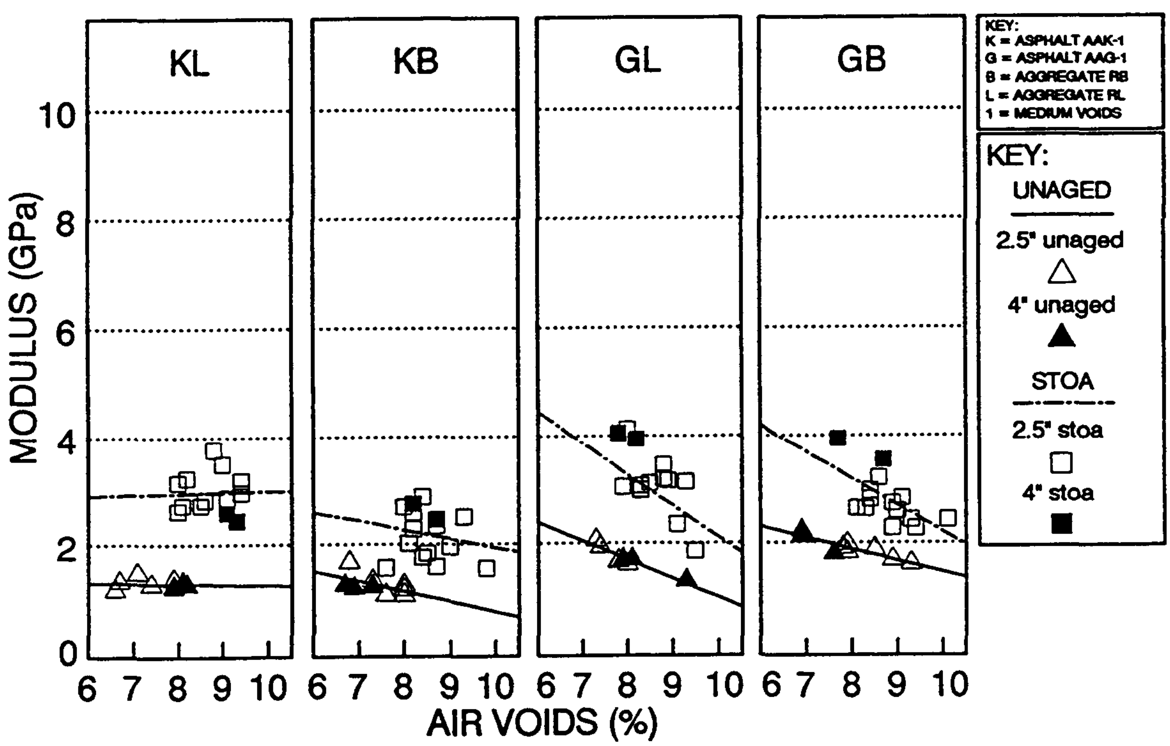
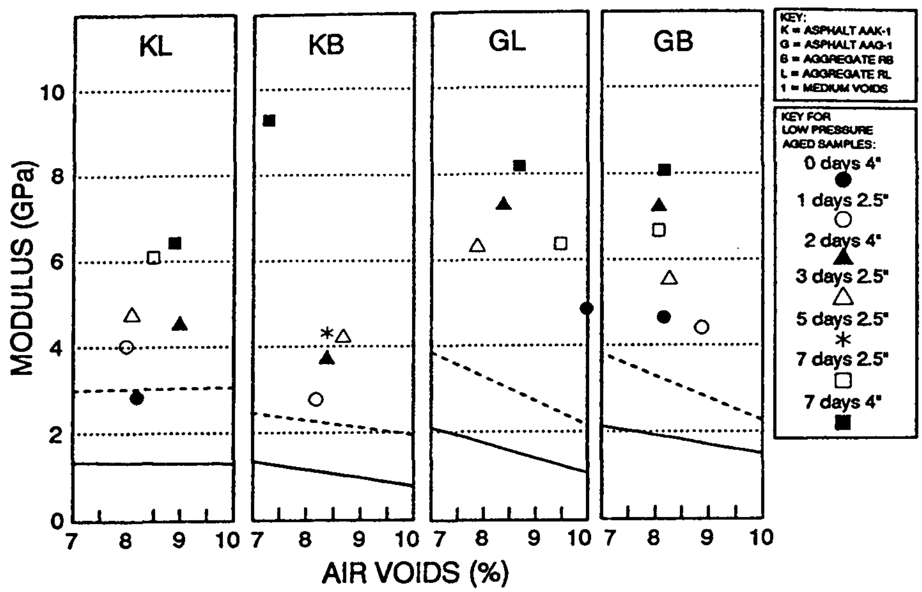
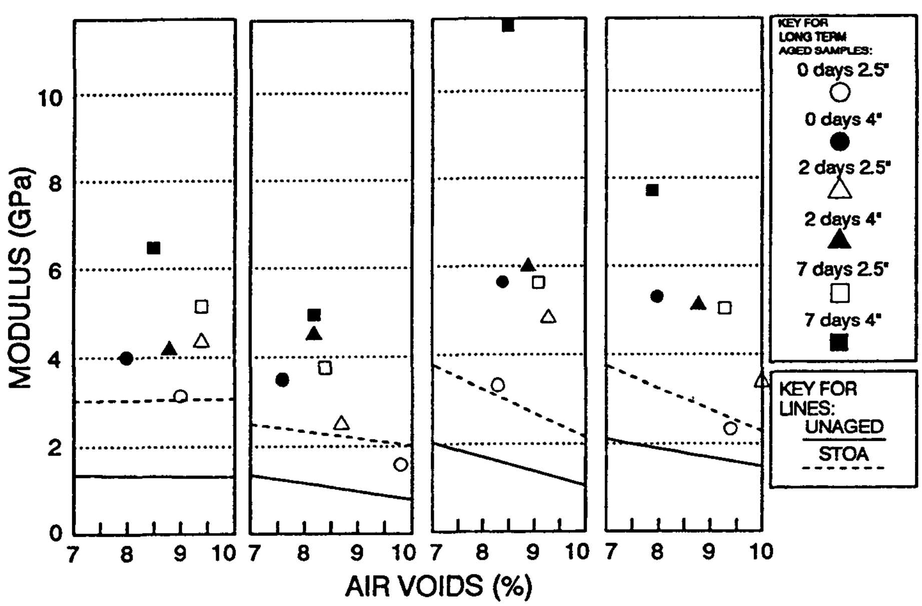


Figure 6.4. Unaged and short-term oven aged results



a) Low Pressure Oxidation at 85°C with Prior Short Term Oven Aging (STOA).



b) Long Term Oven Aging at 85°C with Prior Short Term Oven Aging (STOA).

Figure 6.5. Summary of aging data

The unaged and STOA diametral modulus data from the supplementary test program [64 mm (2.5 in.) high specimens] are shown in Figure 6.4. This figure also includes data from the expanded test program [100 mm (4 in.) high specimens]. Linear regression lines are shown for each group of data. These help to illustrate the effect of the aggregate on short-term aging noted earlier for each of the test series, i.e., that for either asphalt, aggregate RB shows less aging and the effect is more pronounced with asphalt AAK-1.

Work by the SHRP A-002A contractor using sand-size aggregate mixes with asphalt appears to confirm that aggregate has an effect on aging. The A-002A researchers suggest that the aggregate may inhibit oxidation by causing molecular structuring in the asphalt. The extent to which this occurs depends on the asphalt and aggregate. In this study it appears that asphalt AAK-1 structures more than asphalt AAG-1, and that aggregate RB promotes structuring more than aggregate RL, thus inhibiting aging.

Figure 6.5 shows the LTOA and LPO data at 85°C (185°F) and the regression lines for unaged and short-term aged specimens (shown in Figure 6.4). Figure 6.5 gives a general impression of the effects of the long-term aging method and time period relative to the unaged and STOA data. In general, Figure 6.5 shows that a seven-day, long-term aging period will double the modulus achieved after STOA. The figure also shows that the LPO approach is more severe than the LTOA approach, since ten of the modulus values are above 6 MPa for the LPO approach but only three are above 6 MPa for the LTOA approach.

6.3 Expanded Test Program

Based on the results presented in the preceding section, an expanded test program was conducted using selected short- and long-term aging procedures (Bell and Sosnovske, 1993). Materials used in the program are shown in Table 6.1.

The procedure developed for short-term aging involves heating the loose mix in a forced draft oven for 4 hours at a temperature of 135°C (275°F). This simulates the aging of the mix during the construction process while it is in the uncompacted condition.

Two alternate procedures have been developed for long-term aging of the compacted mix. These are designed to simulate the aging of in-service pavements after several years. The following long-term approaches have been found to be appropriate:

1. Long-term oven aging (LTOA) of compacted specimens in a forced draft oven.

2. Low pressure oxidation (LPO) of compacted specimens in a triaxial cell by passing oxygen through the specimen.

With these two methods of aging, alternate combinations of temperature and time were evaluated, including low-pressure oxidation (LPO) 60° and 85°C (140° and 185°F), long-term oven aging (LTOA) at 85°C (185°F), all for 5 days, and LTOA at 100°C (212°F) for 2 days.

The effects of aging were evaluated by resilient modulus measurements at 25°C (77°F) using both the diametral (indirect tension) and triaxial compression modes of testing. Tensile strength determinations were also performed at the conclusion of the modulus tests.

From the results of the expanded study the following conclusions have been made:

1. The aging of asphalt-aggregate mixes is influenced by both the asphalt and aggregate. Aging of the asphalt alone and subsequent testing does not appear to be an adequate means of predicting mix performance because of the apparent mitigating effect aggregate has on aging.

2. The aging of certain asphalts is strongly mitigated by some aggregates but not by others. This appears to be related to the strength of the chemical bonding (adhesion) between the asphalt and aggregate.

3. The short-term aging procedure produces a two-fold change in resilient modulus. For a particular aggregate, there is not a statistically significant difference in the aging of certain asphalts. The eight asphalts investigated typically fell into three groups, i.e., those with high, medium, and low susceptibility to aging.

4. The four long-term aging methods produce somewhat different rankings of aging susceptibility compared to the short-term aging procedure and to each other. This is partially attributable to variability in the materials, aging process, and testing. However, it appears that the short-term aging procedure does not enable prediction of long-term aging.

5. The low pressure oxidation procedure for long-term aging causes the most aging, and less variability in the rankings of aging susceptibility relative to the short-term aging ranking.

6.4 Field Validation

Field validation was conducted using the procedure shown in Figure 6.6. Short-term oven aging (STOA) at 135°C (275°F) and long-term oven aging (LTOA) at 85° and 100°C (185° and 212°F) were the procedures evaluated in this program.

Following site selection and material gathering, cores from the field were trimmed and analyzed to determine their air void levels. Whenever possible, the asphalt content and aggregate gradation, determined by extractions from prior studies, were retrieved for use in this study. **Laboratory specimens were prepared to the field gradations, and asphalt contents and the target air voids were determined from the field cores.** Laboratory specimens were subjected to varied aging treatments and both field and laboratory mixes were tested for resilient modulus. The results of these tests were compared to evaluate the effectiveness of the aging treatments to simulate stiffening of the mixes in the field.

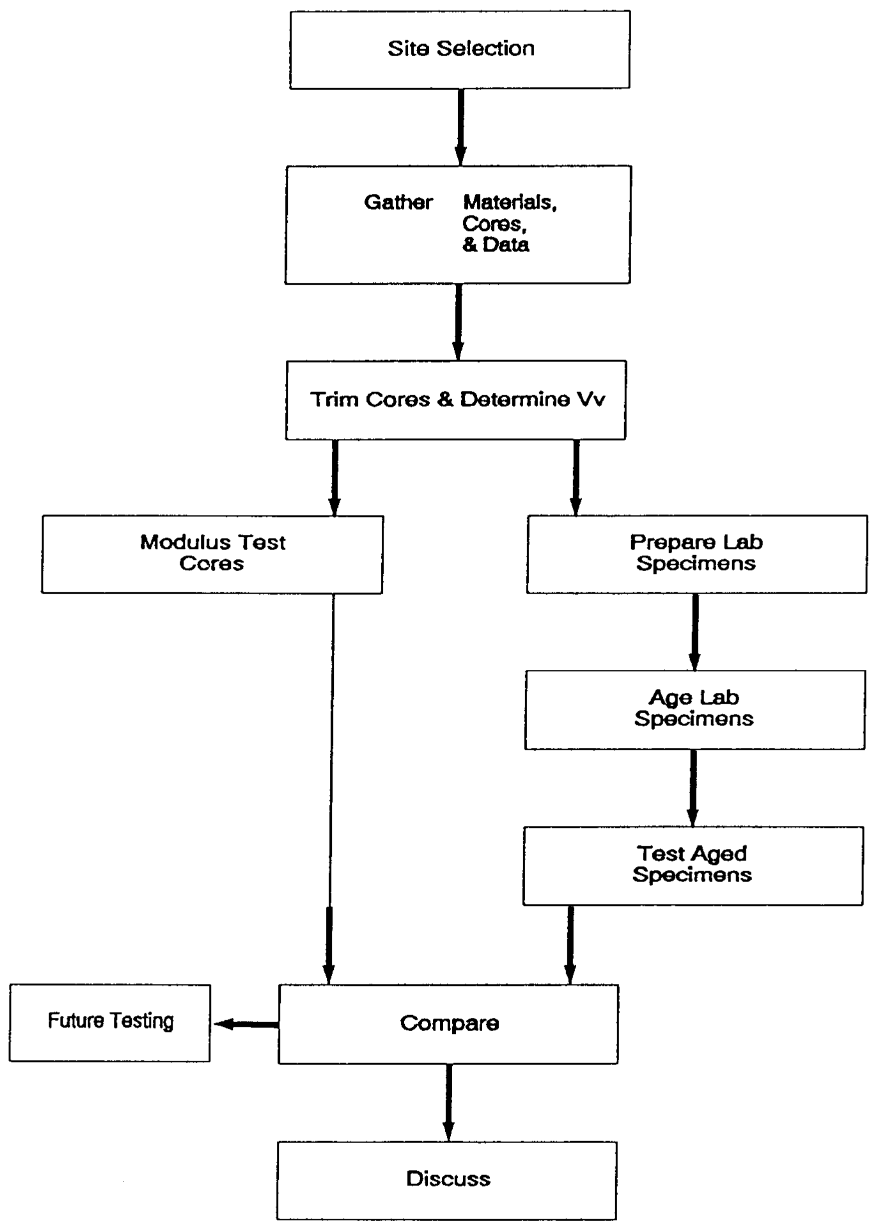


Figure 6.6. Study approach — expanded validation study

Materials were gathered from 20 different projects to enable new, young, and old pavements to be represented. The details of this program are described by Bell, et al. (1992). Table 6.2 summarizes some basic data for each site.

The four new sites, located in Oregon, were used to validate STOA. Laboratory mixes were subjected to 0 h, 4 h, and 8 h of STOA at 135°C (275°F) prior to compaction. The modulus of compacted laboratory mixes was compared with that obtained for field cores.

The nine young projects and seven old projects were all subjected to four hours of STOA at 135°C (275°F). Long-term oven aging was then conducted at either 85°C (185°F) for 0, 2, 4, or 8 days, or at 100°C (212°F) for 0, 1, 2, and 4 days. Only the data for STOA and LTOA at 85°C (185°F) will be reported herein.

6.4.1 Aging Methods

6.4.1.1 No Aging

Laboratory specimens were prepared at the time of mixing to represent an "unaged" condition. These specimens were prepared in the same manner as the others except that they were not cured for 4 hours at 135°C (275°F). As soon as mixing was complete the specimens were placed in an oven and brought to the proper equiviscous temperature (665 ± 80 centistokes) for that mix. Once the proper temperature was achieved the specimens were compacted using a California Kneading Compactor.

6.4.1.2 Short-Term Aging

Another set of specimens was subjected to short-term aging in a forced draft oven at 135°C (275°F) prior to compaction. This was usually for four hours, but for validation of the STOA procedure, a period of 8 hours was also used. During the curing period the mix was placed in a pan at a spread rate of approximately 21 kg per square meter. The mix was also stirred and turned once an hour to ensure that the aging would be uniform throughout the sample. After the curing period the samples were brought to an equiviscous temperature of 665 ± 80 centistokes and compacted using a California Kneading Compactor.

6.4.1.3 Long-Term Oven Aging

For validation of long-term aging, sets of specimens were subjected to long-term oven aging. The procedure is carried out on compacted specimens after they have been short-term aged. The specimens were placed in a forced draft oven, preheated to 85°C (275°F), and left for five days. Alternatively, a temperature of 100°C (212°F) and 2 days was used. After the aging period, the oven was turned off and left to cool to room temperature. The specimens were then removed from the oven and prepared to be tested at least 24 hours after removal

Table 6.2. Summary of field validation sites

a. new projects

Site and Project Number	Construction Date	Asphalt	% AC[a]	Admix	Climate
Stag Hollow - Wapato Road, #913		AC-15	6.2	None	Wet-Freeze
Butter Creek - Old Oregon Trail, #816		AC-15	5.9	None	Dry-Freeze
Rock Creek - Aniauf, #852		AC-20	5.3	PBS	Wet-No-Freeze
Lobert, #874		AC-15	5.8	Lime	Dry-Freeze

[a]By weight of total mix

b. young projects

Site and Project Number	Construction Date	Asphalt	Admix	Climate
Arizona SPS-5 (AZ5)	1990	AC-40	Type II Portland Cement	Dry-No-Freeze
Arizona SPS-6 (AZ6)	1990	AC-20	Lime	Dry-Freeze
California AAMAS Batch (CAB)	1989	AR-4000	-	Dry-Freeze
California AAMAS Drum (CAD)	1989	AR-4000	-	Dry-Freeze
California GPS- (CAG)	1991		-	Dry-No-Freeze
French A Section	1986	40/50 Pen	-	Dry-No-Freeze
French B Section	1986	40/50 Pen	-	Dry-No-Freeze
French C Section	1986	40/50 Pen	-	Dry-No-Freeze
Georgia AAMAS (GAA)	1989	AC-30	Lime	Wet-No-Freeze
Michigan SPS-6 (MI6)	1990	AC-10	Flyash	Wet-Freeze
Minnesota SPS-6 (MN6)	1990		-	Wet-Freeze
Wisconsin AAMAS (WIA)	1989	200/300 Pen	Recycle	Wet-Freeze

Table 6.2. Summary of field validation sites (continued)

Site and Project Number	Construction Date	Asphalt	Admix	Climate
SR-14, #1801	1973	85/100 Pen	None	Wet-No-Freeze
SR-522, #6048	1977	AR 4000W	None	Wet-No-Freeze
SR-167, #6049	1972	85/100	None	Wet-No-Freeze
SR-12, #1002	1988	AR 4000W	None	Dry-Freeze
US 97, #1006	1982	AR 4000W	PBS	Dry-Freeze
US 195, #1008	1978	AR 4000W	PBS	Dry-Freeze
US 195, #6056	1986	AR 4000W	PBS	Dry-Freeze

from the oven. In this program, LTOA at 85°C (185°F) was conducted for 0, 2, 4, and 8 days. For LTOA at 100°C (212°F), this program used 0, 1, 2, and 4 days of aging.

6.4.2 Evaluation Methods

The resilient modulus was determined at 25°C (77°F) using the diametral (indirect tension) (ASTM D 4123) and triaxial compression modes of testing with a 0.1 second loading time at a frequency of 1 Hz. A constant strain level of 100 microstrain was maintained throughout the test.

Less variability was experienced with the diametral modulus data; approximately ± 10% versus ±15% with the triaxial modulus data. This difference was attributed partially to the relatively short specimen used [10 cm (4 in.)] in the triaxial mode. Details of the data for both diametral and triaxial modes of testing have been presented by Bell et al. (1992). For the field validation program many of the field cores were of insufficient length to enable triaxial modulus to be determined.

6.4.3 Results

6.4.3.1 New Projects

Figure 6.7 shows a typical result from this phase of the field validation. The intersection of the value of modulus for the field material with the laboratory data gives the estimate of amount of STOA needed to represent short-term aging in the field. Similar data were obtained for each of the four new projects. A STOA procedure of 4 hours was selected for future use. This is consistent with that recommended by Von Quintus et al. (1991), and appears to be conservative.

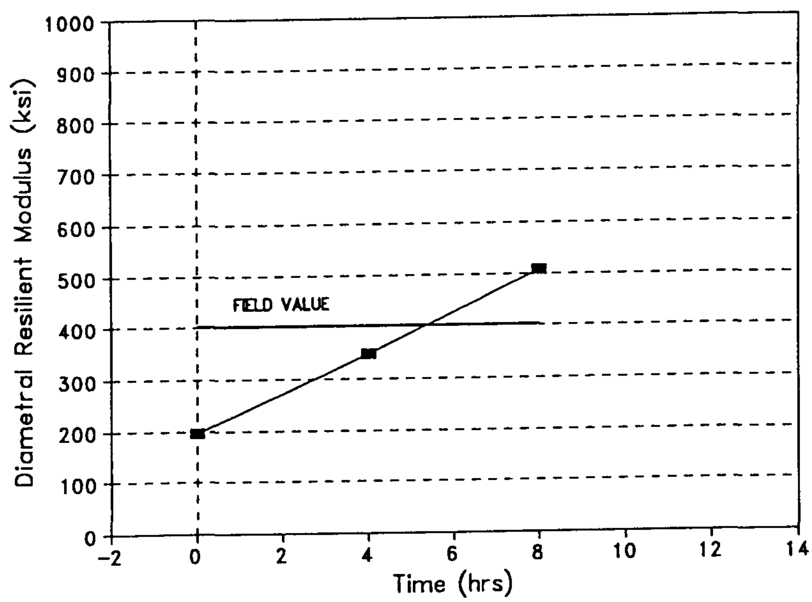


Figure 6.7. Short-term aging at 135°C (275°F) — ODOT #816

6.4.3.2 Young Projects

A statistical analysis of the moduli for field cores and laboratory specimens was done to determine which of the laboratory treatments most closely matched the field aging for each site. Both Tukey and LSD multiple comparison approaches were used. Analysis was favored, since it produces tighter confidence intervals, allowing differences to be better detected. The LSD method is commonly used when planned comparisons are made, as in this study, with several treatments being compared to the field. Only examples and summaries of the LSD analyses are presented here.

Figures 6.8 and 6.9 illustrate the LSD statistical analyses performed on the young and old projects respectively. The vertical lines plotted are the confidence intervals for each set of modulus values. Any aging treatment with a confidence interval including any of the field confidence interval is considered to be statistically similar to the field. Tables 6.3 and 6.4 summarize the results for young and old sites respectively for LTOA at 85° and 100°C (185° and 212°F).

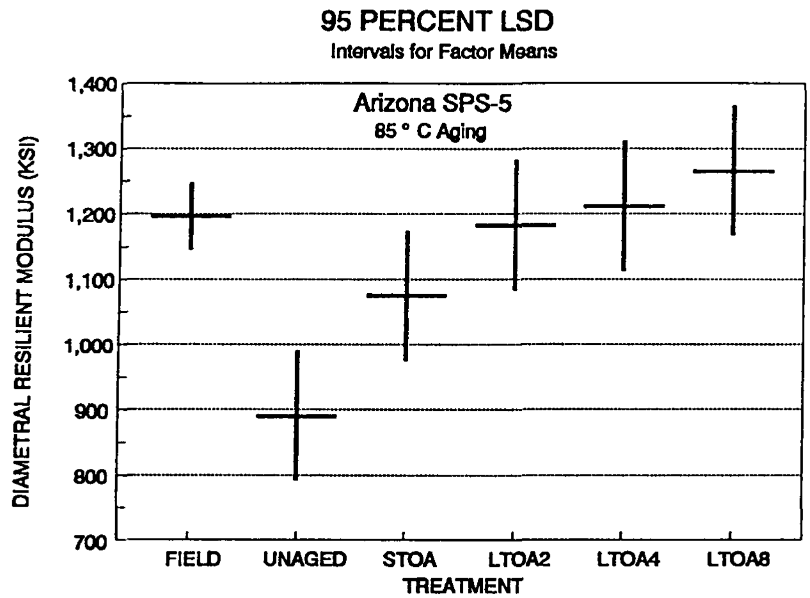


Figure 6.8. Arizona SPS-5 LSD comparison, LTOA at 85°C (185°F)

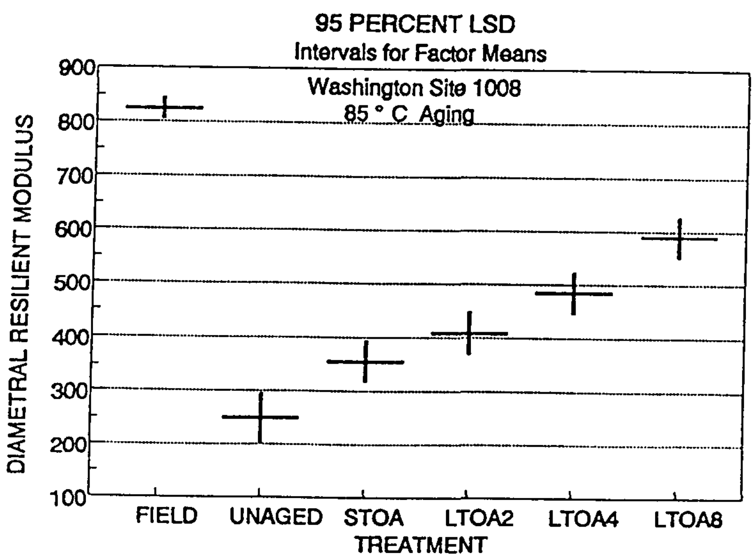


Figure 6.9. Washington site 1008 LSD comparison, LTOA at 85°C (185°F)

Table 6.3. Field validation study — aging treatments means "not significantly different" from field means, young projects

Site	Age	LSD (85°C, 185°F)	LSD (100°C, 212°F)
Arizona SPS-5 (AZ5)	6 months	ALL	LTOA 1, 2, 4
Arizona SPS-6 (AZ6)	Few months	Unaged, STOA	Unaged, STOA
California AAMAS Combined (CAC)	Over 2 years	Unaged	Unaged
California GPS-6 (CAG)	Few months	STOA	STOA
French A, B & C Combined (FRC)	5 years	LTOA 8	None
Georgia AAMAS (GAA)	2 years	None	None
Michigan SPS-6 (MI6)	6 months	STOA	STOA, LTOA 1, 2
Minnesota SPS-6 (MN6)	1½ years	Unaged, STOA, LTOA	Unaged, STOA, LTOA 1

Key: All = all of the aging treatments are not significantly different from the field.
None = all of the aging treatments are significantly different from the field mean.
Mean = average of Diametral Resilient Modulus Tests.

Table 6.4. Field validation study — lab modulus means "not significantly different" from field means, old projects

Site	Age (years)	LSD (85°C, 185°F)	LSD (100°C, 212°F)
1801	18	LTOA 8	None
6048	14	STOA, LTOA 2, 4, 8	STOA, LTOA 1*, 2, 4
6049	19	LTOA 4, 8	LTOA 4
1002	3	STOA	STOA
1006	9	None	Not tested
1008	13	None	None
6056	5	LTOA 2, 4	LTOA 1*, 2

Key: All = all of the aging treatments are not significantly different from the field.
None = all of the aging treatments are significantly different from the field mean.
Mean = average of Diametral Resilient Modulus Tests.

6.4.4 *Analysis of Results*

6.4.4.1 Field Validation Program

Figures 6.10, 6.11, and 6.12 are summaries of the statistical analysis for young and old projects. They show aging treatments which were similar to the field, and also show where the field average modulus lies relative to the average modulus for each of the aging treatments. The age of each site is also noted, and the sites are grouped according to climatic region. Since the three French sections utilized the same asphalt grade (but different suppliers) and were subjected to the same traffic and environmental conditions, the data from those sites were combined. The same was done for the California AAMAS sites, where both drum and batch plants were used.

Figures 6.10 and 6.11 indicate and Table 6.3 shows that, for the young projects, five of the seven sites between 0 and 2 years old had field means that were statistically similar to the STOA aging treatment. Two of the five also had field moduli similar to unaged specimens. The two sites that were not similar to STOA, Georgia and California AAMAS, had field cores which came from weak or damaged pavements. This resulted in field moduli equal to or lower than the unaged specimens. This data further validated the preliminary study conclusion that 4 hours of STOA at 135°C (275°F) is a good estimate of short-term aging since young projects exhibited similar levels of aging. The two older expanded sites, Wisconsin (3 years), and France (5 years), required short-term and long-term oven aging to match the field modulus average.

Figure 6.12 and Table 6.4 show that five of the old sites required at least 8 days of long-term oven aging to match the field modulus mean. Sites 1006 and 1008, 9 and 13 years old sites in a dry-freeze portion of Washington, had field moduli that were significantly higher than any of the aging treatments. The youngest site 1002 (3 years) required only STOA to match the field, while the 5-year old site 6056 had a field modulus similar to 2 and 4 days of 85°C (185°F) aging.

6.5 Mix Design and Analysis

In the mix design and analysis system two forms of mix or specimen conditioning are utilized: *aging* and *hot water*, plus *repetitive loading*. The role of aging is briefly discussed in this section.

Two levels of aging are used. A *short-term oven aging* (STOA) procedure has been developed to represent the initial conditions of the mix in the pavement. This procedure consists of curing the loose mix for four hours at 135°C (275°F) in a forced draft oven and may be representative of the initial conditions up to one year depending on the severity (temperature regime) of the climate at the site.

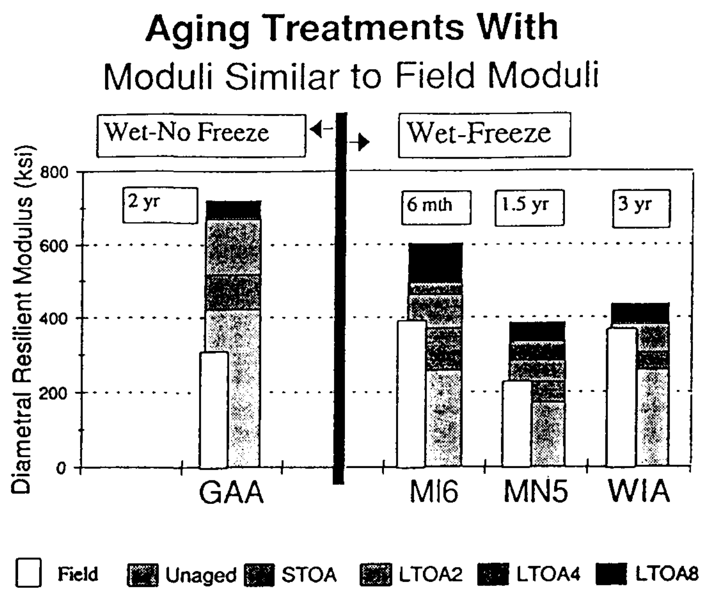


Figure 6.10. Field validation — expanded study modulus comparisons, Wet-Freeze/No-Freeze zones, young projects

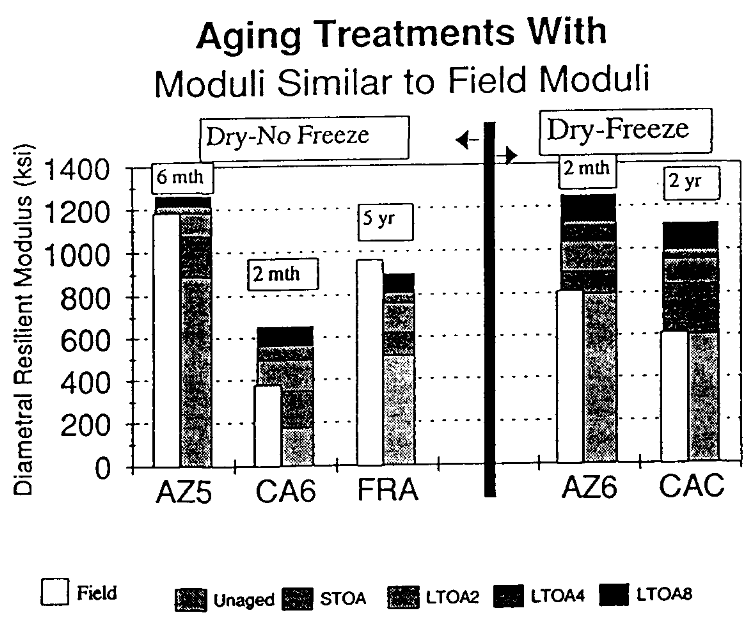


Figure 6.11. Field validation — expanded study modulus comparisons, Dry-Freeze/No-Freeze zones, young projects

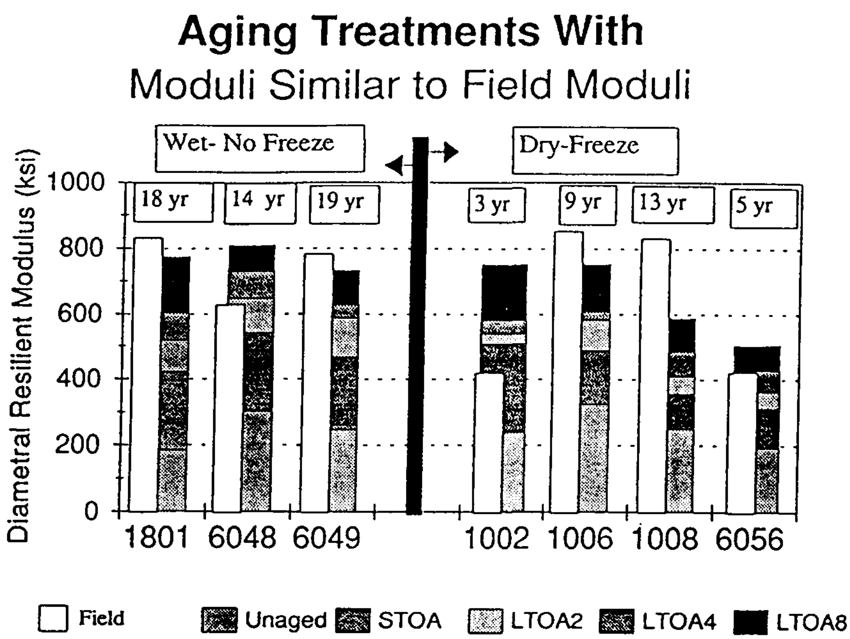


Figure 6.12. Field validation — supplementary study comparisons, Dry-Freeze/Wet-No-Freeze zones, old projects

The second procedure termed *long-term aging* can be accomplished in two ways: 1) conditioning the prepared test specimen for 5 days at 85°C (185°F), termed long-term oven aging (LTOA); or 2) subjecting the specimen to low pressure oxygen for 5 days at 60°C (140°F). The LTOA procedure is preferred and is recommended for dense-graded mixes containing both conventional asphalts and modified binders. For open-graded mixes and dense-graded mixes containing soft binders, the low pressure oxygen aging is recommended since confinement is provided to the specimen during the aging process. Table 6.5 contains the recommended aging procedure for specimens to be tested for fatigue, permanent deformation, thermal cracking, and water sensitivity.

6.6 Summary

The following conclusions can be drawn from the results of this study:

1. The aging of asphalt-aggregate mixes is influenced by both the asphalt and aggregate. Aging of the asphalt alone, and subsequent testing does not appear to be an adequate means of predicting mix performance because of the apparent mitigating effect aggregate has on aging.

2. The aging of certain asphalts is strongly mitigated by some aggregates but not by others. This appears to be related to the strength of the chemical bond (adhesion) between the asphalt and aggregate.

3. The short-term aging procedure produces a change in resilient modulus of nearly a factor of two. For a particular aggregate, there is not a statistically significant difference in the aging of certain asphalts. The eight asphalts investigated typically fell into three groups, i.e., those with high, medium, and low aging susceptibility.

4. Based on the study of new and young field sites, 4 hours of oven aging at 135°C (275°F) appears representative of the short-term aging which occurs in the field during mixing and placement. This is also sufficient for field aging of young projects less than two years old.

5. Two days of long-term oven aging at 85°C (185°F) is representative of pavements up to five years old depending on the climate.

6. Four days of oven aging at 85°C (185°F) appears to be representative of field aging of about 15 years in a Wet-No-Freeze zone and about 7 years in a Dry-Freeze zone.

7. It was not possible to develop guidelines for Wet-Freeze or Dry-No-Freeze zones, because no projects of sufficient age could be located.

8. Oven aging at 100°C (212°F) for 1, 2, and 4 days achieves similar stiffness to 85°C (185°F) aging for 2, 4, and 8 days, but damages the specimens in the process; therefore, 85°C (185°F) aging is recommended.

Table 6.5. Recommended aging procedures for test specimens to be evaluated in specific distress modes

Mode of Distress	Level 1	Level 2	Level 3
Fatigue	Short-term oven aging	Short-term oven aging	Short- and long-term oven aging
Permanent deformation	Short-term oven aging	Short- and long-term oven aging	[a]
Thermal cracking	No mix testing	Long-term oven aging	Short- and long-term oven aging
Water sensitivity	Short-term oven aging	Short-term oven aging	Short-term oven aging

[a]No Level 3 procedure since Level 2 is the comprehensive procedure.

From the results of this study, the following recommendations are made:

1. To further analyze the effectiveness of the short-term aging period of 4 hours, additional sites should be selected. Of the agencies contacted in searching for retained materials, few indicated the use of diametral resilient modulus for testing newly laid pavements.

2. Continued monitoring of field projects is needed, particularly for Dry-No-Freeze and Wet-No-Freeze zones. Increasing the number of sites and the total number of specimens prepared will facilitate the use of regression analysis to develop prediction models. The sites selected should have in-service lives ranging from 1 to 20 or more years to encompass all long-term aging in the field. A reduction in the 95 percent confidence intervals found with the LSD analyses would improve the correlation of the laboratory procedures with the age of the field cores.

3. The field validation study addressed validation of the 4 hours at 135°C (275°F) STOA and LTOA at 85° and 100°C (185° and 212°F). One additional method for long-term aging was developed at OSU and deserves additional validation study — low pressure oxidation (LPO) at 85°C (212°F). This approach may be necessary for mixes with relatively low modulus. The aging effects of LPO have been evaluated in the development of alternative laboratory aging procedures by Bell and Sosnovske (1993). The pressures involved are not high enough to pose safety problems associated with those of the high pressure oxidation procedures studies earlier.

7

Accelerated Performance-Related Tests to Evaluate Water Sensitivity of Asphalt-Aggregate Mixes

7.1 Introduction

This chapter discusses the development of an accelerated performance-related test to define the water sensitivity characteristics of asphalt-aggregate mixes and its use in mix design and analysis. Background information is presented which led to the development of the environmental conditioning system (ECS) to define the effects of moisture and water[13] on mix response. A detailed description of the ECS as developed at Oregon State University (OSU) as well as its use to define water sensitivity is included. Validation efforts for the test are summarized and provide the basis for the selection of criteria which can be used in the mix design and analysis framework recommended herein.

7.2 Literature Evaluation and Hypotheses

Two key mechanisms contribute to the deterioration of asphalt-aggregate mixes by water: 1) reduction in adhesion and stiffness of the asphalt binder matrix; and 2) loss of adhesion between the asphalt binder and the aggregate surface. Both mechanisms may lead to premature pavement distress (Terrel and Shute, 1989).

Early in the SHRP research, it was hypothesized that considerable water damage might occur primarily because of the air void system (Terrel and Al-Swailmi 1993b). Very low and very high void levels tend to have less damage; water cannot access low void mixes, and water flows through high void mixes, rather than being retained. The mid-range of voids, termed

[13]The terms moisture and water are often used interchangeably, but there is a difference between the actions of moisture vapor and liquid water on distress mechanisms such as stripping.

pessimum voids tends to cause the most damage because water gets into the mix and stays there, resulting in long-term damage. Test results showing this concept are also included.

Numerous methods have been developed to determine if an asphalt-concrete mix is sensitive to water and, therefore, is prone to early water damage. In general, there are two categories into which the tests can be divided:

1. Tests which coat "standard" aggregate with an asphalt cement (with or without admixes). The loose uncompacted mix is then immersed in water (which is either held at room temperature or boiled) for a specified period of time. A visual assessment of the amount of stripping is made.

2. Tests which use compacted specimens; either laboratory compacted or cores from existing pavement structures. These specimens are conditioned in some manner to simulate in-service conditions of the pavement structure. The results of these tests are generally evaluated by the ratios of conditioned to unconditioned results of a stiffness or strength test (e.g., diametral or triaxial resilient modulus tests, indirect tensile strength test).

The use of terms such as reasonable, good, and fair are often used in conjunction with the description of how well the results of a test correlate with actual field performance. Stuart (1986) and Parker and Wilson (1986), found that, for the tests they evaluated, a single pass/fail criterion could not be established that would enable the results of the tests to correctly indicate whether or not the asphalt mixes they tested were water sensitive. These results are characteristic of all test methods currently used to assess asphalt-concrete mixes for water sensitivity.

From a review of the literature, several tests have received the most attention and cover the variety of methods used to evaluate water sensitivity. The test designations and an evaluation of each method are summarized in Table 7.1. From the data and experience to date, it appears that a test has yet to be established that is highly accurate in predicting water susceptible mixes and estimating the life of the pavement.

Some tests, as noted in Table 7.1, have demonstrated a "good" correlation with field performance. However, it is not uncommon to find mixed reviews as to the effectiveness of a procedure. There is little evidence of laboratory testing followed by evaluation of field performance to establish a direct correlation. Most "field correlation data" is established from obtaining "approximately the same materials" placed in the field, then testing these materials to establish a correlation (Stuart 1986; Tunnicliff and Root 1984) or using testing procedures to evaluate the performance where the water sensitivity of the material (primarily aggregate) is "known."

Based on this evaluation the decision was made by the A-003A team to develop a new test methodology to evaluate the effects of water mix performance. It was concluded that the method should consider the effects of traffic loading and potential freeze-thaw effects as well.

Table 7.1. Evaluation of water sensitivity test methodologies

Method	Reference	Application of Test Results	Advantages	Limitations	Simulation of Field Conditions	Ease of Use
1. Lottman	NCHRP 246	• M_R • St ratio (diametral)	• Severe test • Wide range of mixes and cores • Good for lime or liquid additives	• Time consuming (3-day cycle) • Equipment is expensive and many not be readily available	• Good correlation with field performance • Simulates freeze-thaw conditions	• Moderately complex
2. Tunnicliff-Root	NCHRP 274	• Diametral St • Visual rating	• Wide range of mixes, cores • Good for additives • Moderately time consuming	• Requires trial mixes to obtain air void level • May not be severe enough	• Initial use shows good correlation with field performance	• Moderately complex
3. Boiling	ASTM 3625	• Stripping potential • Visual rating	• Initial screening • Simple equipment • Lab or field mix • OK for additives	• Subjective analysis • Loose mix only • Water purity has effect	• May indicate potential for stripping	• Simple
4. Texas Freeze-Thaw Pedestal	Kennedy (1983)	• Cracking after number of freeze-thaw cycles indicates degree of moisture susceptibility	• Measures additive effectiveness • Simple	• Only fines used • Time consuming (1-day cycle) • Measures only cohesion	• Only fair correlation with field performance	• Simple, but special equipment required
5. Immersion-Compression	ASTM D 1075 AASHTO T-15	• Visual assessment • Minimum compressive strength	• Use actual mix • Simple	• Time consuming • Air voids play large role • Poor reproducibility	• Correlation not known (if any)	• Simple • Equipment should be readily available
6. Static Immersion	ASTM D 1664 AASHTO T-182	• Potential for stripping: <95% coating; no go • Visual assessment	• Simple, quick • Low cost	• Subjective evaluation • Loose mix only • Not sufficiently severe	• Short-term stripping potential only	• Simple
7. Retained Stability	No standard method	• Ratio of wet (soaked) to dry Marshall stability	• Uses conventional specimens and equipment	• No standard conditioning or criteria	• Not known	• Simple

One of the variables that affects the results of current methods of evaluation is the air voids level in the mix. The very existence of these voids as well as their characteristics can play a major role in performance. Conventional thinking would lead one to believe that voids are necessary and/or at least unavoidable. Voids in the mineral aggregate are designed to be filled with asphalt cement to a point less than full to allow for traffic compaction. But if one could design and build the pavement properly, allowing for compaction by traffic would be unnecessary. In the laboratory, mixes are usually designed at 4 percent total air voids, but actual field compaction may result in as much as 8 percent to 10 percent air voids. The voids provide the major access of water into the pavement mix.

7.3 Environmental Conditioning System (ECS)

The environmental conditioning system (ECS) was developed using the existing technology to provide an improved test methodology. Development of the ECS involved determining the most important factors in the performance of mixes in the presence of water. After the ECS test system was conceived, based on evaluation of current test systems such as the Lottman (Lottman 1971, 1978) and the Root-Tunnicliff (Tunnicliff and Root 1984), the development portion of the ECS program included two phases: 1) preliminary program to plan the basic test system and resolve several mechanical issues with the apparatus, and 2) pilot test program to determine the current test protocol.

Figure 7.1 shows the ECS and its subsystems: 1) fluid conditioning; 2) environmental conditioning cabinet; and 3) loading system. Details of each subsystem are described in Terrel et al., 1993.

7.3.1 Test Development

Preliminary experiments to investigate different proposed methods and procedures for the final ECS test procedure allowed several mechanical issues with the test apparatus to be resolved. These experiments attempted to address many of the concerns that had been raised about other water sensitivity tests, and modifications to equipment as the procedure was refined (Table 7.2). These questions dealt mainly with the mechanics of the specimen testing to determine the physical properties of the specimen, such as resilient modulus and permeability. After resolution of these issues, a pilot test program was initiated to determine the appropriate procedure for the ECS test.

7.3.2 Pilot Test Program

The pilot test program was aimed at evaluating three of the factors that most influence water sensitivity of asphalt paving mixes; temperature, permeability or air void level, and wet conditioning. Several other variables, such as conditioning cycle length, vacuum level, and

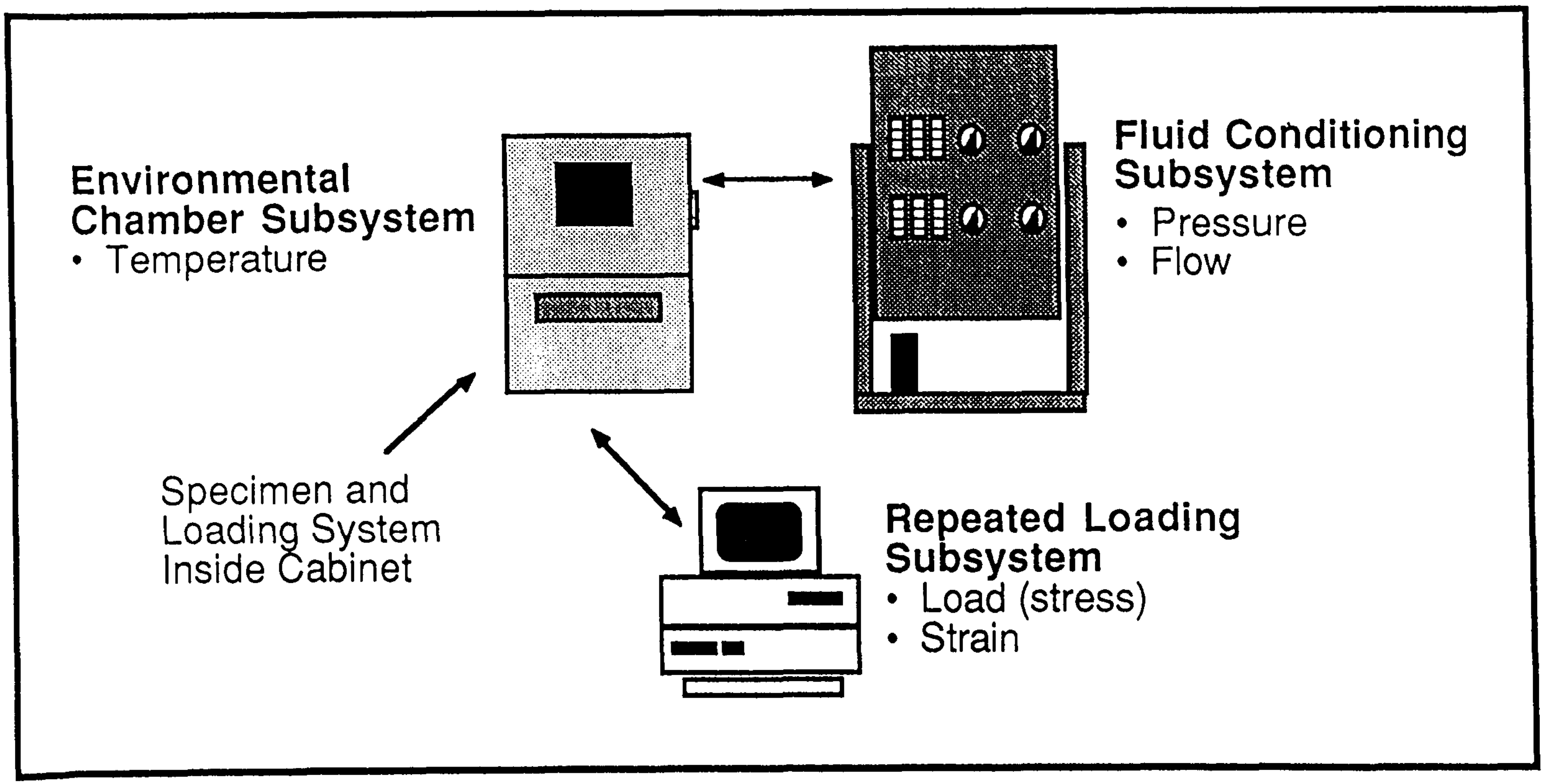


Figure 7.1. Overview of environmental conditioning system (ECS)

Table 7.2. Variables addressed during development of the ECS

Issue	Proposed	Selected Method	Rational
Strength or stiffness measurement	• Diametral modulus • Triaxial modulus • Indirect tensile strength	• Triaxial modulus ($ECS\text{-}M_R$)	• Compatible with flow system • Non-destructive
Specimen dimensions	• 20 cm (8 in.) height x 10 cm (4 in.) diameter (ASTM D 3497)	• 10 cm (4 in.) height x 10 cm (4 in.) diameter	• Typical pavement layer thickness • Reasonable flow path length • Minimize end effects
Strain measurements	• Stain gages • Linear variable differential transducers (LVDTs)	• LVDTs	• Ease in use • Reusable
Frictionless interface between specimen and loading platens		• Perforated teflon disks	• Allows water flow
Surface perimeter flow	• Use latex membrane to seal middle third of specimen • Encase specimen in 15 cm (6 in.) latex membrane	• 15 cm (6 in.) latex membrane	• Simple • Ensures no flow along specimen surface
Specimen end condition preventing flow	• Wet cut ends • Ambient dry cut ends • Cooled dry cut ends • Use specimens as manufactured by the kneading or rolling compactor	• Use specimens as manufactured by the kneading or rolling compactor • Use cooled dry cut for field specimens	• Do not want to introduce water into the specimen • Ambient cutting smears asphalt, sealing specimen

repeated loading level were also evaluated. The purpose of the program was to develop a test procedure with the ECS that would most realistically simulate field conditions, yet be reasonable to conduct in the laboratory. A 3 × 3 × 3 factorial experiment using the ECS was developed; the controlled variables are summarized in Table 7.3.

Materials included two aggregates RB and RL and two asphalts, AAG-1 and AAK-1. Aggregate RL was reported as an aggregate which produced water sensitive mixes.

Table 7.3 summarizes the results of the pilot test program (Terrel and Al-Swailmi 1993a). The current ECS test protocol was developed from the results of these studies, and is summarized in Table 7.4. A servo-hydraulic load system was used for an initial check modulus measurement, and could be used to screen specimens prior to ECS testing. Evaluation of the performance of the test specimen includes three criteria: ECS modulus ratio, coefficient of permeability, and visual evaluation of stripping. ECS modulus ratio is plotted versus conditioning cycle as shown in Figure 7.2. A lower final modulus ratio indicates a greater degree of damage to the specimen.

Table 7.3. Summary of pilot study results

Issue	Proposed	Selected Method	Rational
Conditioning cycle temperature	• hot: 60°C • ambient: 25°C • freeze: 25°C	• 3 hot and one optional freeze	• Hot cycles most severe, result in greatest loss of modulus • Cycling from hot-ambient, freezing-ambient, induces damage in the specimen • Freeze cycle may indicate degradation of the aggregate
Conditioning fluid	• Water • Moist vapor • Dry	• Water	• Water most severe, results in greatest loss of modulus • Dry promotes aging only • Moist air promotes both aging and water damage
Repeated loading	• 200 lb repeated load • Static loading	• Repeated load	• Induces dynamic pore water pressures as seen under traffic loading • More realistic
Cycle length	• 24 hour • 6 hour	• 6 hour	• No statistical difference in performance between 24 and 6 hour cycles • Convenience
Vacuum level	• 25 cm (10 in.) Hg • 51 cm (20 in.) Hg	• 51 cm (20 in.) Hg for initial specimen wetting • 25 cm (10 in.) Hg for hot conditioning cycles	• 51 cm (20 in.) Hg too severe when coupled with repeated loading for conditioning
Saturation level	• Required saturation level (60% to 80%) • Set saturation time [30 min. at 51 cm (20 in.) Hg]	• Set saturation time	• Allows for wetting of all types of specimens • Allows specimens to saturate according to their void matrix

°C = 9 ÷ 5 + 32 °F

The change in the coefficient of water permeability during the ECS test procedure indicates the change to the specimen void structure. Some specimens tend to show a decrease in permeability, indicating a densification of the mix under the action of repeated loading. Others tend to increase in permeability during the first conditioning cycle, possibly due to the initial breaking of asphalt bonds in the specimen.

After the ECS testing procedure is completed, and the specimen has been tested for resilient modulus with either the MTS or the ECS, and a visual evaluation of stripping is taken, specimens are split in half by applying a diametral static load. The two broken faces are examined to determine what percentage of the surface area of the face has been stripped of asphalt. The percentage of stripping is reported to be: 0, 5, 10, 20, 30, 40 or 50 percent using a standard guide. Fractured faces are excluded in the identification of aggregate faces which have lost their asphalt covering.

Table 7.4. Summary of the ECS test procedure

Step	Description
1	Prepare test specimens as per SHRP protocol.
2	Determine the geometric and volumetric properties of the specimen. Determine the triaxial and diametral modulus using the MTS system.
3	Encapsulate specimen in silicon sealant and latex rubber membrane, allow to cure overnight (24 hours).
4	Place the specimen in the ECS load frame, between two perforated teflon discs, determine air permeability.
5	Determine unconditioned (dry) triaxial resilient modulus.
6	Vacuum condition specimen [subject to vacuum of 51 cm (20 in.) Hg for 10 minutes].
7	Wet specimen by pulling distilled water through specimen for 30 minutes using a 51 cm (20 in.) Hg vacuum.
8	Determine unconditioned water permeability.
9	Heat the specimen to 60°C (140°F) for six hours, under repeated loading. This is a hot cycle.
10	Cool the specimen to 25°C (77°F) for at least four hours. Measure triaxial resilient modulus and water permeability.
11	Repeat steps 9 and 10 for two more hot cycles.
12	Cool the specimen to -18°C (0°F) for 6 hours, without repeated loading. This is a freeze cycle.
13	Heat the specimen to 25°C (77°F) for at least 4 hours and measure the triaxial resilient modulus and the water permeability.
14	Split the specimen and perform a visual evaluation of stripping.
15	Plot the triaxial resilient modulus and water permeability ratios.

(after Scholz et al., 1992)

The pilot test program also included an effort to verify the pessimum voids hypothesis in the laboratory, but the ECS laboratory experiment plan was not appropriate to be used directly for this purpose. Another water conditioning study was conducted exclusively to prove the pessimum voids concept by providing free water drainage. A separate conditioning set-up was constructed to permit this conditioning to simulate the action of free drainage following wetting. Three 2-specimen sets of mixes were prepared from the same asphalt-aggregate combination (RL/AAK) and compacted at three air void contents; low at 4 percent, pessimum range at 8 percent, and free draining at 30 percent. The diametral resilient modulus was then determined for each specimen. The six specimens were placed in a vacuum container and a partial vacuum of 56 cm (22 in.) Hg was applied for 10 minutes. Then, the vacuum was removed and the specimens were left submerged in the water for 30 minutes. This wetting process was selected by trial and error to provide partial saturation of 70 percent for the specimens with 8 percent air voids. Using the same procedure, saturations of 99 percent and 38 percent were achieved for open graded and low air void specimens respectively.

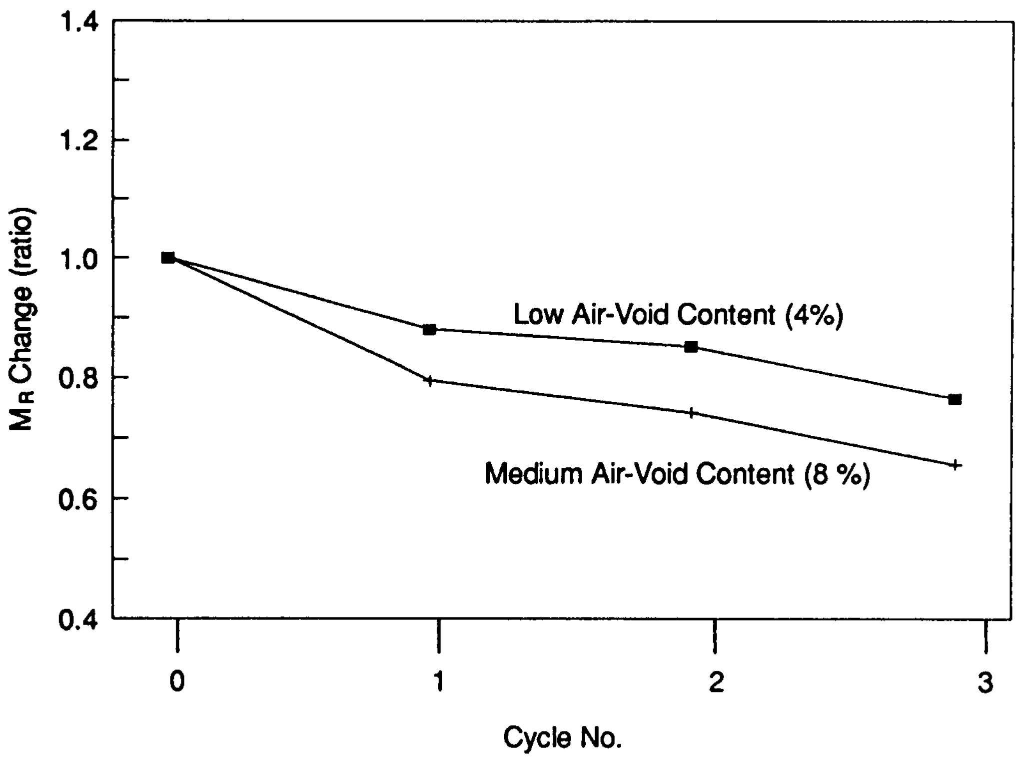


Figure 7.2. Change in ECS modulus ratio

After water saturation, the specimens were placed in an air bath (environmental cabinet) for 6 h at 50°C (122°F), then 5 h at 25°C (77°F) and allowed to drain. Diametral resilient modulus was determined at the end of each conditioning cycle and retained resilient modulus was expressed as the ratio of the conditioned to the original dry resilient modulus. The conditioning temperature was chosen as 50°C (122°F) instead of 60°C (140°F) because of the tendency for open graded specimens to deform under their own weight at the higher temperature. In addition, open graded specimens were enclosed with a 10-cm (4-in.) diameter membrane during conditioning cycles to assist in retaining their original geometry.

The conditioning process [partial saturation, 6 h at 50°C (122°F), then 5 h at 25°C (77°F)] was repeated 20 times (cycles). Figure 7.3 shows the data and the average curve of ECS modulus for the three mixes. Each data point is the average of two specimens. The impermeable mixes shows no water damage, and the open graded set shows a slight decrease in retained modulus. The mix with the middle, or pessimum range, shows significant water damage. Figure 7.4 was prepared to show the data in a format used for the pessimum voids concept (Terrel and Al-Swailmi 1993b). The results confirm the hypothesis that air voids in the pessimum range play an important role in the performance of asphalt concrete in the presence of water. Water retained in these voids during the service life of the pavement would tend to cause more damage than in mixes with either more or less voids.

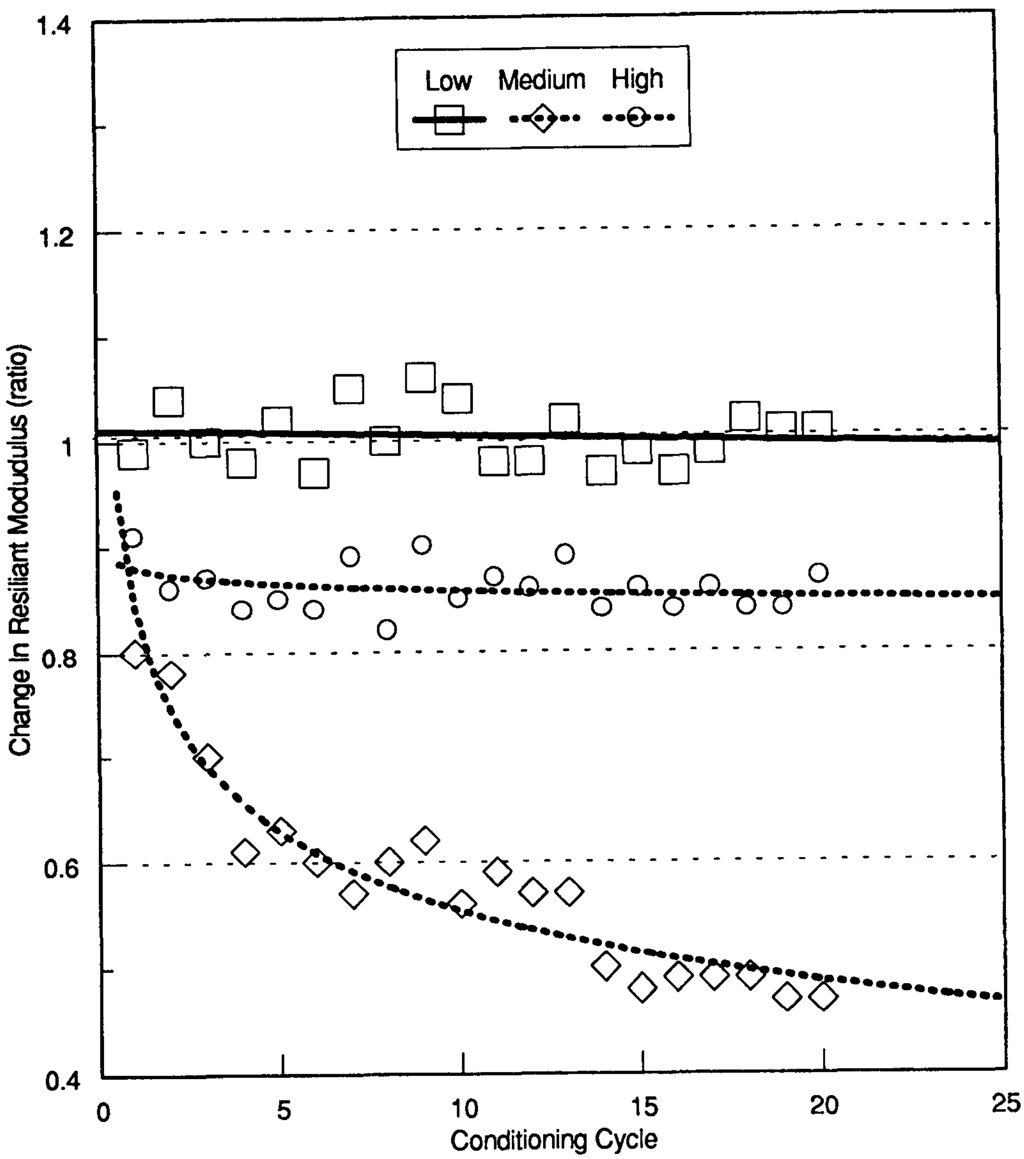
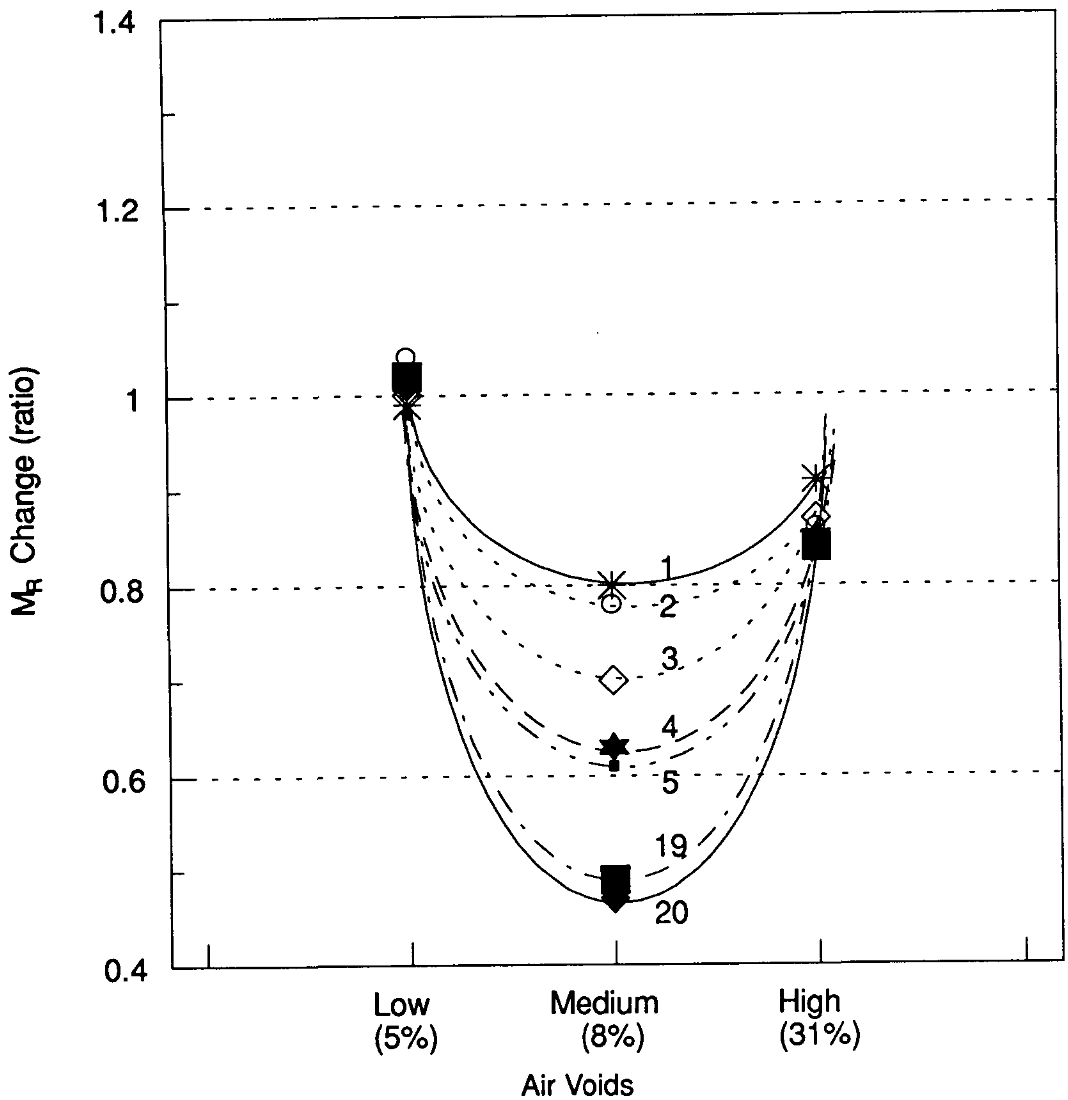


Figure 7.3. Diametral modulus change after free draining water conditioning



Cycle No.	Legend	Low	Medium	High
1		0.99	0.80	0.91
2		1.04	0.78	0.86
3		1.00	0.70	0.87
4		0.98	0.61	0.84
5		1.02	0.63	0.85
19		1.01	0.47	0.84
20		1.01	0.47	0.87

Figure 7.4. Diametral modulus — air void content relationship after free draining water conditioning to illustrate pessimum voids concept

7.4 Field Validation of the ECS

The purpose of the field validation program was to demonstrate that the ECS test can accurately discriminate between superior and inferior asphalt concrete mixes as demonstrated by their performance in full-scale field test sections. In addition, the correlation among the performance of mixes in the ECS, OSU wheel-tracker, and field sections was also examined. The validation effort differs from the previous work conducted with the ECS in that all the mixes used were designed by the respective local highway authorities. In the previous studies, mix designs were developed at the University of California at Berkeley.

This section presents the testing procedures, results, and analysis of data for twelve asphalt-aggregate mixes tested in the ECS, OSU wheel-tracker, and full-scale field test sections. The analysis of data correlates the performance of these mixes, as subjected to the four types of moisture conditioning inherent to either the test apparatus or geographical location where the asphalt mix was used. Based on the data from this effort, criteria for the use of ECS data in a mix design and analysis system were developed.

Figure 7.5 illustrates an overview of the testing program for the field validation work. For evaluation using the ECS procedure, specimens were manufactured using the laboratory kneading compactor. From large-roller-compacted slabs, beam specimens were cut for use in the OSU wheel-tracker, and specimens were cored for use in the ECS. Field specimens were cored from field test sections for evaluation in the laboratory. Table 7.5 summarizes the modes of performance which were monitored.

Twelve field sites were selected for the field validation effort. The basis for selection included the following: availability of original materials (asphalt, aggregate, and admixtures); mix design information; and cooperation from the presiding authority for field coring. At least two sites were selected from each of the four SHRP environmental zones which were as old as possible so that they had been subjected to several seasons of natural environmental conditioning. Unfortunately these requirements restricted the number of field sites that were available for the validation study. The twelve sites selected are identified in Table 7.6. Details of the site materials, construction, climate, etc., are contained in the project report by Allen and Terrel (1993).

Table 7.7 indicates the condition of each field test section from the most recent manual distress survey for the site.

7.4.1 Test Results

Examples of the results from ECS tests are shown in Figures 7.6 and 7.7. Each data curve represents a single ECS specimen. The curves define the change in retained resilient modulus (termed ECS-modulus ratio) as a function of the conditioning level (each cycle represents a conditioning cycle within the ECS with the first three cycles being "hot" cycles and the fourth cycle being the "freeze" cycle). The retained resilient modulus, or ECS modulus ratio, is defined as the ratio of the conditioned resilient modulus to the

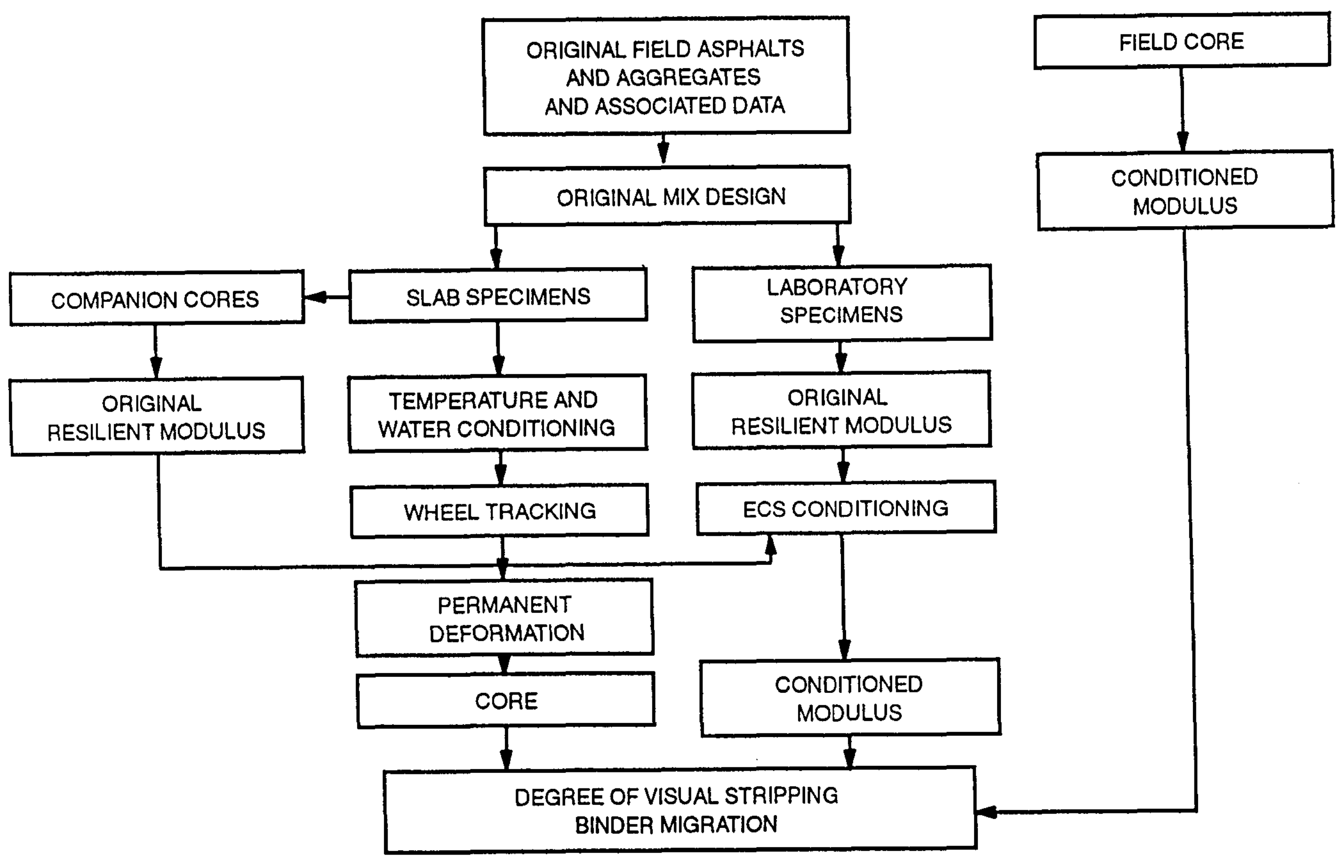


Figure 7.5. Field validation of water sensitivity, test program

Table 7.5. Specimen, test procedure, and performance mode identification used for the field validation program

Specimen	Test Procedure	Performance Mode
Laboratory Kneading Compactor	ECS	ECS modulus Visual evaluation of stripping
Roller Compactor	ECS	ECS modulus Visual evaluation of stripping
OSU wheel-tracker/ Rutted Beam	OSU wheel-tracker	Rut depth MTS modulus Visual evaluation of stripping
Field	Field	MTS modulus Visual evaluation of stripping

Table 7.6. Field site identification

Site	Governing Agency	Mix Designation
Alberta, SPS-5 (AB5)	SHRP	
Arizona, SPS-5 (AZ5)	SHRP	Arizona DOT 3/4-in. modified
California, AAMAS Batch (CAB)	CALTRANS	CALTRANS Type "A" mix
California, AAMAS Drum (CAD)	CALTRANS	CALTRANS Type "A" mix
California, GPS-6b (CAG)	SHRP	
Georgia, AAMAS (GAA)	Georgia DOT	Georgia DOT "B" mix
Minnesota, SPS-5 (MN5)	SHRP	
Mississippi, SPS-5 (MS5)	SHRP	Mississippi DOT Surface SC-1 (Type 8)
Rainier, Oregon (OR1)	Oregon DOT	Oregon DOT open graded "F" mix
Bend-Redmond, Oregon (OR2)	Oregon DOT	Oregon DOT "B" mix
Mount Baker, Washington (WA1)	FHWA	Polymer modified
Wisconsin, AAMAS (WIA)	Wisconsin DOT	Recycled

Table 7.7. Summary of manual pavement condition surveys

Site	Survey Date	Comments
AB5	8/92	In good condition, small amount of cracking
AZ5	8/92	In good condition, some traffic densification
CAB	8/92	In good condition
CAD	8/92	In good condition
CAG	8/92	In good condition
GAA	NA[a]	Covered by wearing course
MN5	6/92	Some low to moderate severity transverse cracking, 5mm - 8 mm rutting, some low to moderate severity bleeding
MS5	Spring 1992	In bad condition, reflection cracking, scheduled for overlay
OR1	NA	Covered by wearing course
OR2	10/92	No visual distress with the exception of .32 cm to .95 cm (1/8 in. to 3/8 in.) of rutting
WA1	9/92	In good condition, no visible rutting
WIA	1991	In good condition, PDI=0, PSI=4.3, 1/4 cm (1/10 in.) rutting measured

[a]Information not available

unconditioned modulus, and is measured at the end of each conditioning cycle. The ECS modulus ratio provides an indication of strength lost in the specimens due to water damage relative to the dry, unconditioned strength of the specimen.

Results from the OSU wheel-tracking program are summarized in Figure 7.8. Each beam was used as a unique specimen, and its rut depth was used for statistical analysis. The average rut depth of two specimens from the same mix was only used in Figure 7.9 for illustration.

Figure 7.9 shows the relationship between the visual stripping shown in field cores, and that observed in specimens from the ECS. Typically, specimens from the same mix appeared very similar for the field cores and the ECS specimens. However, two differences were noted: 1) the asphalt in the field cores appeared a much duller, flat black in color, while that in the ECS specimens was typically a dark, shiny black; and 2) no migration of asphalt binder was seen in any of the field cores. The differences in the appearance of the asphalt between field and ECS specimens may be due to aging of the asphalt in the field. The lack of migration of asphalt binder in field specimens may be due to their relatively short lifetime in the field.

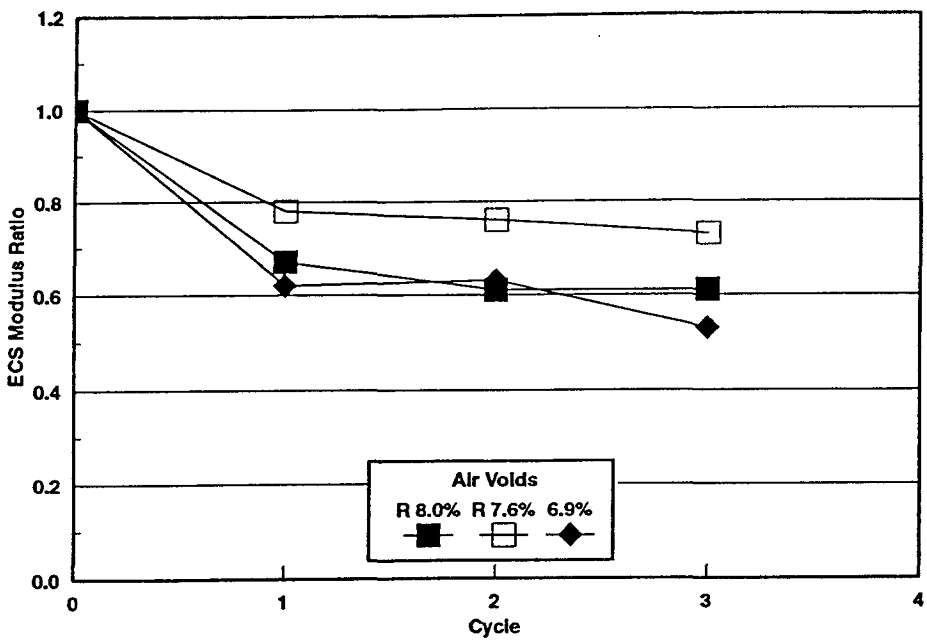


Figure 7.6. Mississippi SP-5 (MS5) ECS results

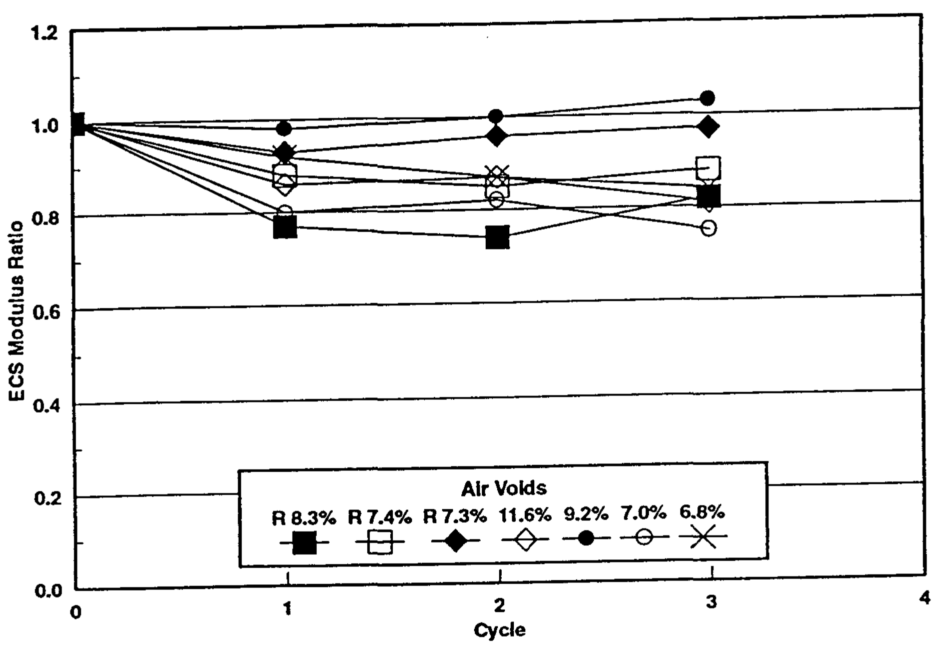


Figure 7.7. Rainier, Oregon (ORI), ECS results

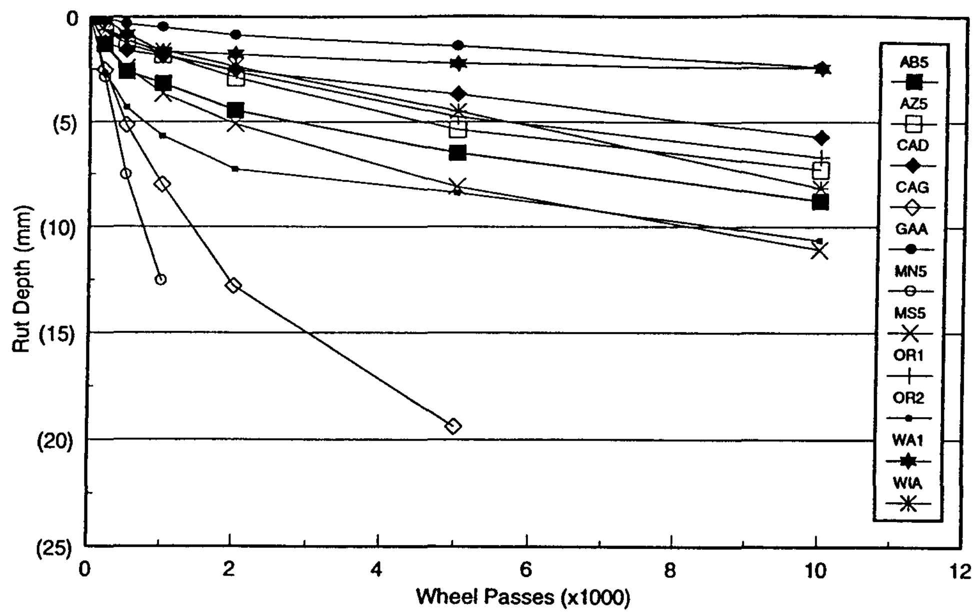


Figure 7.8. Average rut depths for OSU wheel-tracking test program

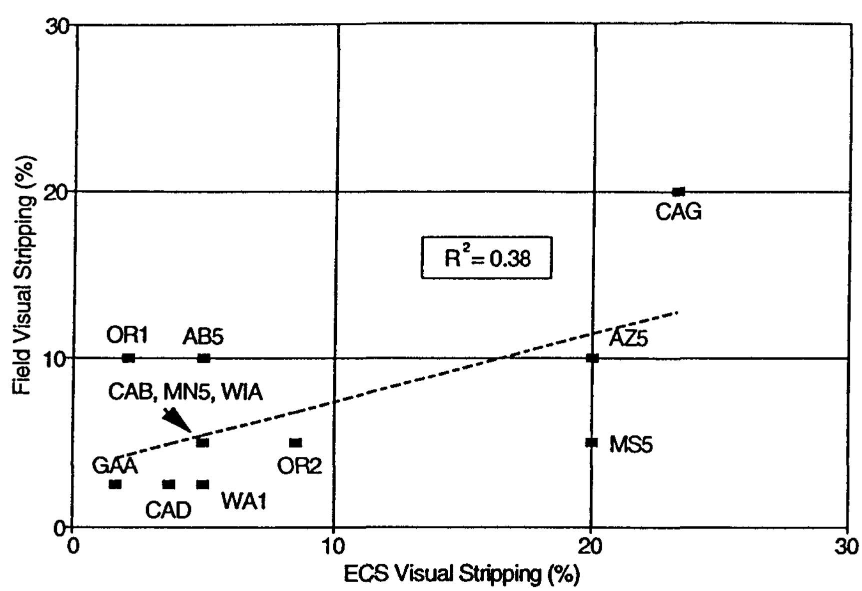


Figure 7.9. Visual stripping, comparison of field and ECS specimens

Figure 7.10 shows the relation between the ECS final modulus ratio and the OSU wheel-tracker rut depth. The beams manufactured from the MN5 mix had air voids at least 200-percent higher than those found in the ECS kneading compacted specimens. If the data points for MN5, on the basis of its high air voids, and OR2, since it is an open graded mix, are removed, Figure 7.11 results. There is no valid reason to remove the data point for CAD from the analysis, even though it represents data from only one beam. A best fit line can be placed through this data using simple linear regression, as shown in Figure 7.11. With the exception of the mixes from California, the data fits this line well.

From the preceding analysis several items of significance have emerged, as follows:

ECS results:

1. A high percentage of the reduction in ECS modulus ratio occurs in the first cycle of ECS testing; however, a significant change may occur between cycle 1 and cycle 3 for some mixes (MN5, AB5, and CAG).

2. Some mixes may experience an increase in modulus ratio after the freeze cycle.

3. The slope of the ECS modulus ratio curve indicates the rate of water damage to the specimen. At this time, a correlation between cycles of ECS conditioning and the corresponding period of field life has not been established.

4. Of the variables considered (mix type, air voids, initial modulus, air permeability, and water permeability), mix type, initial modulus, and air voids have the strongest influence on the final ECS modulus ratio of the mix.

5. There is no statistical significance between the results from mixes that were subjected to the freezing cycle and those which were subjected to only three hot conditioning cycles. This indicates that neither procedure—three cycles with no freeze or four cycles including a freeze—is consistently more severe. The fact that different grades of asphalt are typically used for Freeze and No-Freeze environments may be reflected by this data.

OSU wheel-tracker:

1. The air void levels vary for some mixes between the beam specimens and the corresponding laboratory kneading compactor specimens, and may result in high rut values (MN5) that are not representative of the expected performance of the mix.

2. Anomalous results indicate that several of the mixes should be retested in this apparatus (CAD, CAG, and MN5).

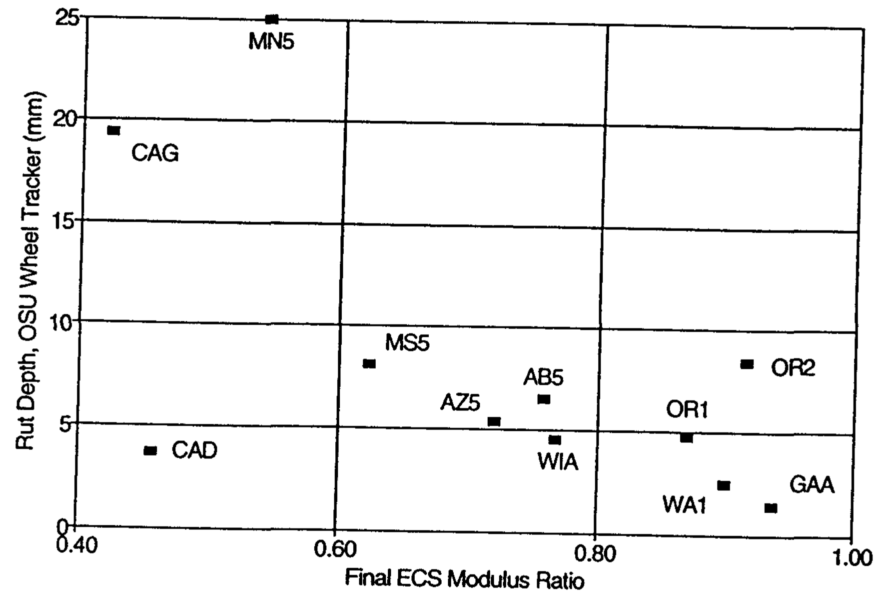


Figure 7.10. Comparison of ECS and OSU wheel-tracker performance

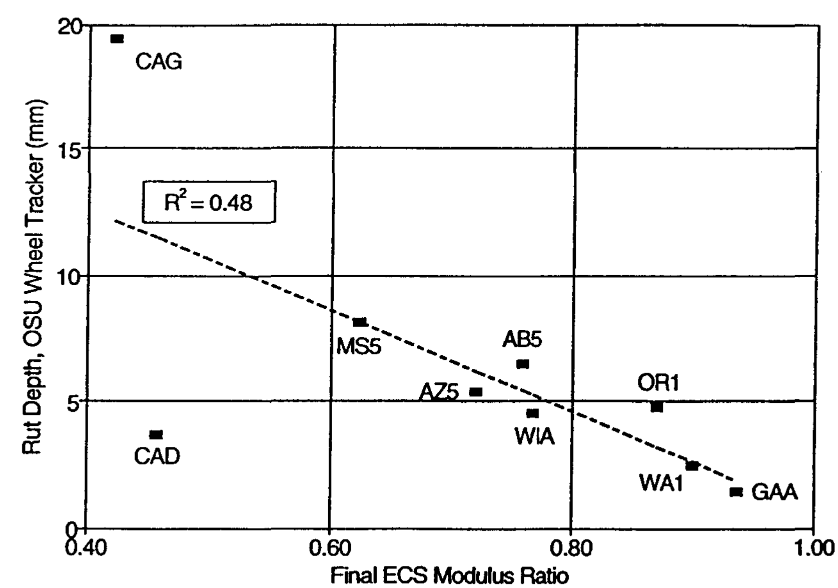


Figure 7.11. Comparison of ECS and OSU wheel-tracker performance, MN5 and OR2 removed

Field data:

1. The sites are all currently in good condition, with the exception of MS5, which is due to be overlaid. This indicates that no water damage has yet been seen in the field mixes.

2. Long-term aging of mixes in the field may be increasing the modulus of field cores.

Comparison of test procedures:

1. ECS and Field Cores: For mixes with 102 mm (4.0 in.) high cores, a comparison of triaxial modulus ratios indicates that the ECS tends to induce more water damage than field conditions; however, the difference is not statistically significant.

2. ECS and Field Cores: The mixes in the field may be experiencing long-term aging, which is not simulated in the ECS test procedure.

3. ECS and OSU wheel-tracker: A correlation between the performance of mixes in the ECS and OSU wheel-tracker is evident.

Table 7.8 indicates the orders of performance ranking for the mixes tested in the three test procedures. For the ECS, this ranking is the final ranking of all twelve mixes, regardless of environmental zone. This listing would be the one that corresponds to the rankings given by the OSU wheel-tracker, which uses both Freeze and No-Freeze conditioning, and the field, which may represent either a Freeze or No-Freeze environment.

A comparison of the severity of the ECS test to the field conditions to which the mix was subjected was performed. First, field cores which were tall enough to allow for MTS triaxial modulus testing were directly compared to ECS specimens by using the laboratory specimen MTS data to produce initial triaxial modulus data for the field cores. This allowed a modulus ratio to be developed. Six mixes were evaluated in this manner. Second, the correlation between the performance of the field mixes, measured by a diametral modulus ratio, and the ECS triaxial modulus ratio was investigated.

To compare triaxial modulus ratios between the ECS and field cores, a regression equation with MTS triaxial modulus as a function of air voids was developed for each mix using the unconditioned kneading compacted specimens. The initial modulus (unconditioned modulus) for the field core was then determined using its present air void level. A model was run using the GLM procedure to compare the final ECS modulus ratios with the field core modulus ratios. Mix type (MIX) and test procedure (TEST) were the independent variables. The interaction between the two variables was also included (MIX*TEST).

Table 7.8. Comparison of ranking of mixes by test method

	ECS		OSU Tracking 5,000 Wheel Passes		Field Cores	
Ranking	Mix	T Grouping[1]	Mix	T Grouping	Mix	T Grouping
1	GAA	A	GAA	A	WIA	A
2	OR2	A	WA1	A,B	AZ5	A,B
3	WA1	A,B	CAD	A,B,C	MS5	B
4	OR1	A,B,C	WIA	A,B,C	WA1	B
5	WIA	B,D,C	OR1	B,C	CAG[2]	B,C
6	AB5	C,D,E	AZ5	B,C,D	OR1	C,D
7	AZ5	D,E	AB5	C,D	CAB MN5	F E
8	MS5	E,F	MS5	D	CAD	F
9	CAB	F,G	OR2	D	GAA	F,G
10	MN5	F,G	CAG	E	OR2	F,G
11	CAD	G	MN5	Failed	AB5	G
12	CAG	G				

[1]Groupings with the same letter designation include means which are not significantly different at the $\alpha = 0.05$ level.
[2]CAG cores from second coring.

A comparison of the performance of the mixes in the ECS and in the field can be seen in Figure 7.12. The diametral modulus ratio of the field cores versus the ECS final modulus ratio is shown. From the most recent field distress surveys, it is known that the MS5 field section is showing signs of rutting and reflective cracking, and is due to be overlaid. This distress developed over the past winter season, after the field cores had been taken in the summer of 1991. At that time the section showed no signs of distress. MS5 is the only field section that at this time shows any substantial distress.

When comparing the results of the ECS testing with modulus ratios developed for the field cores, consideration should be given to the potential for the mixes in the field to be experiencing long-term aging. The mixes that are tested in the ECS are only subjected to short-term aging of the loose mix. In the field, mixes are experiencing long-term aging, which tends to increase the modulus of the mix. In the early life of the pavement, before water damage has developed fully, the increase in strength due to aging may overwhelm any decrease in strength that is beginning to occur due to water damage. The data from CAG may illustrate this point. In Figure 7.13, two sets of cores from CAG are represented. CAG1 represents cores that were taken within one month of paving.

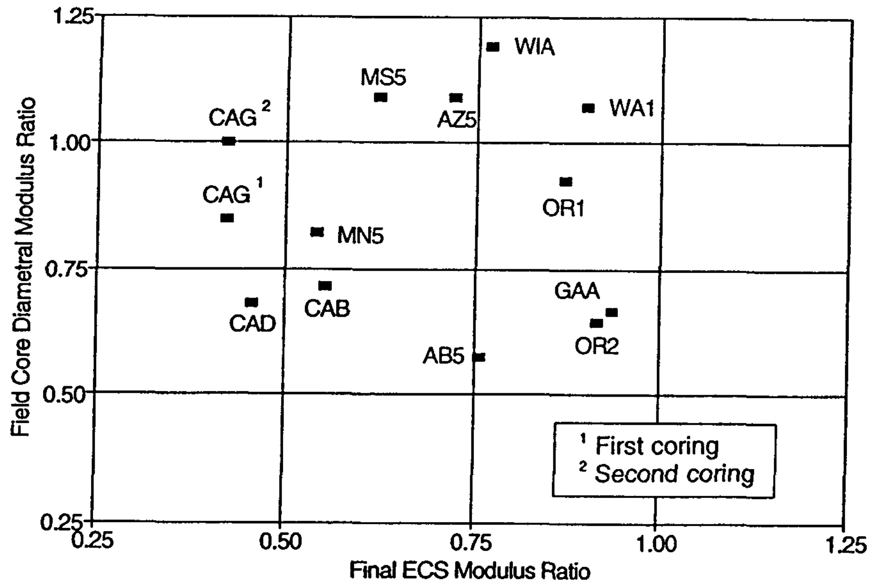


Figure 7.12. Comparison of ECS and field performance

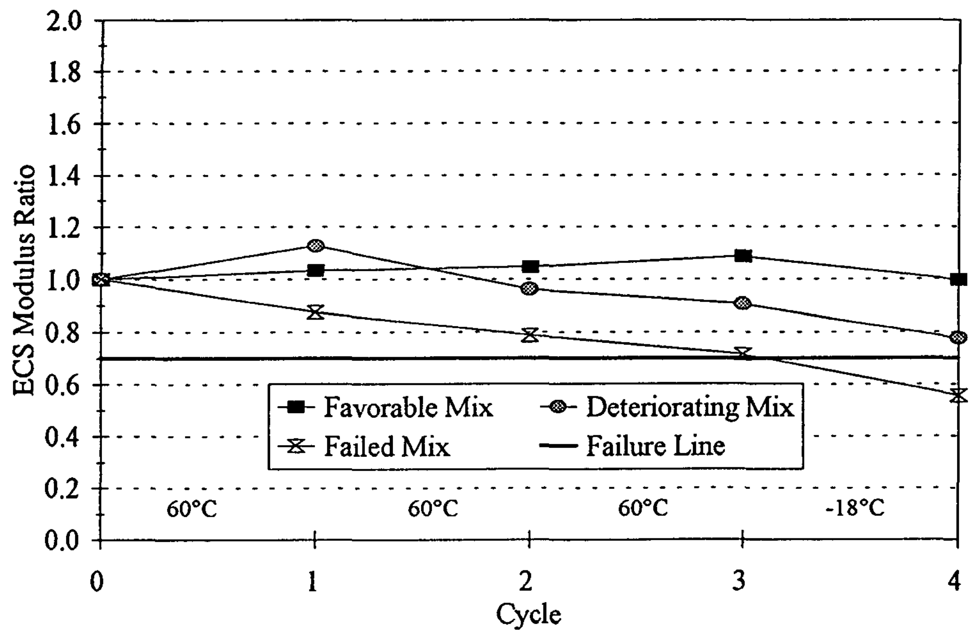


Figure 7.13. ECS M_R ratio trends — Colorado data

7.5 Mix Design and Analysis

As designed in the laboratory, mixes selected in the preliminary volumetric mix design will be subjected to short-term oven aging before compaction into specimens for the ECS. The preliminary mix design will determine the aggregate and asphalt type, the aggregate gradation, and the asphalt content.

ECS specimens will then be compacted at two air void levels — 7 ± 1 percent for Levels 1, 2, and 3; and 10 ± 1 percent for Levels 2 and 3. These void levels where chosen in accordance with the pessimum voids theory proposed by Terrel and Al-Swailmi (1993b). Two specimens will be compacted at each level.

Two specimens of a given mix and and air void content will be run through the ECS procedure using three or four cycles. The fourth, or freeze, cycle is optional for use in environments which experience freeze-thaw conditions. A plot of the ECS modulus ratio versus cycles will be used to rate the specimen performance.

From the field data, a final modulus ratio of 0.7 appears to separate mixes which performed well in the ECS, OSU wheel-tracker, and the field, from those which showed deterioration in the OSU wheel-tracker or the field. This is illustrated in Figures 7.14 and 7.15. It is therefore recommended that the following procedure be used:

> **Level 1**: If the final ECS modulus ratio is less than 0.7, the mix should be treated for moisture susceptibility and retested in the ECS. If the final ECS modulus ratio is greater than 0.8, the slope of the curve between cycles 1 and 3 should be investigated. For mixes with flat slopes, the mix is expected to perform well, and no treatment is recommended. For mixes with steeper slopes, treatment of the mix for moisture sensitivity should be considered, as these mixes may experience moisture damage, only at a slower rate than those with final modulus ratios of less than 0.7.
>
> **Level 2**: For the mix specimens with air voids contents of 7 ± 1 percent, the criteria is the same as in Level 1. For the specimens with air voids content of 10 ± 1 percent, the mix should be treated for moisture susceptibility if the final ECS modulus ratio is less than 0.6. Again, the slope of the curve between cycles 1 and 3 is an indicator of delayed moisture damage to the mix.
>
> **Level 3:** Level 3 varies from Level 2 only by adding on tests after the ECS procedure. Simple shear tests would be performed on both conditioned and unconditioned ECS test specimens for a certain requirement of acceptability. If the mix does not meet this requirement, it would be redesigned to improve performance. Evaluation of mixes with the ECS test procedure should eliminate the mixes that would experience water damage within the first several years of life. Currently, only one of the mixes tested — MS5, with a final ECS modulus ratio of 0.62 and slope of -0.0337 — has failed in the field. Treatment of mixes which show tendencies for water damage over a longer life, as evidenced by steep modulus ratio curves between the first and third cycles, may also be treated to further extend pavement life.

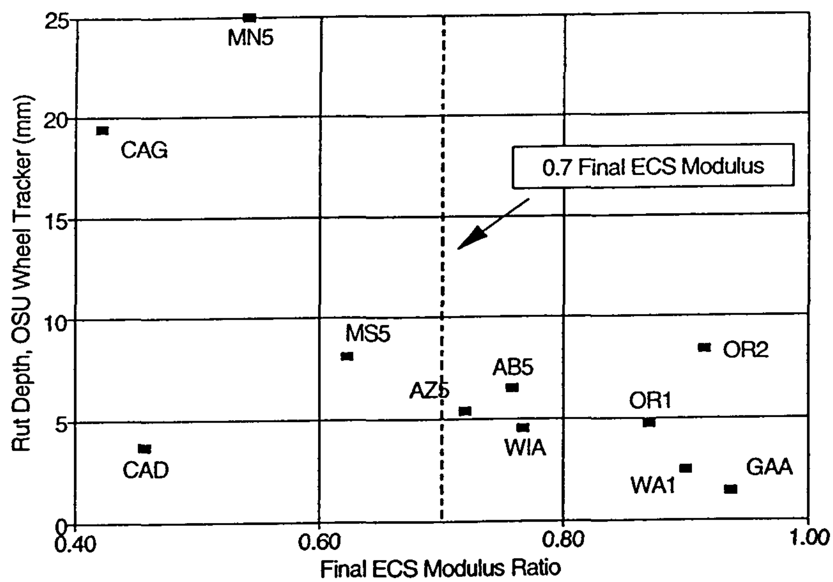


Figure 7.14. Criteria for the performance of mixes, OSU wheel-tracker versus ECS

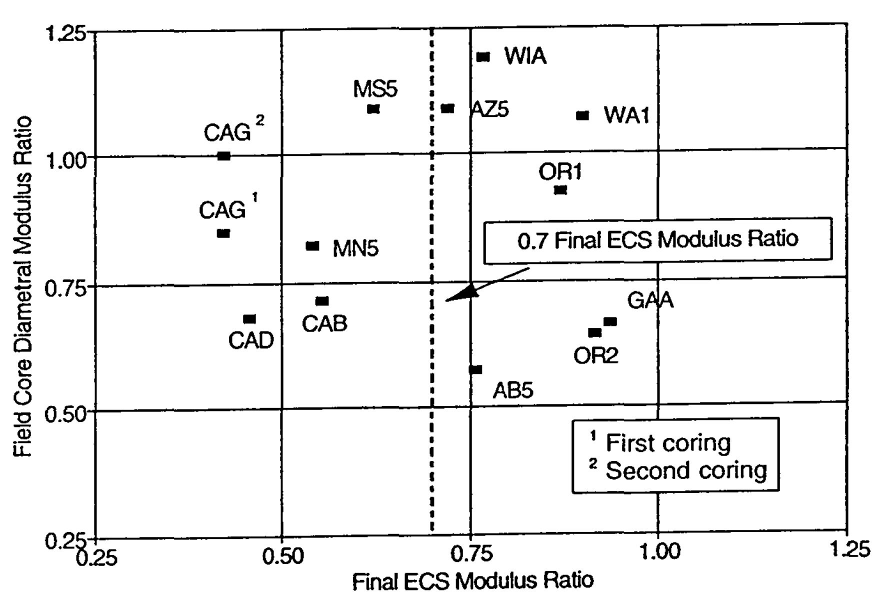


Figure 7.15. Criteria for the performance of mixes, field versus ECS

7.6 Summary

The work performed to evaluate the ECS test procedure during the preliminary test development, using asphalt-concrete mixes from the field, provides an initial database of information on the ECS test procedure, correlating the performance of mixes in the field, in the ECS, and in the OSU wheel-tracker. The limited amount of materials available and the length of time that pavements have been in the field indicates that additional time and testing will only better define the role of the ECS in modern mix design.

The following conclusions can be drawn from the data collected to date with the ECS:

1. The ECS as developed appears suitable for inducing water damage in asphalt-concrete mixes in the laboratory analogous to that seen in field mixes.

2. The ECS procedure requires refinement to fit into a standard 8-hour workday.

3. Evaluation of the criteria for evaluating ECS results is continuing, especially with regard to the visual degree of stripping and binder migration.

4. Initial results of field validation are encouraging, especially with regard to correlation between the ECS and wheel-tracking tests.

The following recommendations can be made to further validate the use of the ECS procedure for determining the water sensitivity of asphalt mixes:

1. A strong correlation between ECS performance and the number of years of expected field performance has not yet been made due to the relative youth of the field sections. A continued program of coring to further validate and refine the role of the ECS test procedure in a mix design program is suggested.

2. A controlled program of materials collection, construction of field sections, and continued coring to provide a larger database for the ECS criteria should be developed. Enough asphalt and aggregate should be sampled at the time of construction to allow both ECS specimens and OSU wheel-tracker beams (at least four) to be manufactured. Several of the primary mixes tested should have been replicated due to anomalous results from the OSU wheel-tracker (CAD, CAG, and MN5). However, there was no opportunity to complete this work due to lack of original aggregates.

3. The procedure evaluating visual stripping and binder migration in mixes should be improved to remove as much of the subjectivity as possible. The use of optical scanners to determine the amount of stripping in a mix is worthy of investigation. Evaluation of aggregate degradation should also be addressed.

4. The ECS should be used to provide a systematic look at the effects in variations in volumetric mix proportions, such as gradation, asphalt content, and air voids, on mix performance. The pessimum voids concept proposed by Terrel and Al-Swailmi (1993b) suggests that mixes with a certain range of air voids level may be prone to water damage due to the structure of the void system. Gradation and asphalt content also will affect the air void structure of a mix.

5. The ECS equipment and procedure should be included as a standard component of mix design.

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