TRANSPORTATION RESEARCH

CIRCULAR

Number 503 December 2001

Perpetual Bituminous Pavements

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CIRCULAR

Number 503, December 2001
ISSN 0097-8515

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Subscriber category
IIIB materials and construction

Website: TRB.org

Transportation Research Board
National Research Center
2101 Constitution Avenue, NW
Washington, DC 20418

Foreword

The concept of perpetual asphalt pavements is rapidly gaining acceptance in the United States. The idea is to extend the 20-year life expectancies of hot-mix asphalt pavement to greater than 50 years. To do so requires combining a rut-resistant, impermeable, and wear-resistant top structural layer with a rut-resistant and durable intermediate layer and a fatigue-resistant and durable base layer. With these pavements, only periodic surface restoration is necessary, offering advantages in speed of construction (user delay costs) and construction costs.

This concept was explored and discussed at a two-part session on perpetual bituminous pavements at the 2001 Transportation Research Board Annual Meeting. The papers in this document were written following the session and are based on the presentations; the papers in this Circular have not undergone peer review.

Appreciation is expressed to Frank Fee, Citgo Asphalt Refining Company, for his efforts in developing this Circular.

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Concepts of Perpetual Pavements

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The construction of long-lasting hot mix asphalt pavements has been practiced for a number of decades in the United States. Full-depth (asphalt courses used for all layers above subgrade) and deep-strength (asphalt surface and asphalt base over a minimal aggregate base above subgrade) pavements were originally designed for 20-year life expectancies. One of the primary advantages to these designs was that the total pavement sections were thinner when compared to conventional designs of asphalt over thick aggregate bases. As these full-depth and deep-strength pavements performed beyond their design lives, the vast majority only required surface restoration such as a thin overlays or mill and overlay. This practice of replacing only the surface offers a number of rehabilitation advantages in terms of speed of construction (user delay costs) and construction costs. The challenge for today is to obtain a longer surface life on a long-lasting asphalt support structure. Recent efforts in materials selection, mixture design, performance testing, and pavement design offer a methodology which may be employed to obtain very long-term performance from asphalt pavement structures (greater than 50 years) while periodically (approximately every 20 years) replacing the surface (top 25 to 100 mm) of the pavement. This concept has been proposed for use in Europe and it is rapidly gaining acceptance in the United States. The common theme in these approaches is to combine a rut resistant, impermeable, and wear resistant top structural layer with a rut resistant and durable intermediate layer and a fatigue resistant and durable base layer.

The concept of long-lasting asphalt pavements is not new. Full-depth and deep-strength asphalt pavement structures have been constructed since the 1960s. Full-depth pavements are constructed directly on subgrade soils and deep-strength sections are placed on relatively thin (4 to 6 in.) granular base courses. One of the chief advantages of these pavements is that the overall section of the pavement is thinner than those employing thick granular base courses. Such pavements have the added advantage of significantly reducing the potential for fatigue cracking by minimizing the tensile strains at the bottom of the asphalt layer. Furthermore, a study by the National Center for Asphalt Technology suggests that when rutting occurs in thick pavements, it is likely to be confined to the top 2 in. of the structure (*1*). When this occurs, an economical solution is to remove the top layer of material and replace it to the same level (mill and fill).

Thick asphalt pavements must be designed and constructed to ensure performance for the traffic, climate, and soils in given areas. For example, unless proper attention is given to frost protection, ride quality may degenerate from good to poor in a matter of months because of differential frost heave in the subgrade. Other performance problems in thick asphalt pavements could include a lack of durability (stripping) in intermediate layers and thermal cracking allowed

to progress to very severe levels causing a loss of ride quality and secondary fatigue cracking. While these distresses are not unique to thick asphalt pavements, their effects become magnified by the amount of material that must be dealt with when correcting them.

In a case study representative of good performing pavements, a recent review of thick (200 mm or greater) asphalt pavements on Interstate 90 through the state of Washington revealed that none of these sections had ever been rebuilt for structural reasons (*2*). The pavement ages ranged from 23 to 35 years, and thick asphalt concrete (AC) pavements on this route comprise 40 percent of the length (about 225 out of 580 km). West of the Cascade Mountains, near Seattle, the average age at resurfacing was 18.5 years. On the eastern side of the state, the average time until first resurfacing was 12.4 years and the time until second resurfacing was 12.2 years. This type of performance, while good, can be significantly improved with new technology.

Recent efforts in materials selection, mixture design, performance testing, and pavement design offer a methodology which may be employed to obtain long-lasting performance from asphalt pavement structures (greater than 50 years) while periodically (approximately every 20 years) replacing the surface of the pavement. This concept has been proposed by Nunn et al. of the Transport Research Laboratory in the United Kingdom (*3*). The California Department of Transportation, with the assistance of a research team at the University of California, Berkeley, is planning the rehabilitation of a concrete freeway near Long Beach using the concept of a perpetual asphalt pavement as an overlay to the broken and seated concrete (*4*). Finally, Von Quintas proposed this approach in the rehabilitation of the southeast corridor in Denver (*5*). The common theme in these approaches is to combine a rut resistant, impermeable, and wear resistant top structural layer with a rut resistant and durable intermediate layer and a fatigue resistant and durable base layer (Figure 1). For areas where the use of an open-graded friction course is desired, this feature can be incorporated and viewed as a renewable wearing course.

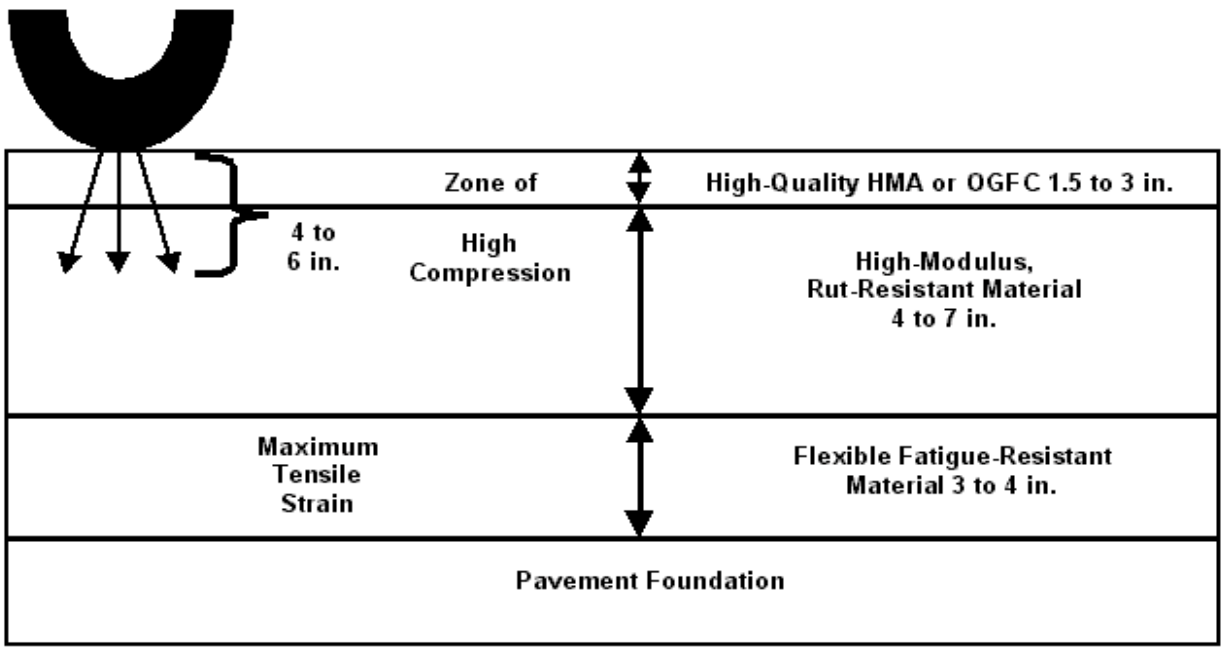


FIGURE 1 Perpetual pavement design concept (HMA = hot-mix asphalt).

MECHANISTICALLY BASED DESIGN

Given the innovative approach to performance-specific design in each layer, it will be necessary to use a structural design method that allows for an analysis of each pavement layer. Most current pavement design procedures do not consider each layer of the asphalt pavement structure. Instead, all of the asphalt layers are considered in combination using a factor such as a layer coefficient in the 1993 *AASHTO Pavement Design Guide*. This layer coefficient represents the behavior of the material relative to overall pavement performance, in terms of serviceability, at the AASHTO Road Test. It cannot be used to explain the load carrying characteristics of the pavement with respect to fatigue, rutting, and temperature cracking. Thus, a newer approach to pavement design is needed—the mechanistic–empirical approach.

The concept of using a mechanistic approach to pavement design is not new. Techniques for calculating stresses in asphalt pavements have been around for over 60 years. Using these tools for design dates back to the 1960s, although wider development and implementation started in the 1980s and 1990s. States such as Washington, Kentucky, Illinois, and Minnesota are in the process of adopting mechanistic design procedures, and under the auspices of the NCHRP, work is proceeding on the development of a new mechanistically-based *AASHTO Pavement Design Guide*.

In mechanistic design, the principles of physics are used to determine a pavement's reaction to loading. Knowing the critical points in the pavement structure, one can design against certain types of failure or distress by choosing the right materials and layer thicknesses. In the case of the perpetual pavement, it would consist of providing enough stiffness in the upper pavement layers to preclude rutting and enough total pavement thickness and flexibility in the lowest layer to avoid fatigue cracking from the bottom of the pavement structure.

Monismith and Long have suggested that the limiting tensile strain at the bottom of the asphalt layers should be no greater than 60 $\mu\varepsilon$, and that at the top of the subgrade the vertical strain should be limited to 200 $\mu\varepsilon$ (*4*). Asphalt thickness proposed in other procedures shows these strain levels to be reasonable (*3*, *5*).

In order to initially implement this design procedure, assumptions will have to be made regarding the expected performance of the pavement system. This can be accomplished using existing performance relationships for fatigue failures as done by Von Quintus (*5*), although they would be somewhat conservative. More precise estimates of rutting in the surface and top-down fatigue cracking will have to be developed over time to improve the design concept and avoid overly conservative pavement designs.

MATERIAL CONSIDERATIONS

Since the hot-mix asphalt (HMA) pavement will be tailored to resist specific distresses in each layer, the materials selection, mix design, and performance testing will need to be specialized for each layer material. The mixtures' stiffnesses will need to be optimized to resist rutting or fatigue cracking, depending upon which layer is being considered, and durability will be a primary concern for all layers.

Base Layer

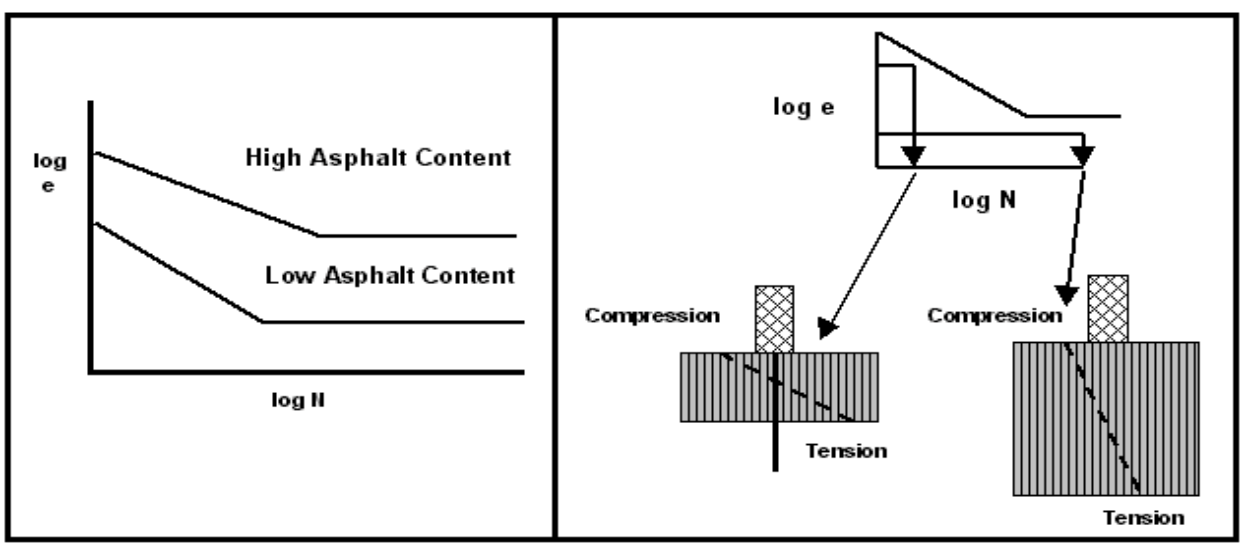
The base layer of HMA must resist the tendency to fatigue crack from bending under traffic loads. The main mixture characteristic which can help guard against fatigue cracking is a higher asphalt content (Figure 2a). The use of a finer aggregate gradation can also make a mixture more fatigue resistant. These characteristics, in combination with an appropriate total asphalt thickness, will provide insurance against fatigue cracking from the bottom layer (Figure 2b). Durability of this layer will be provided primarily by the higher asphalt content.

The mix design for this layer can be accomplished using Superpave guidelines for lower pavement layers. The AC should be defined as that which results in a high in-place density. The asphalt grade used in this layer should be high enough to provide protection against rutting at this layer in the pavement. The low-temperature characteristics should be the same as those of the intermediate layer. If this layer is to be opened to traffic during construction, provisions should be made for rut testing the material. Performance testing for the material in this layer should include a fatigue or stiffness test as well as a moisture susceptibility test.

Another approach to ensuring the fatigue life is to design a thickness for a stiff structure such that the tensile strain at the bottom of the asphalt layers is insignificant. This would allow for the use of a single-mix design in the base and intermediate layers, precluding the need to switch mix types in the lower pavement structure.

Intermediate Layer

The intermediate or binder layer must combine the qualities of stability and durability. Stability in this layer can be obtained by achieving stone-on-stone contact in the coarse aggregate along with using a binder with an appropriate high-temperature grading. The internal friction provided by the aggregate can be obtained by using crushed stone or gravel and ensuring an aggregate skeleton by testing for the voids in coarse aggregate (*6*). One option would be the use of a large



(a) **(b)**

FIGURE 2 Fatigue resistant asphalt base: (a) improve fatigue resistance with high asphalt content mixes, and (b) minimize tensile strain with pavement thickness (log e = log of strain; log N = log of number of cycles to failure).

nominal maximum size aggregate (37.5 mm), but the same effect could be achieved with smaller aggregate sizes so long as stone-on-stone contact is maintained. The high-temperature grade of the asphalt should be the same as the surface to resist rutting. However, the low-temperature grade could probably be relaxed one grade, since the temperature gradient in the pavement is relatively steep and the low temperature in this layer would not be as severe as the surface layer (Figure 3). The mix design should be a standard Superpave approach, and the design asphalt content should be the optimum. Performance testing should include rut testing and moisture susceptibility. Although a test for fundamental permanent deformation properties is currently being developed in a NCHRP project, it is recommended that a rut testing device be used in the interim to evaluate mixtures in order to protect against early rutting.

Wearing Surface Layer

The wearing surface requirements depend on local experience and economics. In some cases the need for rutting resistance, durability, impermeability, and wear resistance dictate the use of a stone matrix asphalt (SMA). This might be especially true in urban areas with high truck traffic volumes. Properly designed and constructed, a SMA will provide a stone skeleton for the primary load carrying capacity, and the matrix (combination of binder and filler) gives the mix additional stiffness and impermeability. The matrix can be obtained by using a polymer-modified asphalt, relatively stiff unmodified binder with fibers, or an asphalt binder in conjunction with specific mineral fillers. Maryland, Georgia, and Wisconsin have had great success in applying SMAs on high-volume roadways. Durability can be achieved by minimizing the voids in the in-place mixture.

In instances where the overall traffic is not as high or in cases where the truck traffic is lower, the use of a well-designed, dense-graded Superpave mixture might be more appropriate.

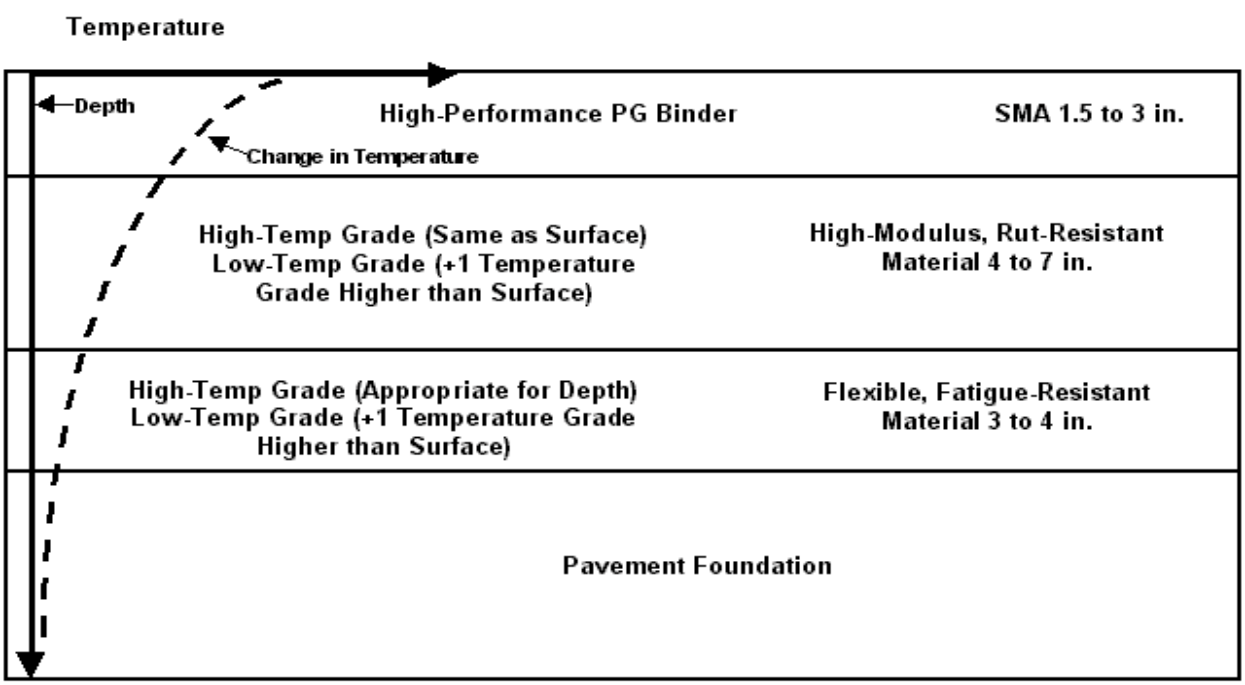


FIGURE 3 Impact of temperature gradient on asphalt grade (PG = performance grade).

As with the SMA, it will be necessary to design against rutting, permeability, weathering, and wear. It is recommended that some type of performance test be done during mixture design; at a minimum, this should consist of rut testing.

Depending upon climate, to avoid rutting the performance grade should be bumped to at least one high temperature grade greater than normally used in an area. The low temperature grade should be that normally used in the area for perhaps a 95 or 99 percent reliability in resisting thermal cracking.

Likewise, by minimizing the total voids in the mixture, the impermeability of the mix is ensured. Wear resistance may be provided by using a high-quality aggregate with a low-polish value. The stiffness of this layer is critical in obtaining a 20-year surface without rutting. The aggregate structure should be evaluated using the approach described for the intermediate layer, and performance testing should consist of rut testing at least.

CONSTRUCTION

Construction of this type of pavement will require great attention to detail and a commitment to build with quality from the bottom up. In the process of building the roadway, modern methods of testing should be employed to give continuous feedback on the quality of materials and construction.

The foundation must be able to support paving and compaction operations. Materials for this layer may include sand or sandy-gravel subgrades, stabilized fine-grained subgrade, unstabilized or stabilized granular base materials, or rubblized concrete. Thus, this layer must be well compacted, smooth, and stiff enough to support construction traffic and provide resistance to rollers. Although no specific guidance exists on what the construction stiffness of the underlying layer should be, this should be relatively simple to determine with tools such as the dynamic cone penetrometer.

In service, one objective would be to minimize volume changes in the foundation layer due to swelling soils or frost heave. Local experience would best dictate how to handle these situations by, for instance, the use of stabilizers, overburden, or soil mixing. Weakening of soils during certain seasons of the year would also need to be addressed, and it might be necessary to provide drainage or a granular interlayer to ensure a consistent foundation during the service life. Nunn suggests a minimum design modulus value of about 50 kPa for the foundation layer (*3*).

With modern HMA pavements, good construction practices ensure good performance. With the possible use of polymer-modified asphalts, it will be critical to avoid overheating the binder in the construction process. New industry guidelines are being developed to ensure the proper handling and application of polymer-modified asphalt binders. Segregation in coarse aggregate mixtures is another area of concern, but again, proper handling of the material during manufacture, transport, and laydown can prevent the problem. Segregation may be measured with infrared temperature techniques and laser texture methods such as the Rosan procedure (*7*). Achieving density in the various layers of HMA can be done by following the lessons learned during the implementation of Superpave and the successful applications of SMA (*8, 9, 10*).

Volumetric control of the mixtures by the contractor will be the key to consistency and quality in the final product. The contractor should have access to a fully equipped and staffed quality control laboratory. Periodic testing and data analysis with good quality control and inspection techniques will ensure that the desired characteristics will be imparted to the pavement. Nuclear methods of testing may be used for the assessment of in-place density, thickness can be continuously monitored with ground penetrating radar, and smoothness can be evaluated with new lightweight profilometers.

PERFORMANCE MONITORING AND RESURFACING

To maintain the pavement in its optimal state, it will be necessary to periodically monitor its performance. The chief idea is to keep all forms of distress in the top few inches of the HMA. Thus, distresses such as top-down fatigue cracking, thermal cracking, rutting, and surface wear may be confined to no deeper than the original thickness of the wearing course. Once the distresses have reached a predetermined level, the surfacing would be programmed and an evaluation of the pavement structure would be undertaken.

It will be important to identify the distress types and levels that should trigger the resurfacing activity. Annual surveys of pavement distress and ride quality will need to be conducted to monitor surface conditions and to track deterioration with time. The structural evaluation would be accomplished by thickness verification using either cores or ground-penetrating radar and by deflection testing. The cores and radar could also be used to indicate problems with moisture susceptibility. The design assumptions inherent in the original design can be verified through deflection testing and interpretation by backcalculation of layer moduli. In the event of changes such as a weakening of the underlying soil through increased moisture content, a slight additional thickness may be planned for the resurfacing to ensure the perpetual nature of the structure.

The first step in the resurfacing process will be the removal of the existing surface to the depth of the distress. This could vary between 1 and 4 in. of milled depth. The milled material would be replaced, and, if needed, a slight additional thickness could be placed. This layer would need to have the same characteristics as the original surface, i.e., rut resistance, durability, thermal cracking resistance, and wear resistance. If new and more promising asphalt surfacing materials are available in the future, they could be employed in the resurfacing. Essential to the performance of the resurfaced pavement is the assurance of bonding between the new wearing course and the existing pavement. A tack coat will be needed to ensure this bond. Pavement monitoring and the programming of future resurfacing would proceed as before.

SUMMARY

The perpetual pavement offers engineers the ability to design for specific modes of distress. Resistance to bottom-up fatigue cracking is provided by the lowest asphalt layer having a higher binder content or by the total thickness of pavement reducing the tensile strains in this layer to an insignificant level. The higher binder content in this layer will also provide durability. The intermediate layer will provide rutting resistance through stone-on-stone contact and durability through proper selection of materials and a greater film thickness on the aggregates. The uppermost structural layer will have the qualities of resistance to rutting, weathering, thermal cracking, and wear, and these may be provided by SMAs or dense-graded Superpave mixtures. An open-graded friction course may be employed in some areas to promote surface drainage and act as a renewable surface.

The knowledge and engineering capability to design and build such a structure exists. However, this information and the design procedure must be synthesized into a useful system for engineers. Also, refinements need to be made to ensure that the concept may be employed at a variety of local levels. It has been suggested that an international team of experts be assembled to accomplish this task. They would be responsible for producing broad guidelines for materials selection, mixture design, pavement design, construction, performance monitoring and

resurfacing. It is expected that individual countries and regions would then tailor the process according to their needs.

The long lasting pavement is a valid concept, and it is gaining national and international momentum. The development of guidelines for pavement design, materials selection and construction could be relatively rapid. Validation of the pavement design procedure and refinement of the materials selection process would need to occur over a longer time period.

REFERENCES

1. Brown, E. R., and S. A. Cross. *A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements.* Report No. 92-5, National Center for Asphalt Technology, Auburn University, Ala., 1992.
2. Baker, M. J., and J. P. Mahoney. *Identification and Assessment of Washington State Pavements with Superior and Inferior Performance.* Report No. WA-RD 437.1, Washington State Department of Transportation, Olympia, 2000.
3. Nunn, M. E., A. Brown, D. Weston, and J. C. Nicholls. *Design of Long-Life Flexible Pavements for Heavy Traffic.* Report No. 250, Transportation Research Laboratory, Berkshire, U.K., 1997.
4. Monismith, C. L., and F. Long. *Overlay Design for Cracked and Seated Portland Cement Concrete (PCC) Pavement—Interstate Route 710*. Technical Memorandum TM UCB PRC 99-3, Pavement Research Center, Institute for Transportation Studies, University of California, Berkeley, 1999.
5. Von Quintus, H. L. *Pavement Structural Design Study: A Simplified Catalog of Solutions.* Report No. 3065, Fugro-BRE, Inc., Austin, Tex., Dec. 2000.
6. Vavrik, W. R., W. J. Pine, G. Huber, S. H. Carpenter, and R. Bailey. The Bailey Method of Gradation Evaluation: The Influence of Aggregate Gradation and Packing Characteristics on Voids in the Mineral Aggregate. *Asphalt Technology*, Vol. 70, Association of Asphalt Paving Technologists, St. Paul, Minn., 2001.
7. Stroup-Gardiner, M., and E. R. Brown. *NCHRP Report 441: Segregation in Hot-Mix Asphalt Pavements.* TRB, National Research Council, Washington, D.C., 2000.
8. *Special Report 180: Superpave Construction Guidelines*. National Asphalt Pavement Association, Lanham, Md., 1998.
9. *Designing and Constructing SMA Mixtures: State-of-the-Practice.* Quality Improvement Series 122, National Asphalt Pavement Association, Lanham, Md., 1999.
10. Brown, E. R., R. B. Mallick, J. E. Haddock, and J. Bukowski. *Performance of Stone Matrix Asphalt (SMA) Mixtures in the United States.* Report No. 97-1, National Center for Asphalt Technology, Auburn University, Ala., 1997.

Development and Uses of Hard-Grade Asphalt and of High-Modulus Asphalt Mixes in France

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Hard-grade paving asphalts, i.e., those having a penetration at 25°C lower than 25 mm/10, have experienced in France a significant development over the past 20 years. A brief review of the evolution of asphalts in France is provided, mentioning the current recommendations for the selection of asphalts and the rheological characteristics of the different hard asphalts produced in France. Data on the effect of rolling thin-film–oven and pressure-aging–vessel tests are given. Then high-modulus asphalt mixes are discussed. The different types of application of hot-mix asphalt concrete (HMAC) are indicated. Information is given about the composition of HMAC together with the current specifications in Association Française de Normalisation standards. Results are presented with respect to compactibility with the gyratory shear compactor, stiffness, resistance to rutting and to fatigue cracking, and about field performance. Two examples illustrate the potential economical benefit when making use of HMAC in new pavements instead of conventional base asphalt concrete.

Hard-grade paving asphalts, i.e. having a penetration at 25°C lower than 25 mm/10, have experienced in France a significant development over the past 20 years. They have been developed to provide technical solutions to the problem of mitigation of rutting of surface layers and to increase the rigidity of the base courses of asphalt pavements. The production of hard-grade asphalt in France was 39,000 tons in 1990, 77,000 in 1995, and reached 100,000 tons in 2000. This placed France in a leading position for the use of this type of binders according to the survey carried out by the Permanent International Association of Road Congresses (PIARC)—World Road Association in 1998. Though the general objectives guiding the use of hard asphalts were identified a long time ago, it took several years before methods of processing led to binders which exhibit the desired hardness together with good long-term performance in the field. The development of hard asphalts is also closely related to the parallel emergence of new asphalt mixtures designed especially for these binders in order to obtain the performances sought in pavements.

This paper is organized in two parts. The first one deals with hard asphalts and provides a short history of the evolution of paving asphalts in France, followed by a presentation of data on rheological characteristics and results of aging tests for the hard asphalts produced in France. The second part is devoted to the high-modulus asphalt mixtures. The uses of these pavement materials are presented. Indications are given on the composition of the mixes and on their main mechanical characteristics.

HARD ASPHALTS

Evolution of Asphalts in France in Brief

The 1960s: Early Stages and First Changes

Until the beginning of the 1960s in France, almost all paving asphalts were 80/100 and 180/220 penetration grade (PG). They were produced by direct distillation from heavy crudes imported from Central America. With a strong increase in heavy vehicle traffic at that time, the search for a higher rigidity and a better resistance of the asphalt mixes to plastic deformations (rutting) led to the use of harder asphalts. Production of 40/50 and 60/70 PG asphalts started in 1966 and the first 20/30 PG appeared in 1968 (*1*). The publication, by the French Road Directorate, in September 1969, of a directive for the construction of the surface courses in asphalt mixes, had a decisive role in this evolution. This directive introduced a climatic criterion for the choice of the asphalt, with the division of the French territory in three zones:

- Zone 1: Mediterranean type of climate characterized by mild winters and hot summers;
- Zone 2: Oceanic type of climate, characterized by both mild winters and summers; and
- Zone 3: Continental type of climate, characterized by cold winters and hot summers.

Recommendations made at the time for the choice of the asphalt are summarized in Table 1. In order to further increase the resistance to rutting of surface layers, the technique of air-blowing was used to reduce the thermal susceptibility of the asphalt, at the end of the 1960s, for the production of 40/50 PG asphalt with a high penetration index (*2*). If this proved to be effective with respect to rutting, on the other hand it led very quickly to extensive cracking at the pavement surface, which brought to the abandonment of this solution.

The explanation of this poor performance is to be linked to the effect of the manufacturing process on the structure of the asphalt. Air-blowing at high temperature produces a dehydrogenation of certain molecules followed by their reticulation. It results in an increase of the size of the naphteno–aromatic resins. If one refers to the separation of the components of the asphalt into saturated compounds, aromatics, resins, and asphaltenes, the proportion of asphaltenes increases, that of the resins and aromatics decreases, that of saturated coumpounds remains about constant. This results in an increase of the colloidal instability index:

$$IC = \frac{\text{asphaltenes + saturated}}{\text{aromatics + resins}}$$

TABLE 1 Recommendation for the Choice of Asphalt Grades in Surface Layers (1969 Road Directorate Directive)

	Zone 1	Zone 2	Zone 3
Altitude ≤ 500 m	40/50	40/50 or 60/70	60/70 or 80/100
Altitude > 500 m	60/70 or 80/100*	60/70 or 80/100	80/100

* if altitude >1000 m

This evolution of the structure of the asphalt, in which the asphaltene micelles are not any more completely peptized but are more or less flocculated, leads to a change in behavior from a "sol" type to a "gel" type. The penetration index (IP), which reflects the susceptibility to temperature, then reached values of +2 and more.

The 1970s: Consequences of the Oil Crises

The 1973 and 1979 oil crises deeply modified the French oil market with regard to both the volume of refined oil and the origin of the imported crudes. These crises involved the need for an adaptation of industry, with the closing of several refineries but also the construction of new and more powerful units of distillation. The development of the share of the crudes imported from the Middle East, less heavy than those from Central America, and the demand for 40/50 and 60/70 PG asphalts led to the generalization of the process of partial air-blowing for the production of these grades. This process was applied to rather soft bases which led to rather large chemical changes in the asphalts.

The 1980s: Impact of Specifications on the Evolution of Asphalt Processing Methods

The revision of the system of specification for paving asphalts, initiated at the beginning of the 1980s by the French Road Administration with participation of the oil industry and road contractors, has had an important impact on the processing methods. Earlier French specifications were primarily based on the penetration value (the other physical characteristics were only supplementing this information). In 1986, however, the ring and ball (R&B) softening point was introduced as a cornerstone of the new system of grading. The concern over the impact of aging on the field performance of the asphalt mixes led to a second development. A large experimental program involving 100 job sites made it possible to collect relevant data to assess representativeness of the rolling thin-film oven test (RTFOT) for simulation of aging during coating and laying. According to field performance of 35/50 and 50/70 PG asphalts used in wearing courses, specifications were fixed which limit hardening of the binder after RTFOT, with on the one hand a maximum increase in 9°C of the R&B softening point and on the other hand a minimal value of residual penetration (function of the paving grade). These requirements were adopted in 1990 by the asphalt producers and were introduced, in 1992, in the Association Française de Normalisation (AFNOR) standard for paving asphalts.

The 6°C interval for the R&B softening point for the definition of the width of each grade, as well as the requirements on the limitation of aging after RTFOT led the asphalt producers to adapt the methods of production.

The possibility of obtaining a high point of cut was made possible by the use of structured packing internals in the vacuum distillation column. It became thus possible to eliminate fractions with a point of distillation under atmospheric pressure up to 550 °C. Contrary to the process of air-blowing which results, by reticulation, in an increase of the components which already have a large mass, direct distillation involves elimination of the saturated and naphteno-aromatic components of lower masses. This process is applicable only to the heavy crudes which already contain enough asphaltenes. The resulting hard asphalts have a penetration index less than 2 and are less susceptible to aging. Partial air-blowing remains used, in France, by one producer for the hardest grades.

Production of propane-precipitated asphalt, which is a process well adapted to the production of very hard grades, is only used in France in two refineries specializing in the

production of lubricants. All in all, asphalts resulting from this process represent only approximately 5 percent of the French production of asphalts (*3*).

Current Recommendations for the Selection of Paving Asphalts

Recommendations for the choice of plain paving grades were redefined in 1994 in a technical guide issued by the Laboratoire Central des Ponts et Chaussées (LCPC) and the Societe d'Etudes Techniques des Routes et Autoroutes (*4*). As in the above-mentioned 1969 directive, three climatic zones are specified; however, they are based on average values of maximum daily temperatures in July and August and minimum daily temperatures in January and February observed over the past 30 years:

- Zone 1: Dominant oceanic climate ($T_{max} \leq 27°C$ and $T_{min} \geq 0°C$);
- Zone 2: Dominant southern climate ($T_{max} > 27°C$ and $T_{min} \geq 0°C$); and
- Zone 3: Dominant continental or mountainous climate ($T_{min} < 0°C$).

In order to relate France's climatic conditions to the SUPERPAVE approach, Figure 1 shows two maps of France for low and high temperatures with indication of the penetration grade (PG) for the asphalt binder.

For base asphalt concrete (AC), the grade most generally selected is 35/50 if not 50/70. For wearing courses, in the case of heavy traffic, the advised grades are indicated in Table 2 (in the case of less severe traffic conditions, one would accept a softer grade 50/70 instead of 35/50 and 70/100 above 1000 m).

The use of a harder grade is to be considered in the case of very demanding situations (important slow or channeled heavy traffic or high temperatures) or when there are constraints on how deep or high the pavement can be.

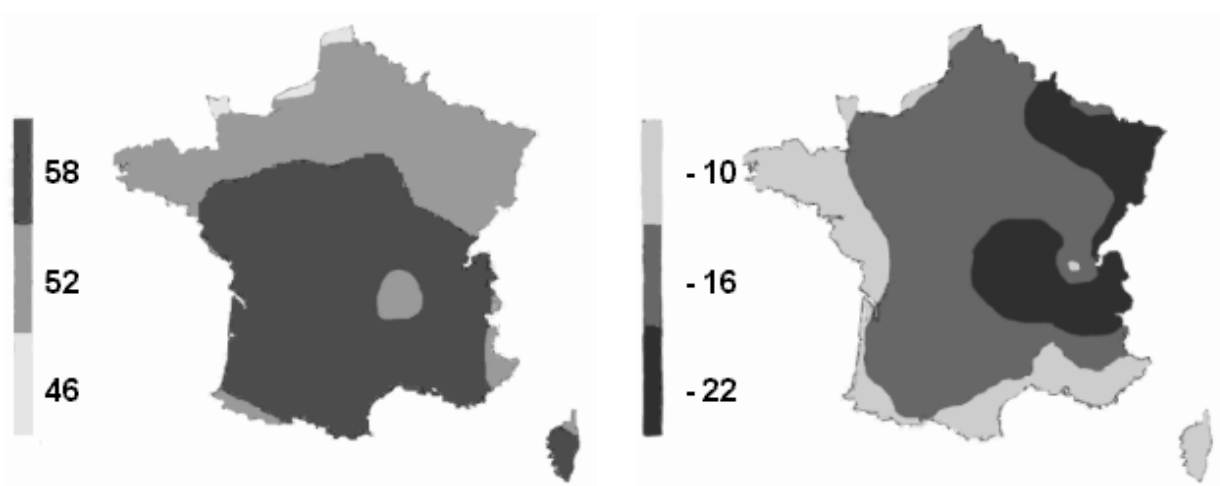

FIGURE 1 Maps of France PG pavement temperatures.

TABLE 2 Plain Asphalt Paving Grades for Wearing Courses in Cases of Heavy Traffic (1994 Recommendations for the French National Network)

Type of Climate	1	2	3
Altitude < 500 m	35/50	35/50	35/50
Altitude from 500–1000 m	50/70	50/70	50/70
Altitude > 1000 m	*of no concern*	50/70	70/100

Rheological Characteristics of Hard Asphalts

Hard asphalts are defined here like having a penetration less than 25 mm/10 at 25°C. There are three grades: 15/25, 10/20, and 5/10. Currently, grade 5/10 is at a trial stage, whereas the others have been marketed for several years. Typical characteristics of these hard asphalts are given in Table 3.

As an example, Figures 2 and 3 present the results of dynamic shear tests on four different PG, 10/20, 25/35, 35/50, and 50/70: master curves of the complex modulus G^* at 20°C (Figure 2) and the phase angle at 7.8 Hz.

Penetration at 25°C does not determine of course the rheological characteristics. Performances, for asphalts of the same PG, vary depending on the crude and the manufacturing process. Table 4, extracted from Glita and Conan (*5*), compares the rheological characteristics of seven 10/20 PG asphalts with, as a reference, those of a plain 35/50 asphalt. One notes that the modulus can vary, for the same test conditions, in a ratio from 1 to 2, and that the sensitivity to permanent deformation, as indicated by $G^*/\sin\delta$, is definitely different from one asphalt to another.

One can see from this table that the temperature for which the viscous and elastic components of the modulus are equal (phase angle equal to 45°) is definitely lower for the 35/50 asphalt than for 10/20 asphalts. The hard asphalts thus should have a lower capacity of healing than the softer 35/50 asphalt.

Brittleness at low temperature can be estimated from the temperature corresponding to the maximum of the viscous component G'' of the complex modulus, or by the value of the phase angle at low temperature, here at –10°C. Table 4 shows, according to these two criteria, that asphalts B, D, and E can be regarded as most fragile, whereas asphalts A, F, and G show characteristics similar to those of the 35/50 asphalt.

TABLE 3 Typical Hard Asphalt Characteristics (Before Aging)

Grade	15/25	10/20	5/10
R&B softening point (°C)	66	62 to 72	87
Pfeiffer IP	+0.2	+0.5	+1.0
Dynamic viscosity at 170°C (mm²/s)	420	700	980
Complex modulus at 7.8 Hz, $\lvert E^*\rvert$, (MPa)			
Complex modulus at 0°C	425	700	980
Complex modulus at 10°C	180	300	570
Complex modulus at 20°C	70	110	300
Complex modulus at 60°C	0.4	0.7	7

NOTE: IP = penetration index; $\lvert E^*\rvert$ = complex Young modulus.

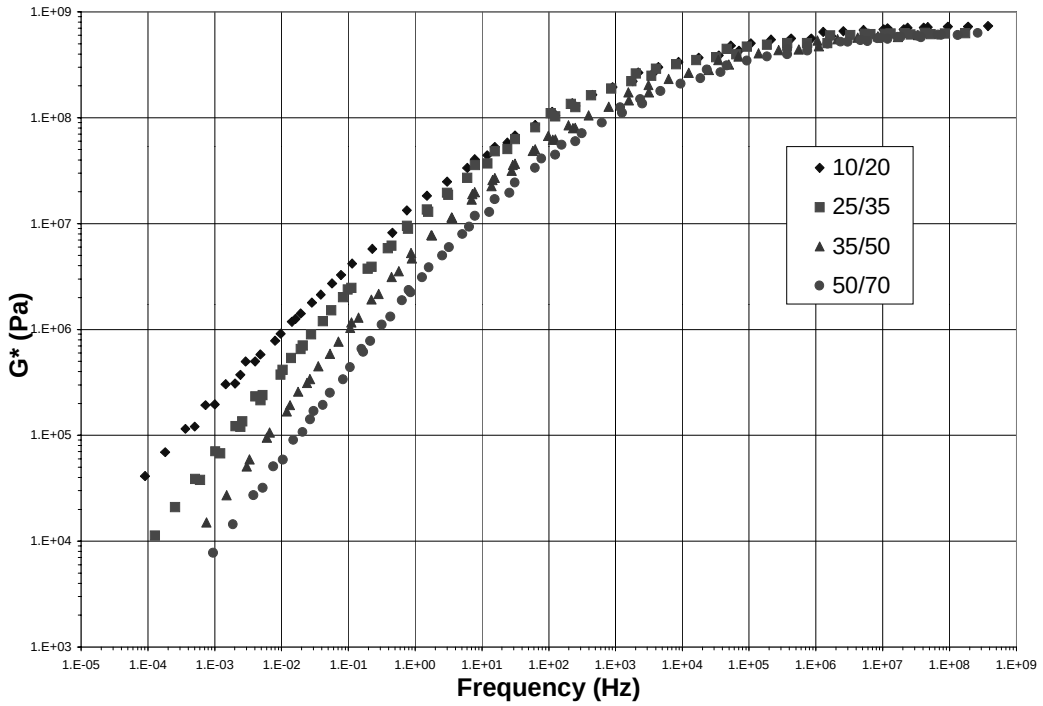

FIGURE 2 Master curves of complex modulus *G of four different asphalts: 10/20, 25/35, 35/50, and 50/70 penetration grades.**

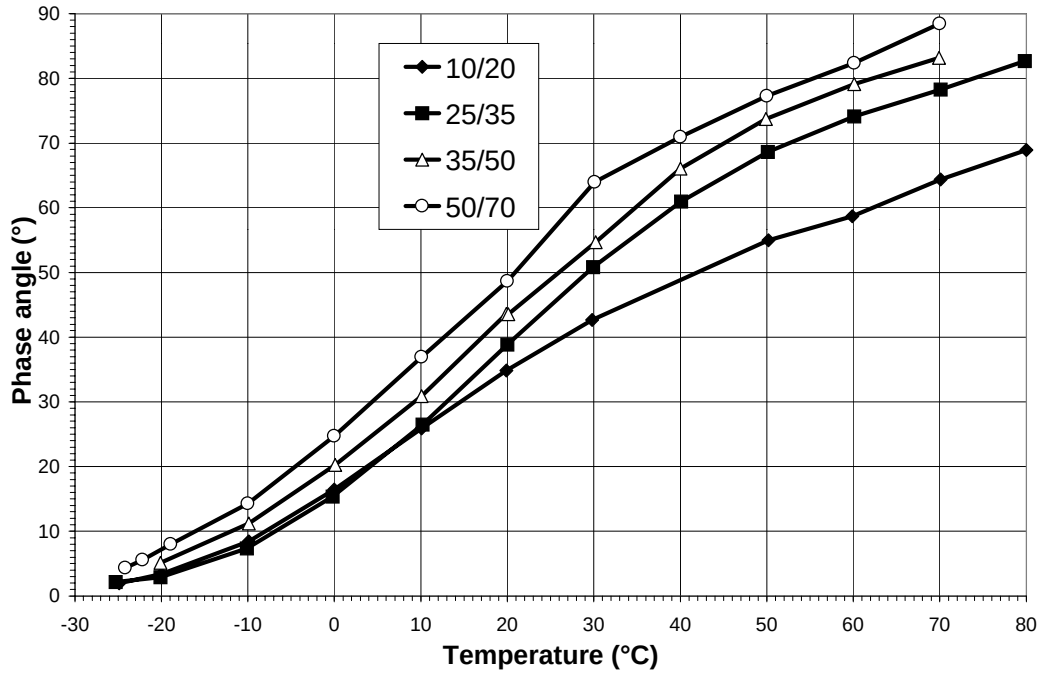

FIGURE 3 Variation of phase angle with temperature at 7.8 Hz for four different asphalts: 10/20, 25/35, 35/50, and 50/70 penetration grades.

TABLE 4 Rheological Characteristics of Seven 10/20 Asphalts and a 35/50 Asphalt

	35/50	A	B	C	D	E	F	G
$\lvert G^* \rvert$ (15°C; 10 Hz) (MPa)	34.5	53.7	88	88	83.7	71.1	43.7	47.3
$\lvert G^* \rvert$ / sinδ (60°C; 5 Hz) (MPa)	0.016	0.131	0.184	0.247	0.122	0.165	0.184	0.103
SR	3.55	4.3	3.64	3.94	3.3	3.58	4.53	4.1
T (°C) pour $G' = G''$	17	28	29	31	24	26	34	25
T (°C) pour G'' max	–10	–10	0	–5	0	–5	–15	–10
δδ (–10°C; 5 Hz)	12.2	11.6	7	9.1	6.5	8.3	12.7	11

NOTE: SR = standard deviation of the relaxation spectrum.

Table 5 gathers information included in the French "*Avis techniques*" (French technical assessments for innovative products issued by the "*Comité Français des Techniques Routières*, French Committee for Road Techniques). It presents hard asphalt characteristics, produced in France for use in high modulus AC. These values cannot be guaranteed because they are likely to vary with the origin of the crude oil.

Influence of Aging

The incidence of aging, as simulated by the RTFOT for the phases of coating and laying of the asphalt mix, then with the pressure-aging vessel (PAV) for field evolution in the pavement, was studied on various 10/20, 35/50, and 50/70 paving asphalts by the Regional Laboratory of Bridges and Roads of Aix en Provence. It considered composition, the traditional empirical characteristics (penetrability, R&B softening point, Fraass temperature), and the rheological behavior from bending beam rheometer (BBR) tests.

From the point of view of the change in the composition of the asphalt, if one considers the n-heptane asphaltenes content, the increase in asphaltenes after aging tests is all the more larger since the grade of the asphalt is soft. After RTFOT + 20 h of PAV, the ranges are from 50 to 80 percent for 50/70 asphalts, from 40 to 65 percent for 35/50, and from 15 to 35 percent for 10/20 grade. Iatroscan chromatography, which provides a global analysis without preliminary separation of asphaltenes, gives a more complete image of the evolution of the generic components of the asphalt. The change is qualitatively comparable for the various grades, namely:

- The proportion of saturated remains about constant;
- The proportion of aromatics varies little after RTFOT, but the decrease is important after RTFOT + PAV and results in a transformation into resins; and
- The proportion of asphaltenes increases slightly, much less than the change observed with the n-heptane precipitation method (the difference comes from the fact that certain resins are dragged by the solvent in one case while in the other case they are precipitated).

With respect to the rheological behavior at low temperature, the BBR test shows little influence of RTFOT on the temperatures of iso-modulus 300 MPa and m = 0.300. On the other hand, the effect of RTFOT + PAV is important. For the hard asphalts tested, the magnitude of the changes is comparable with that observed on softer grades 35/50 and 50/70, namely a rise in 2°C to 3°C for the temperature of iso-modulus 300 MPa and 4°C to 7°C for the temperature of iso-slope m = 0.300.

TABLE 5 Data on Hard Asphalts Produced in France for High-Modulus AC

Avis Technique	26	96	76	22	72	42	40
Asphalt Before Aging							
Penetration (0.1 mm) at 25°C	16	21	21	12	13	13	16
R&B (°C)	63.5	66	68	72	66	65	71
IP (LCPC)	+0.7	+1	+1.3	+0.4	+0.4	–0.2	+0.5
Fraass temperature (°C)	–6	–8	–6	–5	–6	+3	–3
Modulus E (MPa) (7.8 Hz; 25°C)	54	40	34	60	56	61	66
Phase angle (°) (7.8 Hz; 25°C)	37	39	38	35	29	34	—
Modulus E (MPa) (7.8 Hz; 60°C)	0.6	0.6	0.5	0.9	0.9	0.6	1
Phase angle (°) (7.8 Hz; 60°C)	64	62	63	62	64	64	59
Modulus E (MPa) (250 Hz; 60°C)	6	6	5	8	9	7	10
Phase angle (°) (250 Hz; 60°C)	63	56	57	59	60	67	61
Asphalt After RTFOT							
Penetration at 25°C	11	17	18		7/13		
Residual Penetration (%)	69	83	86				
R&B (°C)	75	72	74		62/76		
Increase in R&B (°C)	11.5	6	6				
Fraass temperature (°C)	–4	–6	–6		0/+4		
Increase in Fraass temperature (°C)	+2	+2	0				
Modulus E (MPa) (7.8 Hz; 25°C)	71	39	39				
Phase angle (°) (7.8 Hz; 25°C)	28	35	36				
Modulus E (MPa) (7.8 Hz; 60°C)	1.2	0.72	0.7				
Phase angle (°) (7.8 Hz; 60°C)	60	58	58				
Modulus E (MPa) (250 Hz; 60°C)	10	6	6				
Phase angle (°) (250 Hz; 60°C)	53	54	54		47		

NOTE: LCPC = Laboratoire Central des Ponts et Chaussées.

Tests carried out on two 10/20 asphalts with an initial R&B softening point of 64°C showed, after RTFOT + PAV, a hardening slightly less than for the softer asphalts but nevertheless important: the increase in R&B is 11°C to 13°C, residual penetrability is 45 to 53 percent. The R&B temperatures on aged asphalt exceed here the threshold value of 71°C, which had been correlated with the observation of surface cracking of wearing courses attributed to thermal fatigue. However, these correlations were made on asphalt mixes with 35/50 and 50/70 asphalts (*6*) and cannot be extrapolated directly to the harder grades.

Practical Indications for Use of Hard Asphalts

The higher viscosity of hard grade asphalts necessitates raising the coating temperature; for 10/20 asphalts it will be around 170°C to 180°C. Laying of the mix must be carried out between

150°C and 170°C, approximately, and compaction must in general be carried out at a temperature higher than 140°C. Hence, to achieve correct compaction thin lifts should not be laid when the ambient temperature is low.

HIGH-MODULUS AC

Interest created by the use of hard asphalts—i.e., of grade lower than 25—is reflected by the diversity of the uses which have developed since 1980. The corresponding pavement materials are now standardized under the names of "*enrobé à module élevé* (EME)", for use in base course, and of "*béton bitumineux à module élevé* (BBME)", for use in binder and wearing courses. (Alternatives to the use of hard-grade plain asphalt were also developed during the same period, such as adding asphaltite or polyethylene to 35/50 asphalts, but these solutions will not be developed in this paper.)

Field of Application of Hot-Mix Asphalt Concrete

The first applications of hot-mix asphalt concrete (HMAC) in 1980 involved reinforcement or rehabilitation projects operating with depth constraints. In urban areas, in particular, buried pipes, curbs, and other thresholds frequently limit the depth of possible excavation.

This problem led road contractors to seek pavement materials having a higher modulus than traditional AC in order to produce thinner layers while having a high fatigue resistance so that the solution offers the same service life without premature structural maintenance operation.

The first material of this kind appeared in 1980 patented under the name of GBTHP ("*Grave Bitume à Très Hautes Performances*"). The first applications on state roads began in 1981 as base courses in reinforcement after or without milling or partial excavation of the old pavement (*7*). However, the number of applications became really significant only after 1985—that is, 4 to 5 years after the very first applications.

The oil crises were also a factor which stimulated the search for solutions reducing the quantity of asphalt while maintaining the performance criteria for the pavement. Thus, HMAC was used not only in rehabilitation works, but also in base courses for new pavements, resulting in an economic benefit over traditional solutions using 35/50 or 50/70 grades. It was also imagined to take benefit from the performance of hard asphalts to allow the use of local aggregates with a weak crushing index. The use of HMAC in base courses of new pavements initially appeared on toll motorways. The 1994 edition of *SCETAUROUTE's Manual of Pavement Design for Motorways* (*8*) considers the use of HMAC in base course on an unbound subbase, or in full-depth asphalt pavements (subbase and base courses in HMAC). The new 1998 edition of the Road Directorate's catalogue of new pavements also considers this use of HMAC (*9*).

To reduce the risk of rutting, HMAC has very often been combined with very thin AC (VTAC) (*Béton Bitumineux Très Mince, BBTM*) as a wearing course. This solution for heavily trafficked pavements benefits from the advantages both techniques:

- The low voids content and high stiffness of HMAC provides protection to the base course and great resistance to rutting; and
- The high surface texture due to the discontinuous grading of VTAC provides high and durable skid resistance.

Because of the hardness of the asphalt and of the low voids content of these mixtures, a wearing course is necessary to get enough surface texture and to ensure the thermal protection of the HMAC base layer.

During the 1980s, the road contractors prompted the production of these hard binders and developed AC mixes to answer the above-mentioned objectives. Specific productions of asphalt not calling upon the old process of blowing (because of the sensitivity to thermal cracking) were started. Because of the level of development reached by HMAC, its diversification among all major road contractors, and the experience gained in pavement construction, it was decided to codify this technique in an AFNOR standard, published in October 1992, under reference NF P 98-140 (*10*).

Characteristics of HMAC

The basic idea of HMAC was to design a mix with a very hard grade asphalt at a high binder content, around 6 percent (ratio by weight of the asphalt to the aggregate), comparable with that of AC used in wearing courses. The hard grade of the asphalt confers a higher modulus to the mix which allows, with equal thickness, to reduce the stresses transmitted to the subgrade; enrichment of the asphalt content makes it possible to increase the compactness of the mix and its resistance to fatigue.

Mix are designed in order to permit laying of lifts 7 to 15 cm thick. Figure 4 shows a typical continuous grading curve for a 0/14 mm HMAC.

Because of the higher binder content, as compared to traditional base AC, HMAC will exhibit lower air voids content for the same number of gyrations in the gyratory shear compaction test, as shown by Figure 5.

The standard distinguishes two classes of performance for HMAC:

- EME 2, corresponding to the first generation of these materials; and
- EME 1, introduced only since 1988, with a reduced asphalt content close to that of traditional base AC. Since these materials exhibit lower durability and resistance to fatigue, they are rather used for the layers subjected to compression, their advantage being resistance to rutting. Their use has, however, hardly developed.

Table 6 summarizes the performance requirements fixed by standard NF P 98-140 for the two classes of HMAC and, by comparison, that of a traditional base AC ("*grave-bitume*" of class 2 of French AFNOR standard NF P 98-138).

As seen from Table 6, for EME2 the requirement for the binder content is a minimum richness factor *K*, of 3.4. This *K* factor is defined by the following equations:

$$K = \frac{TL}{\alpha \sqrt[5]{\Sigma}}$$

where

TL = binder content (ratio by weight of asphalt to aggregate)
α = 2.65/*Gse* (*Gse* effective specific gravity of aggregate)

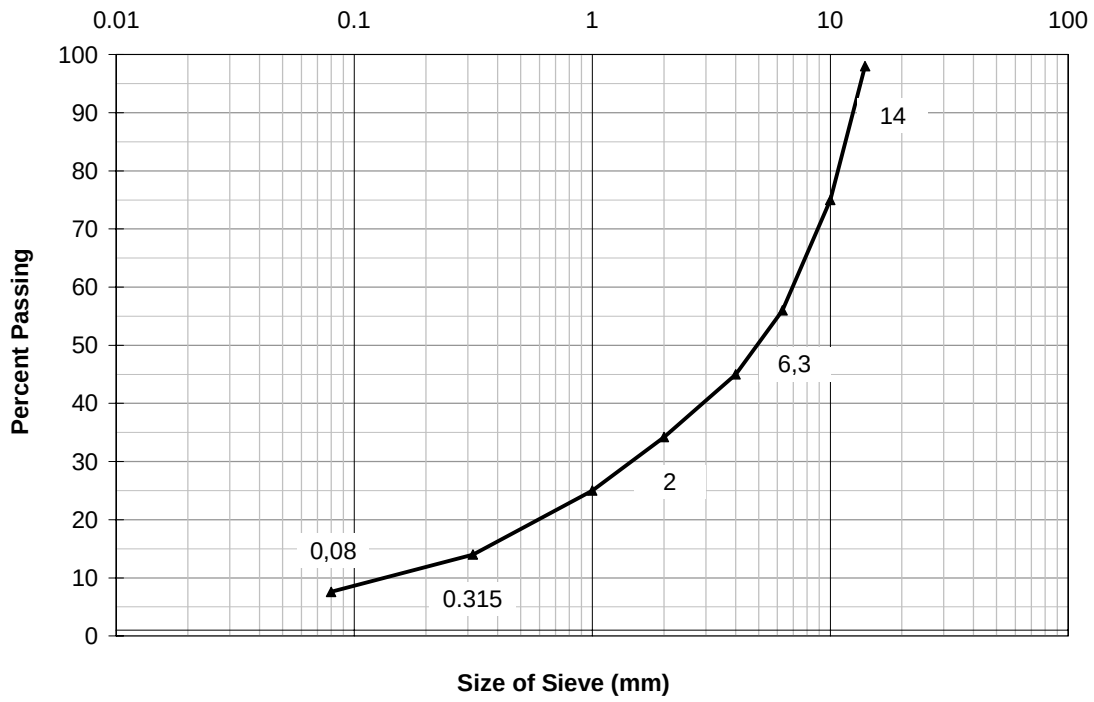

FIGURE 4 Sieve analysis = typical grading for a 10/14mm HMAC with 6.2 percent asphalt content.

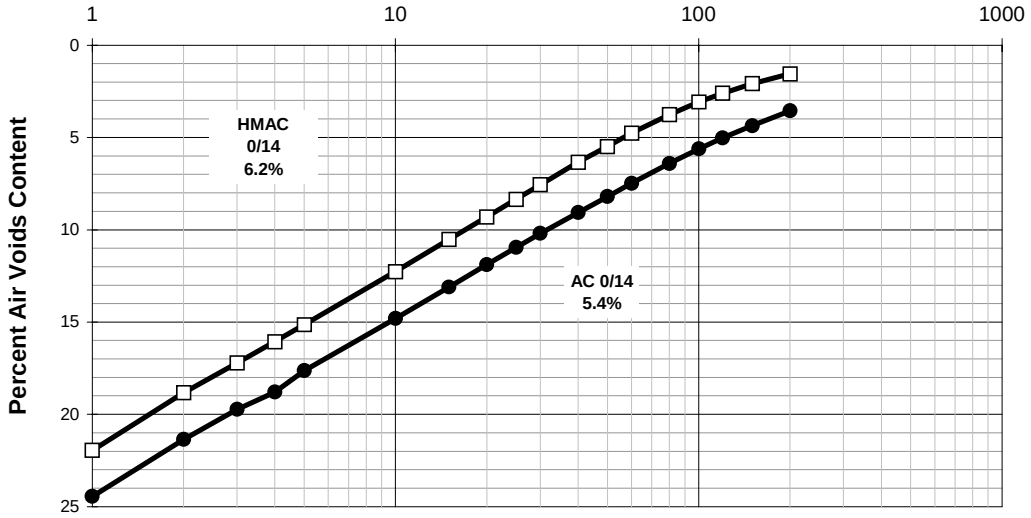

FIGURE 5 Gyratory shear compaction tests: comparison between HMAC and a traditional base asphalt concrete.

$$100\Sigma = 0.25G + 2.3S + 12s + 135f$$

G = % > 6.3 mm
S = % between 6.3 and 0.315 mm
s = % between 0.315 and 0.08 mm
f = % < 0.08 mm

Hence the asphalt content depends on the gradation of the mix. For a 0/14 mm HMAC this leads to a minimum binder content by weight (with respect to the aggregate weight) of about 5.7 percent.

Stiffness

The use of hard asphalt as a binder increases the stiffness of the asphalt mix. This is illustrated by Figure 6, which presents the master curves of the complex modulus E^* determined for a traditional dense asphalt mix made with a 50/70 PG asphalt, a HMAC and the same dense asphalt mix made this time with an Ethylen-Vinyle-Acetate (EVA) modified asphalt.

Resistance to Rutting

It has been known for a long time that there is a narrow correlation, at service temperatures, between the modulus of the binder and that of the asphalt mix and that the increase in modulus of the binder is accompanied by a reduction of the sensitivity to permanent deformations (*11*). This good resistance of HMAC can be judged from the results with the LCPC wheel-tracking rutting tester at 60°C (standard NF P 98-253-1), which is used to fix the French specifications with respect to rutting. A series of tests were carried out to determine the influence of the binder on the performance of asphalt mixes having the same aggregate skeleton. The tests involved 10/20 and 50/70 PG, multigrade-type asphalt, polymer-modified asphalts with 4 percent Styrene-Butadiene-Styrene or 7 percent EVA, and the addition of polyethylene or fibers. Results obtained with the

TABLE 6 Performance Requirements Fixed by Standard NF P 98-140 for HMAC and Comparison with a Base AC "Grave-Bitume" of Class 2

	EME 1	EME 2	GB 2
Granularity and average thickness of lifts	0/10 6 to 10 cm 0/14 7 to 12 cm 0/20 10 to 15 cm	0/10 6 to 10 cm 0/14 7 to12 cm 0/20 10 to 15 cm	0/14 8 to 12 cm 0/20 10 to 15 cm
Minimum richness factor for asphalt content (K)	≥2.5	≥3.4	≥2.5
Binder content for 0/14 grading	≥4.2 pph	≥5.7 pph	≥4.2 pph
Duriez test (r/R)	≥0.70	≥0.75	≥0.65
Wheel-tracking rutting test (60°C, 30,000 cycles)	≤7.5 %	≤7.5 %	≤7.5 %
Modulus (MPa) (15°C, 10 Hz)	≥14,000	≥14,000	≥9,000
Fatigue test ε_6 (10^{-6}) (15°C, 25 Hz)	≥100	≥130	≥80
Max voids content (%)	≤10	≤6	≤11

NOTE: EME = enrobé à module élevé, GB = grave-bitume.

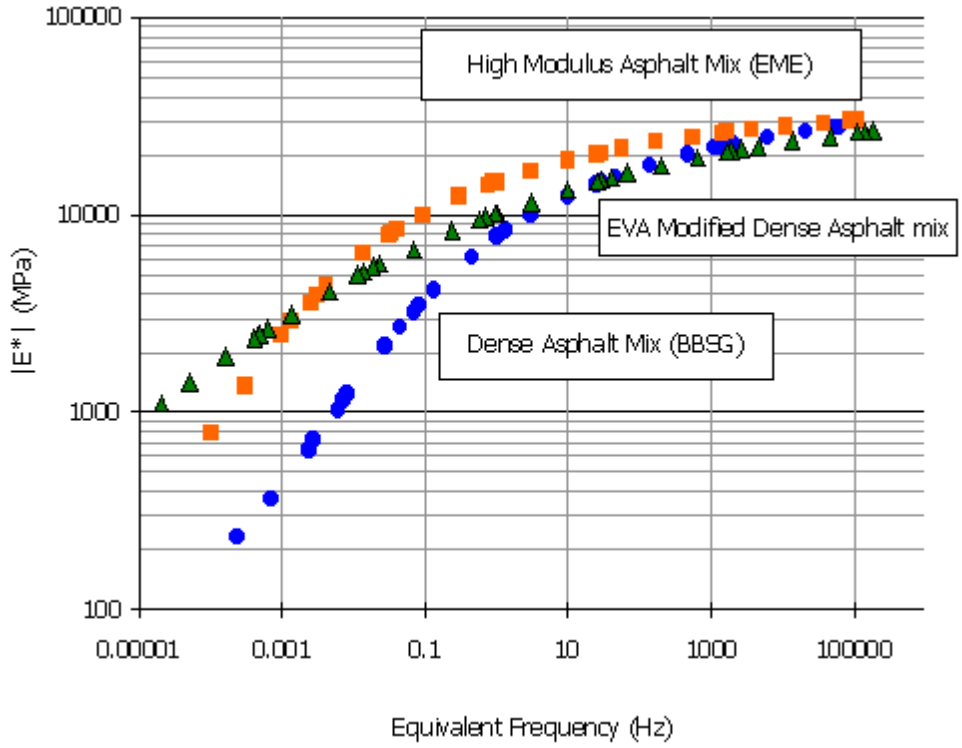

FIGURE 6 Complex modulus master curves on three different asphalt mixes (BBSG = Béton Bitumineaux Semi-Grenu).

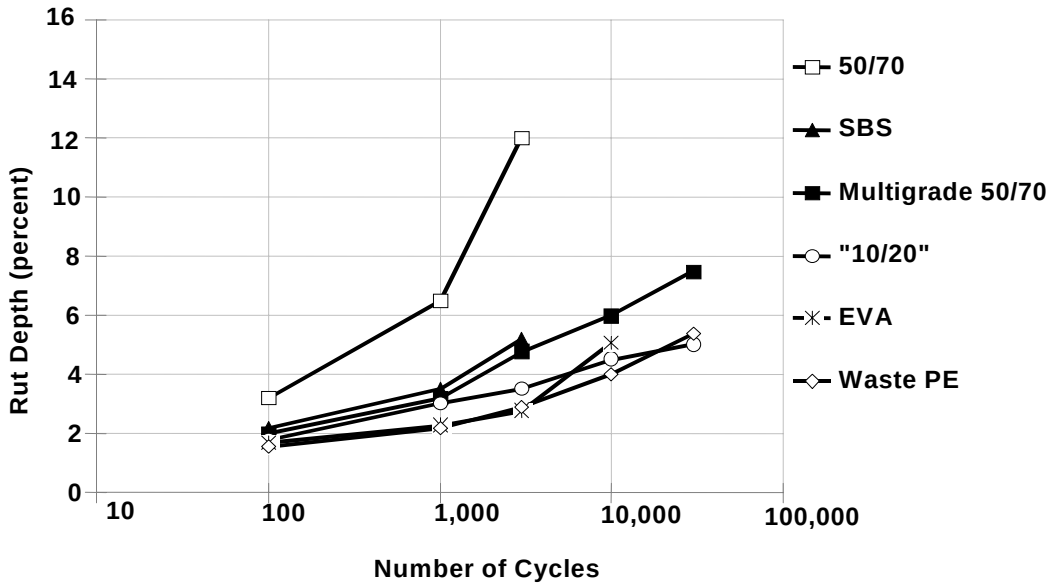

FIGURE 7 Wheel-tracking test results showing the influence of the asphalt binder on the resistance to rutting of similar AC mixes (PE = polyethylene).

FIGURE 8 View of LCPC's accelerated pavement testing facility.

LCPC wheel-tracking rutting tester are shown in Figure 7. The assessments from these laboratory tests have been confirmed by experiments with LCPC's accelerated loading test facility (Figure 8). These full-scale experiments confirmed the very good behavior of HMAC + VTAC in resistance to rutting and in durability of surface macrotexture (*12, 13*).

Resistance to Fatigue

Resistance to fatigue cracking is assessed by the two-point bending test on trapezoidal samples with controlled displacement imposed at the top of the beam. The higher asphalt content together with the lower air voids content in HMAC as compared to traditional base AC provide a better resistance to fatigue as indicated by the increase in the strain for which failure as reached after 10^6 cycles. The HMAC standard requires a minimum value of 130 10^{-6} for ε_6 for EME2 when the requirement is only 80 10^{-6} with a traditional base such as AC GB2 (see Table 6). A typical example of fatigue curves is shown by Figure 9. The mean value of the slope of the fatigue curve found over 16 different HMACs was –1/6 (with values ranging from a minimum of –1/7.5 to a maximum of –1/5.2). The mean value is –1/5 for traditional base AC.

Pavement Design with HMAC

The French pavement design method (*4*) can be described as a rational approach which makes use of a mechanical model together with the results of complex modulus and of two-point bending fatigue tests. Taking into account certain simplifications of the model and the approximate character of the fatigue laboratory test, the calculation of the working strains $\varepsilon_{t,ad}(N)$ uses a shift factor k_c which is derived from an adjustment between model predictions and monitoring of in-service pavements.

$\varepsilon_{t,ad}(N) = \varepsilon_6 f(N) k_c$

where N stands for the number of load cycles to failure and ε_6 is the applied strain at failure after a million cycles in the two-point bending fatigue test.

For the traditional techniques, which have been under a long period of observation (i.e., traditional pavement structures have sustained traffic for at least as long as the project service life), the shift factor was determined by consideration of a representative set of monitored pavements. In the case of the introduction of a new technique like HMAC, design of such pavements was made possible by using the results of accelerated loading tests. To this end, LCPC carried out a series of three experiments between 1990 and 1994 in cooperation with the toll-motorways companies association. Each trial ring comprised four relatively thin asphaltic base layers (8 to 12 cm) resting on an untreated subbase. The results of these experiments (development of cracking versus the number of load cycles) were analyzed in a manner relating the behavior of the various sectors (*14*). From these experiments it was concluded that a lower shift factor should be used for HMAC than for pavements with traditional base AC (respectively 1 and 1.3). However, because of the large difference in fatigue resistance, the working strain of HMAC will still be larger. Combining k_c and ε_6 values from Table 6 gives for traditional base; AC GB2, $\varepsilon_{t,ad}(N) = 104f(N)$; and for HMAC, $\varepsilon_{t,ad}(N) = 130f(N)$.

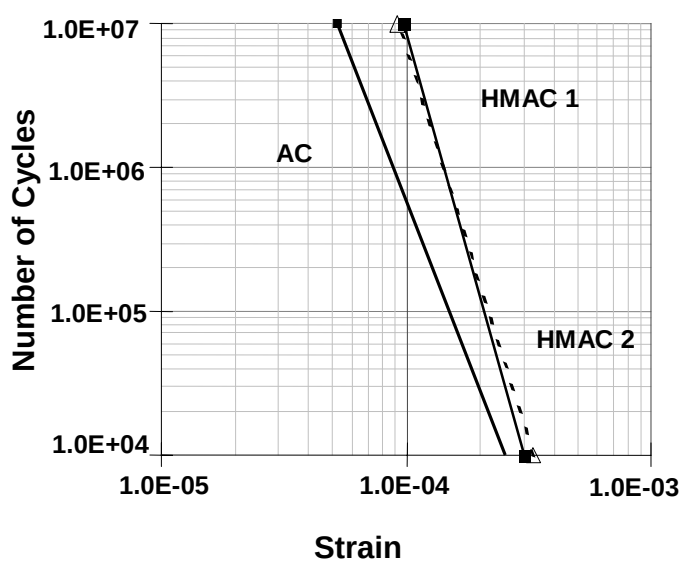

FIGURE 9 Two-point bending fatigue tests (one traditional base AC, two HMACs having the same composition but different hard asphalt).

Illustration of the Possible Economic Gain with the Use of HMAC

The potential saving in the cost of a new pavement when using HMAC as compared to a traditional base concrete solution, can be illustrated by the following examples.

The first one is taken from the 1997 SCETAUROUTE's catalogue. Table 7 presents a comparison for flexible pavements between a HMAC solution as a base layer and a traditional base AC. With the HMAC solution, the reduction in thickness represents 33 percent of the total thickness of the asphaltic layers. If one considers now the asphalt quantities, because of the higher binder content and the lower voids content of the HMAC, the difference between the two solutions is reduced to approximately 24 percent.

The second example presented in Table 8 is related to full-depth asphalt pavements; it is taken from the 1998 French Road Directorate catalogue (*9*). Here the reduction in total thickness of the pavement is 25 percent, which represents about the same reduction in the quantity of aggregate; the reduction in the asphalt quantity is only 4.5 percent. The French Road Administration has preferred to adopt a binder layer to provide protection to the HMAC base course.

Assessment of Performance of HMAC Pavements

In 1997, an assessment of the performance of HMAC pavements built since the beginning of the 1980s was published (*15*). This report covered the use of over 10 million tons of asphalt mix at 47 sites, with pavements from 2 to 14 years old. The conclusions can be summarized as follows:

- For pavements between 2 and 6 years of age, there were no or only minor degradations,
- For pavements between 6 and 10 years old, the percentage of cases presenting cracks grew but the gravity of cracking was low to moderate,
- For the oldest sites, the cracking was similarly moderate and did not require maintenance.

TABLE 7 Comparison Between HMAC and Traditional AC Solution for Flexible Pavements (1997 SCETAUROUTE's *Manual*)

	Flexible pavement Traffic: 600 HV/day, 4% increase/year, 15-year design Subgrade modulus: 120 MPa	
	Traditional AC solution	HMAC solution
	Wearing course: 2.5 cm *BBTM* Binder course: 6 cm *BBL* Base course: 13 cm *GB3* Subbase: 20 cm unbound gravel	Wearing course: 2.5 cm *BBTM* Base course: 12 cm *EME2* Subbase: 20 cm unbound gravel
Difference in thickness of asphalt layers	7 cm (33%)	
Difference in asphalt quantity	–24%	
Difference in aggregate	–33%	

BBTM: béton bitumineux très mince (VTAC)
BBL: béton bitumineux de liaison (AC for binder layers)
GB3: grave-bitume class3 (base AC)
EME2: enrobé à module élevé class 2 (HMAC for base layers)

TABLE 8 Comparison of HMAC and Traditional AC Solution for Full-Depth Asphalt Pavement (1998 Road Directorate's Catalogue)

	Full-depth asphalt pavement Traffic: 20 million ESALs (130 kN) Subgrade modulus: 120 MPa	
	Traditional AC solution	**HMAC solution**
	Wearing course: 2.5 cm *BBTM* Binder course: 6 cm *BBL* Base course: 14 cm *GB2* Subbase: 14 cm *GB2*	Wearing course: 2.5 cm *BBTM* Binder course: 6 cm *BBME* Base course: 9 cm *EME2* Subbase: 10 cm *EME2*
Difference in thickness of asphalt layers	–9 cm (25%)	
Difference in asphalt quantity	–4.5%	
Difference in aggregate	–24%	

BBTM = béton bitumineax trés mince (VTAC)
BBL = béton bitumineax de liaison (AC for binder layers)
GB2 = grave bitume class 2 (traditional AC for base layers)
EME2 = enrobé à module élevé class 2 (HMAC for base layers)
BBME = béton bitumineux à module élevé (HMAC for binder layer)

Transverse cracks were found in two cases only, which shows that thermal cracking is, in the French climatic context, a marginal phenomenon with these hard asphalts used in base course.

Low-Temperature Cracking

It is interesting to report here the extreme case of a trial section with a high-modulus AC made with a very hard asphalt. Initial characteristics were a penetration of 5/10 mm, a R&B softening point of 88°C which led to a Young modulus for the HMAC of 21,600 MPa (direct tension test at 15°C and 0.02 s). Tests on the recovered binder from a core taken from the pavement gave a softening point of 93.5°C, and BBR's temperatures $T_{m=0.300}$ = +1.7°C and $T_{G=300}$ = –5.7°C (T_m is the temperature when the slope m of the creep curve = 0.300; T_G is the temperature when the sheer modulus G = 300 MPa). Cracking was observed after the first winter, with minimum recorded temperatures of –10°C to –13°C.

HMAC for Surface Courses

The search for a solution for thick wearing courses, offering a good resistance to rutting, led to the definition of another kind of high modulus AC, the "*Bétons Bitumineux à Module Elevé,*" BBME, codified since 1993 in French AFNOR standard NF P 98-141. These materials can also be used in binder courses, too.

These mixes generally have a continuous grading of 0/10 or 0/14. The minimum richness factor for the 0/10 mixes is 3.5, which corresponds to an asphalt content of the order of 5.6 percent (by weight of aggregate) for aggregate with a density equal to 2.75.

The performance requirement for resistance to rutting is, in the case of heavy traffic, less than 5 percent rutting after 30,000 cycles for wearing courses and less that 7.5 percent rutting after 30,000 cycles for binder courses when the wearing course thickness is less than 5 cm.

The use of hard asphalt in the wearing course, which is the layer most exposed to

temperature variations and thermal shocks, should be considered cautiously because of the risks of low-temperature cracking or thermal fatigue. In order to limit this risk, a proposal has been made to improve the performance of hard asphalt at low temperature by a modification with polymers. Table 9 presents the characteristics at low temperature of a plain 20/30 PG asphalt and of a hard asphalt modified by reticulation with a styrene-butadiene polymer. This product currently is in an experimental phase.

CONCLUSIONS

Hard asphalts, produced in France for nearly 20 years, have offered for the French climatic context very interesting technical solutions for rutting mitigation of asphalt pavements and for construction of stiff asphaltic base layers. Field performance has indicated no susceptibility to low temperature or thermal fatigue cracking. On the contrary, poor performance had been observed in the past with asphalts produced by air blowing; the search for very high IP values by means of air blowing is detrimental to durability.

The mechanical properties of the hard asphalts are strongly dependent on manufacturing process because this directly influences the composition and the colloidal structure of the asphalts. The rheological tests showed in particular that the results (modulus, phase angle) can vary within a broad interval for asphalts having the same PG at 25°C. This can result in significant differences in behavior at low temperature. Research appears to be still necessary, however, to better assess behavior at failure in order to identify the predominant factors linked to the composition of these asphalts.

To benefit from the qualities of these hard asphalts, it is necessary to have an adequate design for the mix. This cannot just be a simple substitution of the binder. It is worthwhile stressing that, in the concept of HMAC developed in France, these base materials are designed

TABLE 9 Low Temperature Characteristics of Plain 20/30 Asphalt and Styrene-Butadiene–Reticulated Hard Asphalt

	SB Reticulated Hard Asphalt	20/30 Plain Asphalt
Penetration (0.1 mm) at 25°C	27	25
Fraass temperature (°C)	–15	–10
Direct tensile test (5°C, 100 mm/min) Strain at failure (%) Energy at 400 percent strain (J/cm^2) at failure (J/cm^2)	 600 23 37	 fragile
Temperature G´´ (°C) (5 Hz) max	–7	–1
G^* (MPa) (–10°C; 5 Hz)	355	463
Phase angle (°) (–10°C; 5 Hz)	9.6	6.3
Temperature G^* (°C) (7.8 Hz) = 133.3 MPa	5.5	10

with a higher binder content and a lower air voids content. Such provisions are intended to compensate for problems with fatigue, the lower capacity of the hard-grade asphalts for healing as compared to the softer traditional grades.

Hard asphalts have mainly been used in base and binder courses with a surfacing which ensures a certain thermal protection. There is much less experience with HMAC in thick wearing courses, and questions remain with respect to the behavior at low temperature. A solution may be in a complementary modification with polymers or in multigrade-type asphalts.

REFERENCES

1. Sauterey, R. *Evolution de la consommation des bitumes pour enrobage à chaud*. Bulletin de Liaison des Laboratoires des Ponts et Chaussées, No. 68, Nov.–Dec. 1973, p.16.
2. Simoncelli, J.-P., R. Chanut, et al. *Comportement d'une couche bitumineuse épaisse sous trafic lourd, lent et canalisé*. Bulletin de Liaison des Laboratoires des Ponts et Chaussées, No. 47, Sept.–Oct. 1970, pp.45–54.
3. Bitume Actualités. *Dossier bitume*, No. 91, June 1990.
4. *Conception et dimensionnement des structures de chaussées*. Guide technique LCPC/SETRA. Dec. 1994.
5. Glita, S., and J. Conan. *Analyse rhéologique de 7 bitumes durs 10/20*. Congrès Eurasphalt—Eurobitume, E&E 5.110, Strasbourg, Germany, 1996.
6. Groupe National Bitume. *Etude de la fissuration par le haut des bétons bitumineux*. Rapport CR 22, Etudes et Recherches des Laboratoires des Ponts et Chaussées, Oct. 1999.
7. Langumier, G. *Nouvelles possibilités en construction routière; GBTHP un mélange bitumineux optimisé*. Revue générale des routes et aérodromes, No. 612, pp 47–52.
8. *Manuel de conception des chaussées d'autoroutes*. 3rd Ed. SCETAUROUTE Direction Technique, March 1994.
9. *Catalogue des structures types de chaussées neuves de la Direction des routes.* LCPC/SETRA, Sept. 1998.
10. Enrobés hydrocarbonés. Couches d'assises: Enrobés à module élevé. Norme française NF P 98-140, Oct. 1992.
11. Brûlé, B., and J.-F. Corté. *Rapport général. Session V*. Congrès Eurasphalt—Eurobitume, E&E 5.110, Strasbourg, Germany, 1996.
12. Corté, J.-F., Y. Brosseaud, J. -P. Simoncelli, and G. Caroff. Investigation of Rutting of Asphalt Surface Layers: Influence of Binder and Axle Loading Configuration. *Transportation Research Record 1436*, TRB, National Research Council, Washington, D. C., 1994, pp. 28-37.
13. Corté, J.-F., Y. Brosseaud, J.-P. Kerzreho, and A. Spernol. *Study of Rutting on the Wearing Courses of the LCPC Test Track*, Vol. 2. 8th International Conference on Asphalt Pavements, Seattle, Wash., Aug. 10–14, 1997, pp. 1555-1568.
14. De la Roche, C., J.-F. Corté, J. C. Gramsammer, H. Odéon, L. Tiret, and G. Caroff. *Etude de la fatigue des enrobés bitumineux à l'aide du manège de fatigue du LCPC*. Revue générale des routes et aérodromes, No. 716, 1994, pp. 62-74.
15. *Les enrobés à module élevé*. Note d'information du SETRA, No. 96, April 1997.

ADDITIONAL RESOURCES

Corté, J.-F., and J.-P. Michaut. *Le développement d'une innovation : les enrobés à module élevé*. AIPCR. Contribution à la Question IV, XX Congrès Mondial de la Route–Montréal, Sept. 1995.

Marciano, Y. *Comportement à froid de bitumes durs*. Rapport CR 24, Etudes et Recherches des Laboratoires des Ponts et Chaussées, Jan. 2000.

Design and Assessment of Long-Life Flexible Pavements

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Improved strategies for design and condition assessment are required for flexible pavements, which carry the heaviest volumes of traffic, to decrease the need for maintenance and thereby cause less disruption to the road user. The current philosophy and criteria for design are reviewed, and information that has been collected since the last revision of standards in 1984 on the performance of roads is considered. Results demonstrate that the deterioration of thick, well-constructed, fully flexible pavements is not structural and that deterioration generally starts at the surface in the form of cracking and rutting. The evidence suggests that fatigue and structural deformation originating deep within the pavement structure are not the prevalent modes of deterioration. It also shows that changes that occur to the structural properties of the bituminous materials over the life of the road are crucial to the understanding of its behaviour. They imply that a road built above a minimum strength will remain structurally serviceable for a considerable period, provided an appropriate condition assessment strategy is adopted to enable nonstructural deterioration, in the form of cracks and surface deformation, to be detected and remedied before it can have a serious impact on the structural integrity of the road.

The current pavement design method used in the United Kingdom for fully flexible pavements was established by considering the performance of a wide range of experimental pavements that formed part of the trunk road network (*1*). The method developed was based on the interpretation of the structural performance of these roads in terms of theoretical design concepts.

A design life of 40 years was advocated, which was achieved by strengthening the road after about 20 years. Calculation of the costs of flexible roads over 40 years, taking into account variability of pavement performance, cost of traffic delays, and other costs associated with strengthening, showed this to be the optimum design strategy. Since this method was introduced, traffic levels and the consequent disruption at roadworks has continued to increase (Figure 1). Economic considerations indicate that it is now cost-effective to increase the design life of very heavily trafficked routes to at least 40 years, without the requirement for structural strengthening, in order to reduce future maintenance and the associated traffic delay costs.

In addition, more knowledge has become available, over the last 10 years, on the performance of heavily trafficked roads. This has indicated that deterioration, as either cracking or deformation, is far more likely to be found in the surfacing than deeper in the pavement structure; this evidence is in conflict with conventional theory. Also, it was found that the great majority of the thick pavements examined have maintained their strength or become stronger over time, rather than gradually weakening with trafficking as assumed in the current pavement assessment method based on deflection measurements.

FIGURE 1 Disruption to road users on heavily trafficked roads.

This paper reviews design concepts and draws together up-to-date information from full-scale experimental pavements, studies of deterioration mechanisms on the road network, long-term deflection monitoring of motorways, and condition assessment reports prepared to aid the design of structural maintenance. All of this information is required to produce a design method and strategy for condition assessment for roads expected to last at least 40 years without the need for structural maintenance (*2*). These roads are described as long-life roads. The help of the Highways Agency, in allowing information to be used from extensive research programmes that the Transport Research Laboratory (TRL) has undertaken on behalf of the Highways Agency over many years, is gratefully acknowledged.

PAVEMENT PERFORMANCE

Current U.K. pavement design for fully flexible pavements is based on an interpretation of the observed performance of a number of experimental roads, which had carried up to 20 million standard axles (msa), using structural theory. Considerable extrapolation of the observed performance trends was necessary to provide current designs for over 100 msa.

A staged construction is adopted for trunk roads in the United Kingdom. The road is initially designed to reach an investigatory condition after about 20 years, which is considered to be the ideal timing to use. To use the existing strength of the road to good effect in designing a strengthening overlay to extend life for another 20 years. If the road passes beyond that condition, overlay is considered less effective and reconstruction is necessary. The investigatory condition can be related to the transient deflection under a standard wheel load moving at creep speed. This deflection is believed to increase gradually with increasing traffic until the deflection and the level of rutting and cracking indicate the need for strengthening, as first described by Kennedy and Lister (*3*).

Designs for future traffic levels necessarily have to be based on observations of the performance of full-scale pavements under the lower traffic levels experienced in the past. If the requirement is to design future roads for similar traffic flows, this will not present a problem. However, if the traffic growth rate is substantially greater, there may be economic advantage in designing the structural life of the road to last much longer. In this situation, the problem is how current knowledge, based on previous experience, be best used to design pavements expected to carry much higher levels of traffic. In the previous revision of design standards it was necessary to assume that the measured performance trends could be extrapolated to give realistic estimates of future performance. The question that needed to be considered was whether these extrapolations represented the best means of predicting future performance.

Rutting

Rutting is the result of deformation in one or more of the pavement layers. At one extreme it is restricted to the uppermost asphalt layer or layers, termed *surface rutting*. At the other extreme, the main component of deformation arises in the subgrades, and this is termed *structural deformation*. Deformation within the upper asphalt layers does not have a serious effect on the structural integrity of the pavement. On the other hand, excessive structural deformation is a symptom that the load-spreading ability of the asphalt and granular layers is insufficient to protect the subgrade and, if unchecked, will lead eventually to a breakup of the pavement structure. Measurement of the rutting profile at the road surface only is not sufficient to identify the source of the rutting. The consequences for pavement design and maintenance of deformation originating solely at the surface or deep within the pavement structure are substantially different.

A summary of mean rates of rutting of asphalt pavements is shown in Figure 2. The figure indicates a discontinuous relationship between the rate of rutting and pavement thickness, with data forming in two clusters. Pavements with less than about 180 mm of asphalt material deform at a high rate, but thicker pavements deform at a rate about two orders of magnitude less; the sudden transition suggests a threshold effect.

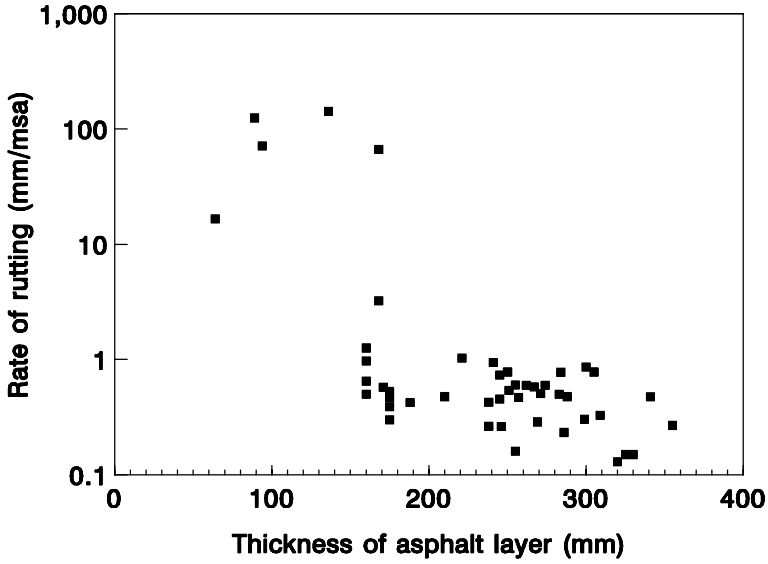

FIGURE 2 Rate of rutting of trunk roads.

Above about 180 mm there is no correlation between the rate of rutting and pavement thickness. The results suggest that for these thicker pavements nearly all the rutting is due to deformation within the upper layers and that the traffic-induced strains in the subgrade are too low to cause structural deformation. It is apparent that below thicknesses of about 180 mm, the much higher traffic-induced subgrade strains have a much greater effect.

Fatigue

The road base is the most important structural layer of the road. All modern analytical design methods for flexible pavements include a criterion, based on laboratory studies, to guard against the possibility of fatigue cracks initiating at the underside of the road base. These methods consider fatigue cracking, caused by repeated traffic loading, to be the major component of structural deterioration. Investigation of the road base fatigue mechanism in full-scale pavements is much more difficult than in the laboratory and it has been noted that, although surface cracking is often observed, there is little evidence of fatigue cracking in the road base of in-service asphalt pavements in the United Kingdom (*4, 5*). Furthermore, it is known that the stiffness of asphalt road base increases with time, and this influences its fatigue resistance. However, this effect has received little attention in the past in relation to road base performance.

The absence of positive evidence of fatigue prompted TRL to initiate an investigation into the residual fatigue life of asphalt road bases from heavily and lightly trafficked areas of the same motorways. The aim was to compare the structural properties of samples of road base measured in the laboratory with the overall condition of the pavements from which they were extracted. This work (*6, 7*) helped to improve basic understanding of the mechanisms of structural deterioration.

Investigation of Roadbase Fatigue in U.K. Motorways

Short sections of four motorways (Table 1), representing a range of age and traffic loading, were selected for detailed investigation. All the pavements examined carried more traffic than they were originally designed to carry.

Cores were cut to enable the structural properties of materials that had been subjected to heavy commercial traffic in the wheelpath of Lane 1 to be compared to the lightly trafficked material of the same age and nominal composition from between the wheelpaths in Lane 3. A comparison of the laboratory-measured residual fatigue life of the lower road base is shown in Table 2.

TABLE 1 Pavements Investigated

Site	Age (years)	Cumulative traffic (msa)	Road Base	Thickness of asphalt (mm)
M4	11	22	HRA	230
M5	19	66	DBM	300
M1	23	71	DBM	350
M62	21	57	DBM	300

TABLE 2 Comparison of Fatigue Life of Road Base

Site	Lane	Number of Tests	Relative Fatigue Life of Road Base
M4	1	80	1.0
	3	76	1.5
M5	1	35	1.0
	3	33	0.7
M1	1	19	1.0
	3	11	1.6
M62	1	28	1.0
	3	31	0.4
Mean	1	162	1.0
	3	151	1.1

If traffic was responsible for weakening the road base, the residual fatigue life of road base material subjected to heavy traffic, in Lane 1, would be significantly lower than that of the lightly trafficked material, between the wheelpaths of Lane 3. Table 2 shows that there was no consistent difference between these measured residual fatigue lives. Most of the difference was accounted for by variations of binder hardness and binder content between the samples extracted from the two lanes. When these factors were taken into account, none of the differences were statistically significant.

All the material tested had a residual fatigue life lower than that of new material. Although traffic loading could not account for this reduction, age will have been an important factor. It is well established that the aging of binders results in an increase in stiffness and a reduction in residual fatigue life of the road base. These changes are unlikely to result in fatigue cracking of the pavement. Calculations using the relationships developed in this study show that the increase in elastic stiffness with age produces a reduction in the traffic-induced, tensile strain responsible for fatigue at the underside of the road base. This reduction more than compensates for any reduction in the laboratory fatigue life of the aged road base. The net effect is that the predicted fatigue life of the road increases with age. This would explain why no positive evidence of fatigue was found.

Structural Assessments

Structural assessments of asphalt roads have failed to detect any evidence of road base fatigue damage. There are no authoritative reports of cracks propagating upwards. In older roads, apart from the absence of road base cracks, the measured elastic stiffness modulus of road base material extracted from the road is usually substantially higher than that expected for new construction. This is indicative that fatigue weakening is not occurring.

In a study carried out by the Road and Hydraulic Engineering Division of the Dutch Ministry of Transport, 176 sections of flexible pavement were examined in order to verify their pavement design method (*8*). This study revealed that in pavements with an asphalt pavement thickness greater than 160 mm, cracks initiated at the surface and generally did not penetrate the full depth of the asphalt. In thinner pavements, where the asphalt was fully cracked, a structural analysis indicated that the cracks also initiated at the surface and propagated downwards. The overall

conclusion of this work was that, *conventional fatigue will rarely or never be the predominant failure mechanism, but surface cracking will be the main cause of structural distress.*

Surface Cracking

Cracking observed at the surface of thick, mature, flexible pavements is relatively common. The most usual form is longitudinal cracks in the wheel tracks. This has often been regarded as evidence of conventional fatigue in which cracks have initiated at the bottom of the road base and then propagated to the surface. However, where this cracking has been investigated by cutting cores, it has invariably been found that the propagation is downwards rather than upwards.

Longitudinal surface cracks have been observed at 10 sites investigated by TRL. At these sites, provided the crack had not propagated into the road base, there was no observable or measurable damage to the road base directly beneath the cracks. An example of longitudinal cracking in the M1 site is shown in Figure 3.

Surface cracking is not always longitudinal. At other sites transverse cracks occurred in all lanes of each carriageway, and they were not confined to wheel tracks. As with longitudinal cracking, transverse cracks generally penetrated only up to 100 mm into the surfacing.

FIGURE 3a Longitudinal cracking in the near-side wheelpath of the M1 site.

FIGURE 3b Longitudinal cracking in the near-side wheelpath of the M1 site.

The mechanism of surface cracking is complex, and there is no satisfactory explanation of this phenomenon. Calculation of the traffic-induced stresses at the pavement surface is complicated because the vertical contact stresses are nonuniform and radial horizontal forces are present. Consequently, significant horizontal tensile stresses can be generated at the surface of the pavement.

Thermally generated stresses will also contribute toward the initiation and propagation of surface cracks. This is especially so for transverse cracking in which thermal stresses are likely to be the principal cause of the tensile condition required for crack initiation. Age hardening of the binder in the wearing course, especially the top few millimeters, will also play a part, with hardening over time progressively reducing the ability of the wearing course to withstand the thermal and traffic-generated stresses at the surface.

Curing of Asphalt

It has long been known that the bitumen in pavement layers stiffens with time. Whereas a gradual hardening of the main structural layers appears to be beneficial, and is more accurately described as *curing*, excessive ageing of the wearing course can lead to cracks initiating at the surface.

During the mixing and laying process, the penetration of the bitumen in standard dense bitumen macadam (DBM) typically drops from an initial nominal value of 100 to about 70. In subsequent service, further reduction takes place resulting in values often as low as 20 after 20 years. Chaddock and Pledge (*9*) demonstrated, in test pavements, that the curing behaviour was variable and that the stiffness of DBM road base could change by over 100 percent during the first year in service.

In addition to the test pavements, data on the curing of road base material from a large number of in-service roads has been collected by TRL. These data include the measured properties of the recovered binder and stiffnesses of materials from pavements of different ages. Figure 4 shows the variation of penetration of the recovered binder, with time, for DBM road base manufactured with a

nominal 100 penetration grade binder.

Figure 4 clearly illustrates that the penetration has reduced from about 70, shortly after laying, to a value in the range of 20 to 50 after 15 years. These binder changes will result in a progressive increase in the elastic stiffness modulus, which is a measure of load-spreading ability. This increase in stiffness of asphalt materials has major implications for pavement design.

Long-Term Pavement Strength

The fact that the elastic stiffness, and hence the load-spreading ability of asphalt road base, in thick, well-constructed roads increases steadily over time is an indication that traffic-associated deterioration of the road base does not occur. This improvement in load-spreading ability should manifest as a reduction in deflection over the life of the road.

The measurement of pavement deflections under a slow-moving, standard wheel load is the normal method of routine pavement structural assessment in the United Kingdom. The deflections are expected to increase with the passage of traffic, reflecting a weakening of the structure. Increased deflections imply increases of the traffic-induced strains in the road base and subgrade, which are considered to control pavement deterioration. The deflection histories of 10 heavily trafficked sections of motorway were examined to investigate whether the strength of thick, fully flexible pavements reduces with time and traffic. Pavement deflection is routinely measured on the trunk road network using deflectographs similar to that shown in Figure 5 (*10*). Deflection trends, based on measurements with deflectographs of four motorways that had carried up to 48 msa, are shown in Figure 6.

The deflections of these sites show considerable fluctuations that may be partly due to the difficulty of applying accurate temperature corrections, seasonal variations in the subgrade strength, and variation in alignment of successive surveys. Further confirmation of these deflection trends has been provided by falling weight deflectometer (FWD) measurements on the same sites (Figure 7). Unlike the deflectograph surveys these deflection measurements have been carried out on

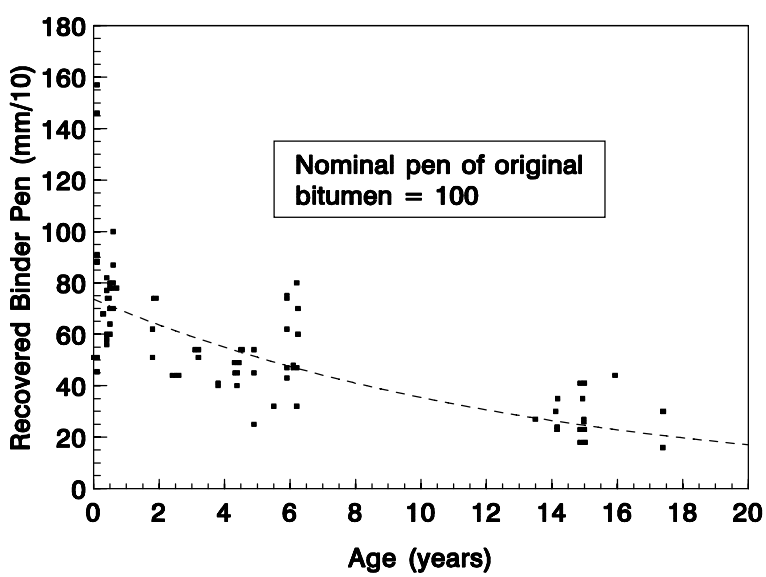

FIGURE 4 Change in PG of binder with time.

FIGURE 5 Deflectograph survey in progress.

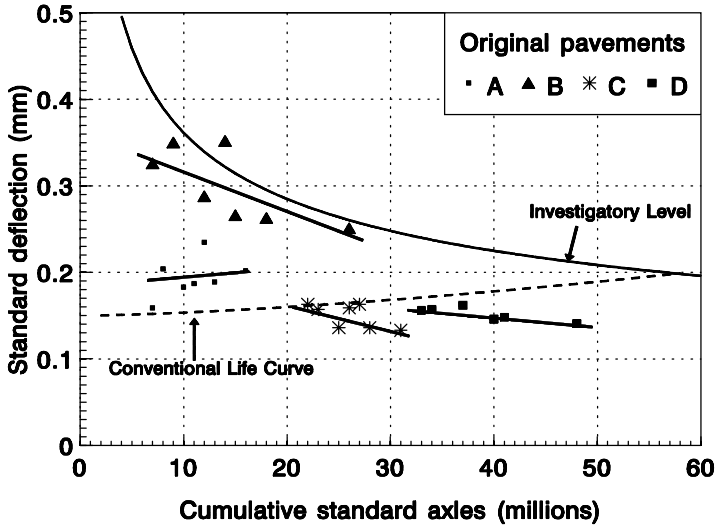

FIGURE 6 Deflection histories of in-service motorways—deflectograph measurements.

exactly the same points each year. Examples of trends from two of the sites over a 13-year period are given in Figure 8.

With one exception, which shows no decisive trend either way, all the sections show a trend of steady or decreasing deflection with age and traffic. These decreases imply that the overall stiffnesses of the pavements are increasing over time and that any traffic-related deterioration is more than offset by curing of the road base or strengthening of the foundation. For whatever reason, the road is becoming stiffer with time, and, hence, the traffic-induced stresses and strains in the road base and the subgrade, which are considered to be responsible for structural deterioration, are decreasing.

FIGURE 7 FWD carrying out measurements.

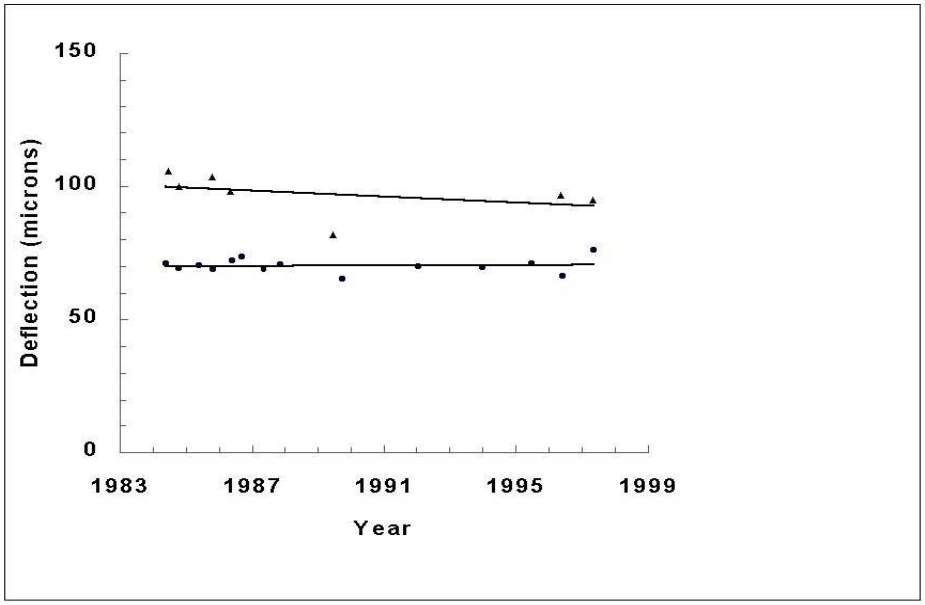

FIGURE 8 Deflection histories of in-service motorways—FWD measurements.

CONSIDERATIONS FOR LONG-LIFE PAVEMENT DESIGN

The conclusion from the review of information on the performance of flexible roads in the United Kingdom is that, above a threshold strength, the road will remain structurally serviceable for a considerable period provided that nonstructural deterioration, in the form of surface-initiated cracks and deformation, is detected and remedied before it can have a serious impact on the integrity of the road. To achieve a long life it is also necessary for the road to be well constructed with good quality asphalt and a good foundation so that deterioration does not result from construction or material inadequacies.

Threshold Strength, Curing, and Traffic Flow

Curing implies that the road is most vulnerable to damage from traffic when it is first laid, before its structural properties have had time to improve. Provided that the road is built strong enough initially, so that its main structural layers are not weakened by traffic loading, curing will progressively improve the load-spreading ability of these layers and make the road progressively less vulnerable to traffic-induced structural damage. This type of behavior implies that there is a minimum threshold strength above which the pavement should have a very long but indeterminate structural life. It is therefore reasonable to assume that the initial strength of the road should be a major factor in determining its future life.

Roads constructed to meet the demands of present-day traffic levels, which may be 10 or 20 times higher than those encountered on many of the older in-service roads, will need to be initially stronger to avoid excessive deterioration in their early life. A conservative calculation indicates that a road constructed with a thickness of more than 260 mm of asphalt would have a long but indeterminate life for traffic of up to 5 msa per year and that 270 mm would be sufficient for any traffic loading. This higher thickness would ensure long life, even if curing did not take place, provided that the effective thickness of the asphalt layer was not reduced by deterioration due to cracking.

Surface Cracking

Surface cracking, in a road that is built just above the minimum strength required for long life, may weaken the road and accelerate the deterioration. To prevent this from occurring, it will be necessary to adopt conservative designs to enable the road to withstand some surface cracks. Timely remedial action should be taken before these cracks can have a severe structural impact on the road; however, cracks may propagate up to 100 mm into the road before this action is taken. The most conservative assumption is to assume that the material down to the depth of the crack penetration does not contribute to load spreading. This would imply that a road constructed with a total of 370 mm of asphalt material would be able to tolerate a surface-initiated crack propagating 100 mm into the road even if the effect of curing was very small.

Provision for Future Changes in Vehicle Characteristics

The current legal maximum axle load is 10.5 tonnes, and this was increased to 11.5 tonnes in 1999. An increase in the thickness of the bound layer of 20 mm would be more than sufficient to provide extra load-spreading ability to compensate for this increase in legal maximum axle load.

Risk of Premature Failure

The addition of the conservative estimates to allow for the factors discussed above suggests that a pavement consisting of 390 mm of asphalt material is sufficient for a long-life road. This summation introduces further conservatism since, for example, surface cracks, if they occur at all, will normally appear several years after the road is laid. By this time the road base will have cured, and the remaining thickness will be well above the threshold strength for a long-life road. A pavement of this thickness will be able to tolerate opening traffic well in excess of 5 msa per year even if the asphalt in the main structural layers does not cure.

A practical way forward is to use the existing design curves for traffic levels up to the level of the threshold strength and then regard the design as long life, with no additional thickness required to provide longer life. Figure 9 illustrates possible design curves using the three standard road base materials that are characterised by three different levels of elastic stiffness or load-spreading ability.

These designs will have built-in conservatism, but some conservatism can be justified, considering the economic importance of these roads. A thickness of less than 200 mm for the asphalt paving is not recommended even for lightly trafficked roads that are required to endure for 40 years. Thin roads may be at risk of structural deformation and rapid propagation of surface-initiated cracks through the full thickness of asphalt.

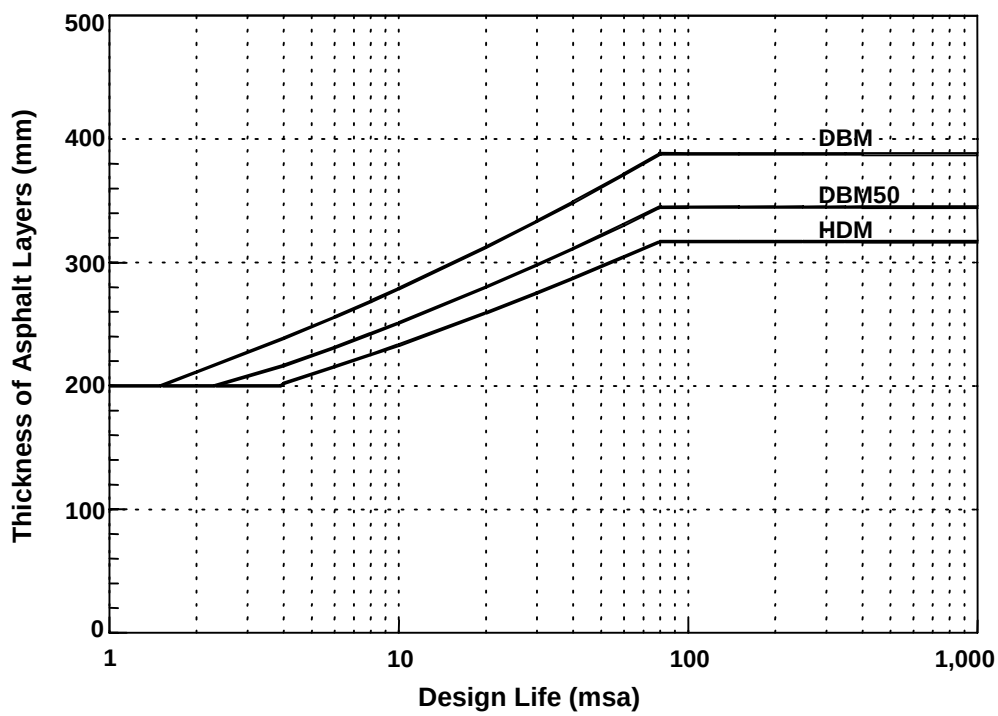

FIGURE 9 Design curve to include long-life pavements.

Pavement Condition Assessment

Premature structural failure may be brought about by poor construction practice or by failure to remedy surface distress. To ensure a long life, procedures need to be developed in which regular inspections of pavement condition are carried out and timely action taken to remedy any surface deterioration detected. This may therefore require a redefining of the methods of monitoring conditions at the network level and redefining levels for detailed investigation and preventative maintenance.

Network Condition Monitoring

The present approach to monitoring the condition of the network is dominated by estimates of structural residual life based on deflection measurements (*11*). The research work reported in this paper, together with other related work carried out for the Highways Agency in the condition assessment field, has raised questions about the validity of the existing deflection design method for some pavements. Apparently, thick flexible pavements do not deteriorate as expected at least in deflection terms. Rather than a slow initial increase of deflection with time and traffic, followed by an unpredictable and rapid increase to failure, many examples of such pavements have been observed to either not deteriorate or even to improve in deflection terms. Thus the present deflection design method, as embodied in the PANDEF software program, will predict ever shorter residual lives as traffic passes over a pavement, even if the deflection level shows no sign of increasing. The deflections measured on such pavements need a different type of interpretation from that currently provided.

On the basis of the observed behaviour of such pavements, criteria to identify long-life in-service pavements on the network can be identified from deflection measurements in conjunction with their layer thicknesses as shown in Figure 10. The main long-life zone may be delineated by

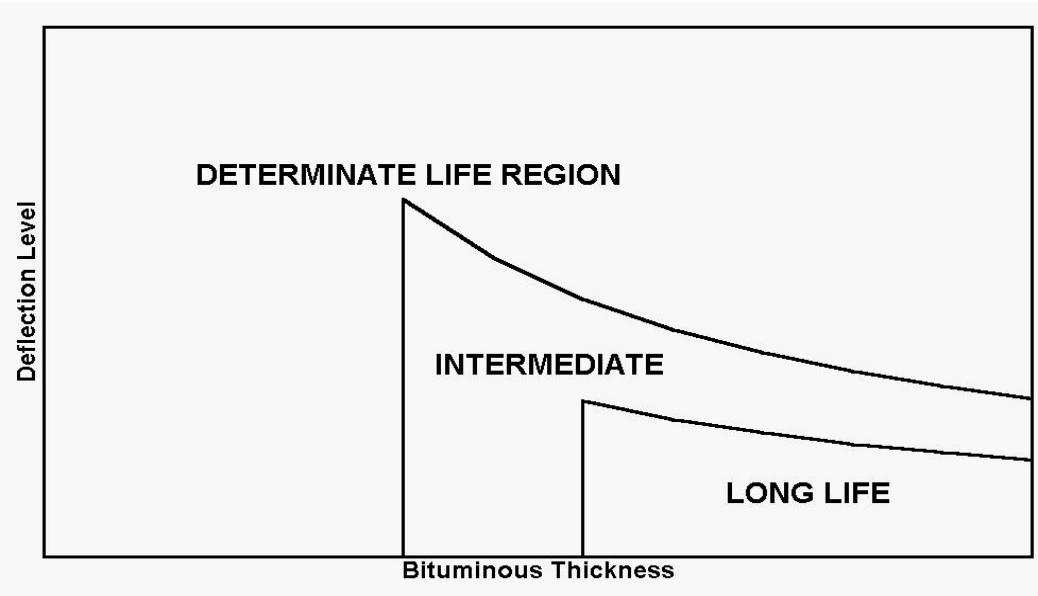

FIGURE 10 Provisional chart for preliminary identification of in-service long-life pavements.

fairly cautious criteria developed in conjunction with those creating long-life designs for new roads. The intermediate zone is bounded by a minimum thickness, the level below which rut rates increased significantly, and by a deflection/thickness curve.

Applying tentative criteria to a sample of the network suggests that around 80 percent of the fully flexible parts of the motorway network are long life. The proportion of the all-purpose trunk network that is long life is, as might be expected, rather lower. For fully flexible roads it is expected to be around 20 percent (mainly because of inadequate bituminous cover rather than high deflection levels). If these two figures are combined, this suggests that around 30 percent of the flexible pavements on the motorway and all purpose trunk road network may consist of long-life pavements.

It should be remembered, however, that long-life pavements do not mean infinite-life or non-maintenance pavements. Replacement of the surfacing layers will still be needed, sometimes to maintain skidding resistance or texture level and sometimes to replace cracked or rutted surface layers. Therefore, the recognition of the existence of long-life pavements will increase the proportion of the network in need of surface treatment, and a means of summarising its condition similar to that for structural condition will be required. This could be relatively easily generated on a simple level from traffic speed surveys, in particular, with the development of new techniques such as those embodied in the HARRIS vehicle shown in Figure 11 (*12*). Thus, surface condition parameters indicating potential structural deterioration such as rutting and cracking could be easily assessed, together with those parameters more related to the functional properties required of the pavement, such as longitudinal evenness and texture depth.

FIGURE 11 HARRIS survey system.

Scheme Selection

Current assessment procedures for assessing the structural needs of road pavements and scheme selection are set out in Volume 7 of the English Highways Agency's *Design Manual for Roads and Bridges* (*13*). The main features are illustrated in the flowchart in Figure 12. Although condition data are available from the High Speed Road Monitor (HRM) and CHART visual surveys, it is the results from deflectograph surveys that most strongly influence maintenance decisions.

The studies developing the concepts for long-life pavements have shown that, in the vast majority of cases on the trunk road network, the deterioration mechanisms encountered concern defects originating and propagating from the surface of the pavement downwards. This suggests that a more appropriate and reliable pavement assessment regime would be achieved by always first considering surface condition before structural condition. A new approach could integrate existing

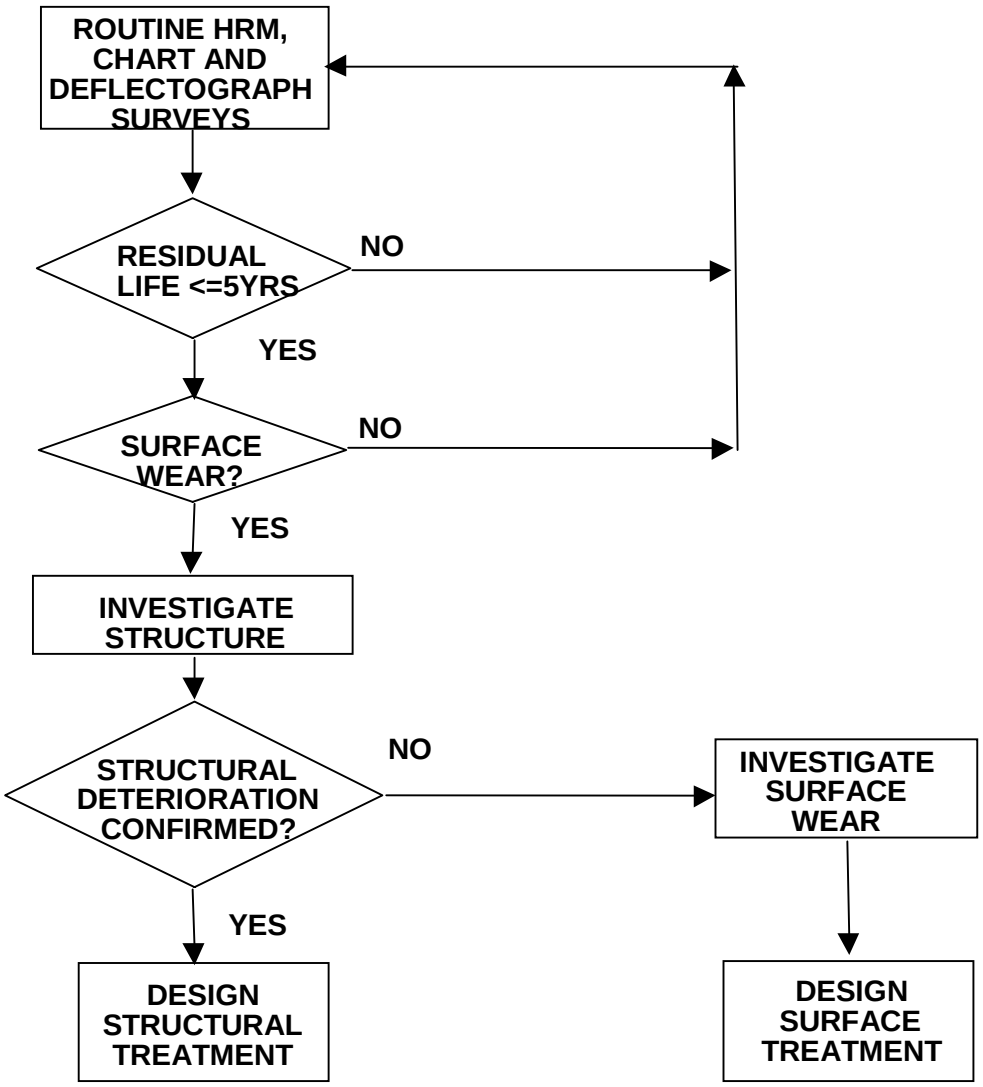

FIGURE 12 Current structural assessment procedure (HRM = high-speed road monitor).

procedures for assessing surface and structural maintenance and assume that no maintenance of any kind is required unless there is evidence of wear on the road surface. Much more reliance would be placed on the results of surface condition surveys that will, in future, mostly be carried out at traffic speed with little traffic disruption. However, routine deflection surveys would need to continue to support network structural assessments and maintenance decisions for all types of pavements, particularly the thinner ones.

Project Level Investigations

The initial aim of project level investigations will be to determine how far the surface deterioration extends into the pavement and, in particular, whether it is only in the surfacing. Details of the exact location and extent of rutting and pattern of cracking will be available from routine survey data. Cores taken on the cracks or at crack ends will determine the depth, direction and propagation of cracks. These cores should penetrate at least halfway into the base on fully flexible pavements to ensure that the full depth of cracking can be recorded. They will also provide evidence of which layers are affected by rutting and any loss of integrity of the materials, such as stripping of the binder. In future, on the basis of earlier TRL research, it may be possible to use ground penetrating radar systems to monitor such penetration nondestructively (*14*). If the cracking is found to penetrate into the structural layers of the pavement, a conventional thorough structural investigation will be necessary.

Treatment Selection

Acknowledgement of the existence of long-life pavements will, of course, affect the selection of suitable maintenance treatments. As was stated earlier, long-life pavements will not be of infinite life. However, as long as the pavement has been well constructed and the foundations remain sound, no structural treatment such as overlays or reconstruction should be necessary. The surface will deteriorate as for determinate life pavements. Accumulated deformation will cause rutting, which will need to be treated by replacement or by thin overlay before it becomes a safety hazard. Cracking may initiate at the surface, which will need treating by replacement before it penetrates into the structural components of the pavement. Skid resistance will deteriorate and initiate the necessary remedial surface treatment. If structural deterioration has occurred, on a determinate-life pavement, strengthening may need to be provided by overlays or reconstruction. However, it should be noted that it may be that only thin overlays are necessary to convert a determinate-life pavement into a long-life equivalent.

CONCLUDING REMARKS

This proposed design method suggests that, in future, the wearing course on fully flexible pavements is likely to be replaced at intervals, whereas the underlying layers will be regarded as permanent. Thus, maintenance costs would be generally limited to the wearing course to maintain safety and comfort for the road user.

Pavement deterioration caused by cracks propagating downwards from the surface has received relatively little attention from researchers. However, a better understanding of deterioration mechanisms in general will result in improved materials and construction practices that prevent or delay the onset of problems. The ultimate goal of pavement design is to develop a mechanistic

method based on a fundamental understanding of the behaviour of road materials. Although this goal may be difficult to achieve, it should be pursued. A fully developed and validated analytical design method would aid economic planning and provide insight into the consequences of future changes in materials or vehicle characteristics.

The relationships developed between design inputs and design thicknesses are based on historical evidence. Therefore, there is a need to continually monitor factors that may affect pavement performance. For the future there are many changes expected, such as heavier lorries, air suspensions, increasing proportions of super-single tyres, and the development of innovative materials and construction practices. All of these will need careful monitoring so that designs can be adjusted as necessary to ensure that road pavements provide good value for money.

The overall conclusion is that well-constructed pavements built above a minimum strength are not likely to exhibit structural damage when subjected to very high levels of commercial traffic for a very long time, provided that deterioration originating in the asphalt surfacing as either rutting or surface initiated cracking is detected and remedied before it has a serious impact on the structural integrity of the road.

Existence of long-life pavements has increased the importance of reliable methods of assessment of surface condition, in particular the identification of surface cracks and their depth of propagation. The recent development of new traffic-speed equipment to identify surface cracks automatically will help with this, but further techniques still need to be developed.

The maintenance of in-service long-life pavements should be relatively simple and low cost if close monitoring of surface condition is employed and any loss of surface integrity rapidly repaired.

ACKNOWLEDGMENTS

The research into the design of long-life pavements described in this paper was funded jointly by the Highways Agency, Quarry Products Association, and the Refined Bitumen Association. Research into the assessment of long-life pavements is funded by the Highways Agency. The paper is published with the permission of these organizations.

REFERENCES

1. Powell, W. D., J. F. Potter, H. Mayhew, and M. E. Nunn. *Design of Bituminous Roads.* TRL Report LR 1132, Transport Research Laboratory, Crowthorne, U.K., 1984.
2. Nunn, M. E., A. Brown, D. Weston, and J. C. Nicholls. *Design of Long-Life Flexible Pavements for Heavy Traffic.* TRL Report 250, Transport Research Laboratory, Crowthorne, U.K., 1997.
3. Kennedy, C. K., and N. W. Lister. *Prediction of Pavement Performance and the Design of Overlays.* Report LR 833, Transport and Road Research Laboratory, Crowthorne, U.K., 1978.
4. Goddard, R. T. N., and W. D. Powell. Assessing the Condition of Bituminous Roads. *Journal of the Institution of Highways and Transportation*, Vol. 34, No. 5, London, U.K., 1987, pp. 26–32.
5. Thrower, E. N. *A Parametric Study of a Fatigue Prediction Model for Bituminous Road Pavements.* TRL Report LR 892, Transport Research Laboratory, Crowthorne, U.K., 1979.
6. Wu, F. *Assessment of Residual Life of Bituminous Layers for the Design of Pavement Strengthening.* Ph.D. thesis. University of Glamorgan, Wales, U.K., 1992.
7. Nunn, M. *A Question of Fatigue?* Symposium on Performance and Durability of Bituminous Materials, University of Leeds, U.K., 1994.

8. Schmorak, N., and A. Van Dommelen. *Analysis of the Structural Behaviour of Asphalt Concrete Pavements in SHRP-NL Test Sections.* International Conference on SHRP and Traffic Safety, Prague, Czechoslovakia, 1995.
9. Chaddock, B., and K. Pledge. *Accelerated and Field Curing of Asphalt Road base.* TRL Project Report PR87, Transport Research Laboratory, Crowthorne, U. K., 1994.
10. Ferne, B. W., and P. K. Roberts. *Deflectograph—The Way Forward for Structural Assessment*. Institution of Highways and Transportation, London, Oct. 1991, pp. 5–6.
11. Jordan, P. G., B. W. Ferne, and D. R. Cooper. *An Integrated System for the Evaluation of Road Pavements*. 6th International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Mich., 1987.
12. Ferne, B. W., L. G. Hawker, and J. Pynn. *A Practical System for Crack Image Collection and Automatic Identification*. 5th International Conference on the Bearing Capacity of Roads and Airfields, Trondheim, Norway, 1998.
13. *Design Manual for Roads and Bridges.* English Highways Agency, Norwich, U.K., 1974.
14. Hawker, L. G., and B. W. Ferne. Routine Assessment of the English Trunk Road Network. *Proc., Alan Brant Memorial Workshop*, Leamington Spa, U.K., April 1992, Institution of Highways and Transportation, London, 1992.

Long-Life Rehabilitation Design and Construction
I-710 Freeway, Long Beach, California

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This paper briefly describes the methodologies used for the mix and structural pavement section designs for the rehabilitation of the Interstate 710 (I-710) freeway adjacent to the Port of Long Beach, California, which include asphalt concrete (AC) replacement structures in the vicinity of three overcrossings and AC overlays on the broken and seated existing portland cement concrete pavement sections. Both structural pavement sections have been designed to accommodate estimated traffic of 200 million equivalent single-axle loads.

Critical to the successful performance of these pavement structures in this heavily trafficked section of the I-710 are the construction quality control and quality assurance and mix design requirements. Important aspects of the construction requirements are also discussed, which include strict controls on the aggregate and binder contents as well as the compaction of the mixes utilized in the pavement sections.

Staging of construction including considerations relating to construction management to meet the 55-h weekend closures planned for the freeway are also described. Finally, the paper emphasizes the importance of the partnered effort between the California Department of Transportation, the Asphalt Industry in California, and the University of California at Berkeley, in developing the requisite design and construction requirements for this project.

Interstate 710 (I-710) is located in Southern California, in Los Angeles County. Rehabilitation is scheduled for Spring 2001 and the project has been selected for a long-life pavement design, with a design life of 30 to 40 years. The freeway is a heavily trafficked route and carries traffic in and out of the Port of Long Beach. The specific section of Interstate Route 710 selected for this project is between the Pacific Coast Highway and the I-405 Freeway (Figure 1).

The existing pavement structural section consists of 200 mm (8 in.) of portland cement concrete (PCC), 100 mm (4 in.) of cement treated subbase, 100 mm (4 in.) of aggregate base and 200 mm (8 in.) of imported subbase material. Two rehabilitation strategies have been selected, one for the majority of the section and the other for under the structures. On the sections where the overhead clearance is acceptable, the existing PCC will be cracked and seated and overlaid

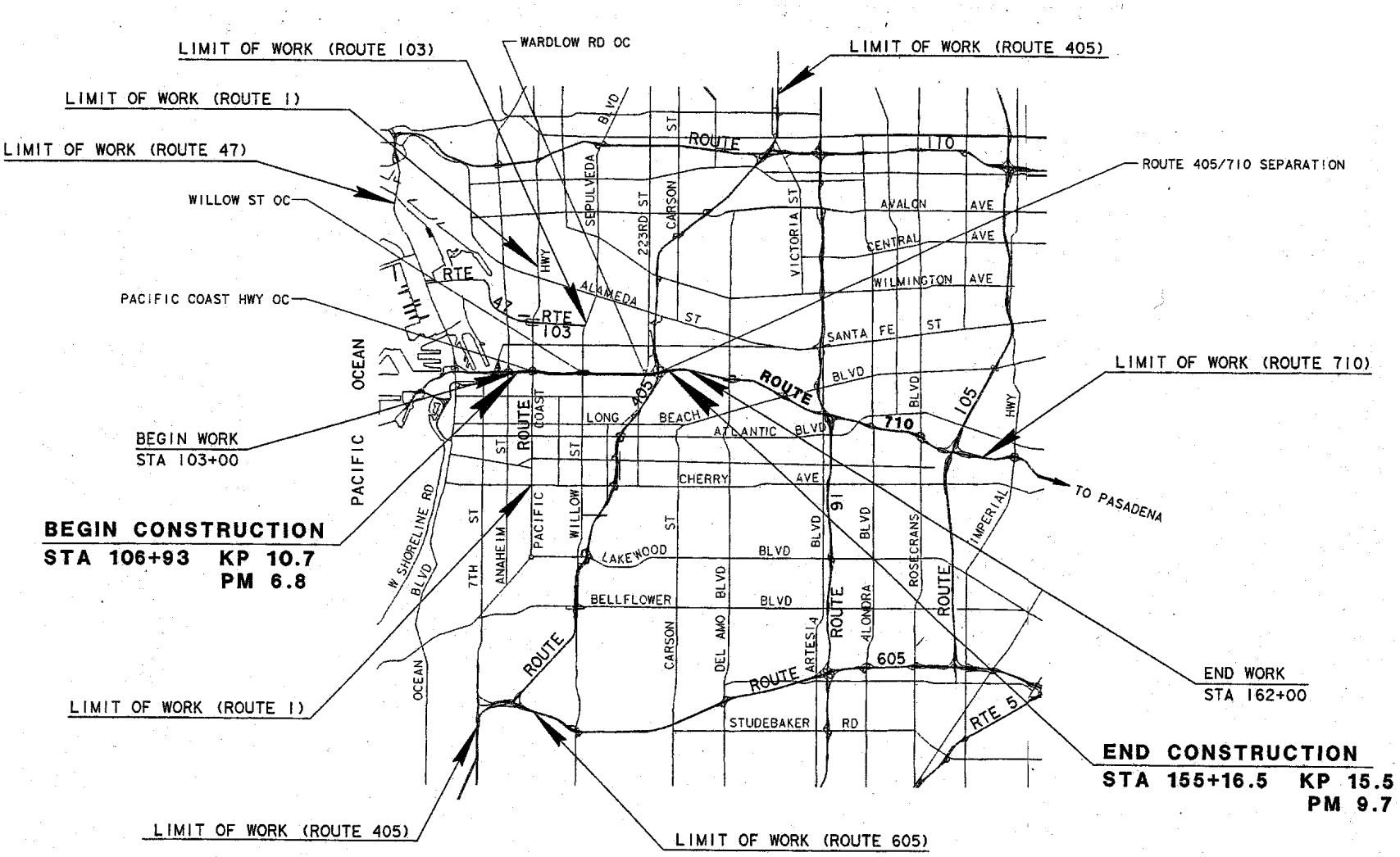


FIGURE 1 Portion of I-710 scheduled for rehabilitation (OC = overcrossing, STA = station, KP = kilometer post, PM = post mile).

with asphalt concrete (AC). Under the structures where minimum clearance requirements do not allow an overlay, full depth AC sections will be utilized and the freeway grade will be reconstructed and lowered to improve the clearance.

This paper briefly summarizes the results of an investigation to design both a suitable AC mix and a full-depth AC structural section containing the mix, or mixes for this segment of the I-710. In addition, included is a brief summary of some of the specification requirements for the mixes to be used for both sections as well as mix compaction requirements and considerations of constructability of the sections within time constraints associated with weekend freeway closure for rehabilitation and reconstruction.

MIX AND STRUCTURAL PAVEMENT DESIGNS

Strategic Highway Research Program (SHRP) mix evaluation technology, enhanced by research resulting from the Caltrans Accelerated Pavement Testing (CAL/APT) Program, was used in preparing both the mix and pavement designs. Binder contents for mixes containing a PBA-6a* polymer modified binder with additional elastomeric components (AASHTO MP-1 designation, PG64-40) and an AR-8000 asphalt cement (AASHTO MP-1 designation, PG64-16) were selected based on the results of the SHRP-developed repeated load simple shear test at constant height (RSST-CH) (*1*). Structural section designs were prepared using mix fatigue test data obtained with the SHRP-developed flexural fatigue test (*2*).

Mix Designs

Laboratory tests were performed on mixes with materials that will be used in construction. The aggregate, an all-crushed San Gabriel material, and the binders were supplied by members of the California asphalt industry, i.e., Vulcan/CalMat and Huntway Refining. The aggregate requirements for the project were changed to 100 percent crushed with the coarse aggregate containing two crushed faces. This is in contrast to current standards for the coarse aggregate which are 90 percent crushed and one crushed face (*3*).

Results of the RSST-CH obtained at 50°C (122°F) were used to select the binder contents for the two mixes. Essentially, the mix design consisted of selecting the highest binder content which permits the mix to accommodate the design traffic at the critical temperature [in this case 50°C (122°F)] without exceeding a limiting rut depth of 12.5 mm (0.5 in.) (*4*). Results of the RSST-CH tests are shown in Figure 2 together with associated traffic repetitions at 50°C (122°F) converted to equivalent laboratory repetitions. Because of its greater resistance to permanent deformation, the mix containing the PBA-6a* binder is planned for use as the surface course. A design binder content of 4.7 percent (by weight of aggregate) was selected for the estimated laboratory equivalent of 660,000 repetitions. Because of structural considerations, the AR-8000 mix was selected for the rest of the pavement section. Since it would likely be subjected to trafficking prior to placement of the PBA-6a* mix, a binder content of 4.7 percent was also selected to preclude premature rutting. The 146,000 repetitions shown in Figure 2 were the maximum equivalent laboratory repetitions that the AR-8000 mix would be expected to sustain prior to placement of the PBA-6a* mix.

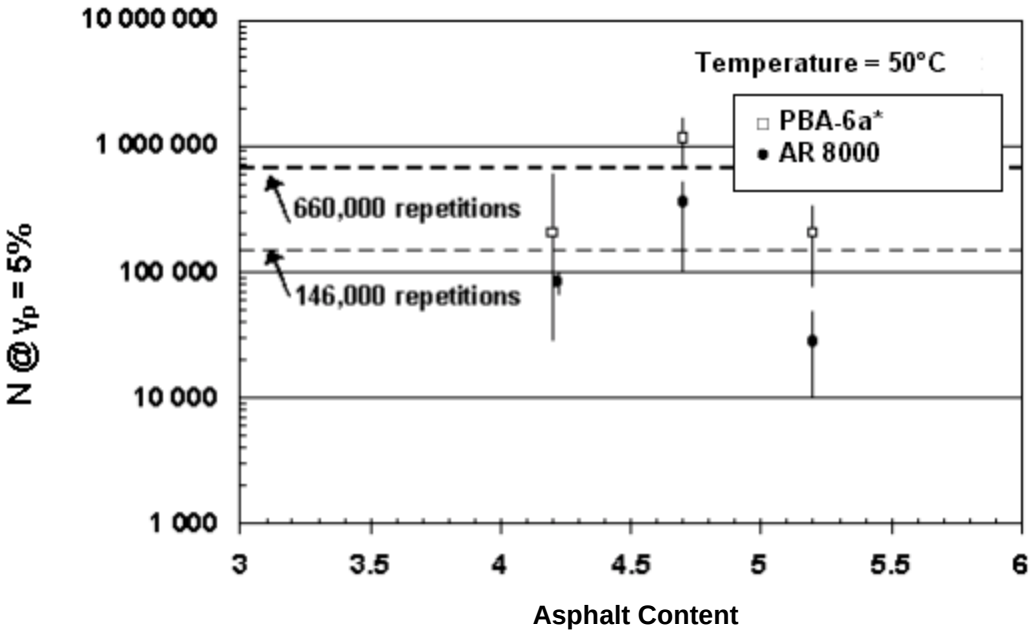


FIGURE 2 RSST-CH repetitions to 5 percent permanent shear strain versus binder content (percent by weight of aggregate).

Structural Section Designs

Underneath the overcrossing, it is not possible to place an overlay and maintain the vertical clearance; therefore, a full depth asphalt concrete section was designed to replace the existing PCC structural section. This design will be used at three locations, 305 m (1000 ft) in length at each location. Over the remainder of the project, an asphalt concrete overlay will be placed on broken and seated PCC.

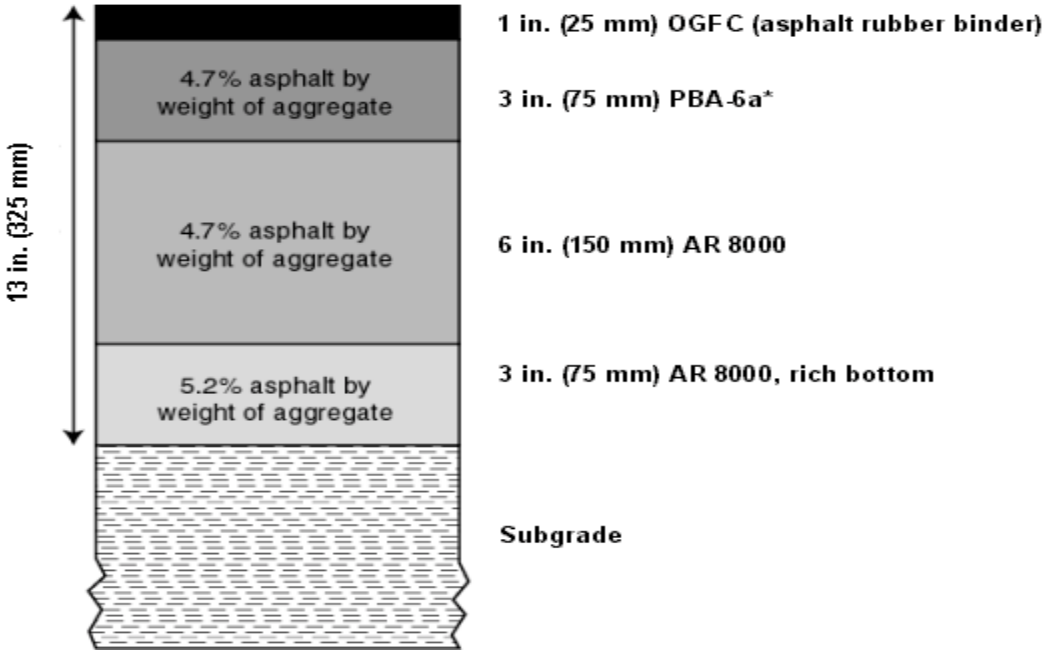
The full depth AC was designed using multilayer elastic analysis. The procedure requires determination of the principal tensile strain on the underside of the AC pavement to mitigate bottom-up fatigue cracking; and determination of the vertical compressive strain at the subgrade surface to minimize the contribution of the layers below the AC to surface rutting. Fatigue resistance and stiffness of the mixes were determined using the SHRP-developed flexural fatigue test, which permits determination of a relationship between the applied tensile strain and the load repetitions to cracking.

The full-depth structural section includes the use of a rich-bottom design for the lower portion of the full-depth AC (*5*); the binder content is 0.5 percent higher than the design binder content. Increasing the binder content facilitates greater compaction, which improves the fatigue resistance of the mix. Rutting resistance of the pavement is not compromised because this layer is at the bottom of the AC.

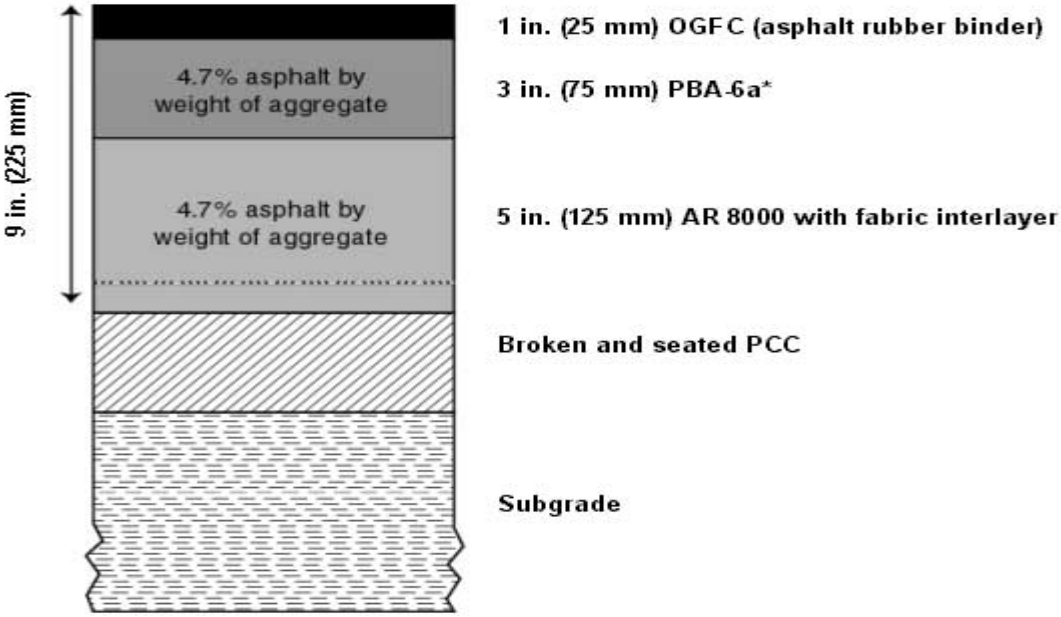
The recommended structural section resulting from the analyses is shown in Figure 3*a*. The section consists of mixes containing the AR-8000 mix (because of its higher stiffness) and the PBA-6a* mix (because of its higher rut resistance). Use of both mixes gave the thinnest pavement section while ensuring the required fatigue and rutting performance. In addition, an open-graded friction course with an asphalt-rubber binder will be used as the wearing course. Its purpose is to reduce tire splash and spray, hydroplaning potential, and tire noise, and to serve as

a layer that reduces the potential for aging in the PBA-6a* mix and that can be replaced periodically.

For the overlay section, a finite element analysis was performed to select the total thickness above the cracked and seated existing PCC pavement. The resulting section is shown in Figure 3*b*. As seen in this schematic, the same materials as used in the full-depth replacement sections



(a)



(b)

FIGURE 3 Proposed design for Interstate 710 rehabilitation: (a) replacement section and (b) overlay section.

are incorporated. Also, the California Department of Transportation (Caltrans) practice of using an asphalt-saturated fabric interlayer has been recommended to reduce the potential for reflection cracking.

Heavy Vehicle Simulator Test

To evaluate the mix design prior to rehabilitation construction, an overlay was constructed on an existing jointed PCC pavement at the Richmond Field Station (RFS) of University of California, Berkeley (UC Berkeley). The overlay consisted of 75 mm (3 in.) of the mix with the PBA-6a* binder over 75 mm (3 in.) of the AR-8000 mix, both at 4.7 percent binder content.

The California asphalt industry shipped aggregate representative of the type likely to be used on the project from Southern California. Industry representatives also supplied the binders (Huntway Refinery), prepared the mixes at a central batch plant (operated by Dumbarton Quarries), and placed the mixes at the RFS site (O. C. Jones Engineering and Construction). Both layers were compacted to about 6 percent air-void content.

Following construction, the heavy vehicle simulator (HVS) shown in Figure 4 was used to load the PBA-6a* mix with about 10,000 repetitions per day of a 40 kN (9,000 lb) load on dual tires with a cold inflation tire pressure of 690 kPa (100 psi). The temperature of the pavement was maintained at the critical temperature, 50°C (122°F), at a 50 mm (2 in.) depth. [A description of HVS and its use in pavement evaluation is included in *Caltrans Accelerated Pavement Test Program* (*6*).]

Results of the accelerated loading on the PBA-6a* mix carried to about 170,000 channelized repetitions are shown in Figure 5. Also shown in the figure are results obtained from an earlier study using both a dense-graded AC with AR-4000 asphalt cement (Stabilometer "*S*" value = 43) and an asphalt rubber gap-graded hot mix (Stabilometer "*S*" value = 23). It will be noted that the PBA-6a* mix performed significantly better in terms of rutting than the other two mixes (*6*).

FIGURE 4 Heavy vehicle simulator.

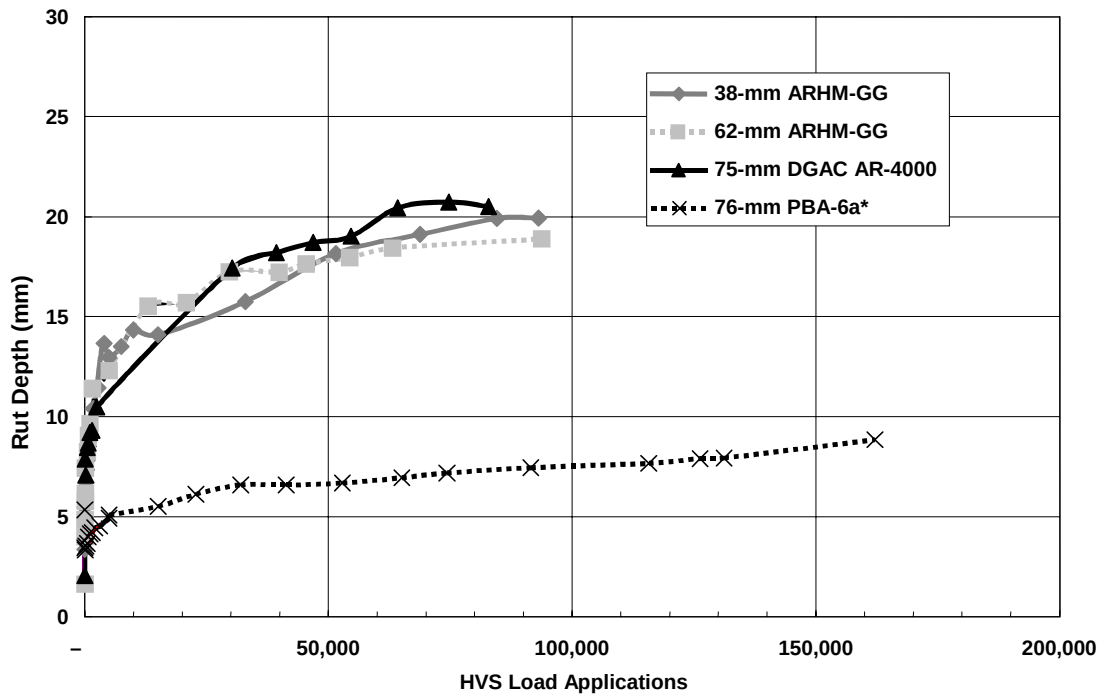


FIGURE 5 Rut depth versus HVS load applications with 40 kN load on dual tires at 50°C.

Construction Considerations

For successful performance of these pavement structures, strict attention to pavement construction will be required. This necessitates careful control both of the mix components and mix compaction.

Prior to construction, shear and fatigue test data must be submitted by the contractor to Caltrans for mix approval. This requirement has been incorporated in the project specifications to insure that the mixes to be used by the contractor meet the performance characteristics shown in Table 1 (*7*). These characteristics correspond to those used in the mix and pavement design processes. In addition, prior to construction, materials must be submitted to Caltrans for verification of these mix characteristics.

During mix production the contractor is required to provide the minimum process control requirements shown in Table 2 (*7*).

Compaction and other quality control requirements are summarized in Table 3 (*7*). The mixes containing the PBA-6a* mix and the AR-8000 at a binder content of 4.7 percent should be compacted to an air void content of about 6 percent [93 to 97 percent of theoretical maximum density (ASTM D 2041)], whereas the rich-bottom mix should be compacted to an air-void content of not more than 3 percent. It should be emphasized that this project, which uses ASTM D 2041 as the basis for compaction control, is a departure from the current Caltrans procedure (*4*).

Current Caltrans practice does not require a tack coat between lifts for multiple lift construction; the decision to use a tack coat is made on a case-by-case basis by the resident construction engineer. For the I-710 project, this practice has been changed and a tack coat will be required between each lift. This change in practice is based upon an evaluation of the performance of HVS test sections as a part of the CAL/APT program.

TABLE 1 Asphalt Concrete Mixture Performance Requirements (*7*, Table 39-3A)

Design Parameters		Test Method	Minimum Requirement
Permanent Deformation	PBA-6a* (modified)[2]	AASHTO TP7-94 modified[1]	660,000 stress repetitions[3,4]
	AR-8000[2]	AASHTO TP7-94 modified[1]	132,000 stress repetitions[3,4]
Fatigue	PBA-6a* (modified) [5,6]	AASHTO TP8-94 modified[1]	7,000,000 repetitions[4,8] 60,000,000 repetitions[4,9]
	AR-8000[5,7]	AASHTO TP8-94 modified[1]	300,000 repetitions[4,8] 15,000,000 repetitions[4,9]

NOTES: [1] Included in the testing guide provided upon request.
[2] At proposed asphalt binder content and with mix compacted to 3±0.3% air-void content.
[3] In repeated simple shear test at constant height (RSST-CH) at a temperature of 50°C.
[4] Mean of 3 specimens.
[5] At proposed asphalt binder content and with mix compacted to 6±0.3% air voids [determined using AASHTO 209 (Method A)].
[6] At proposed asphalt binder content, minimum stiffness at 20°C and a 10 Hz load frequency must be equal to or greater than 150,000 psi (1000 MPa). At proposed asphalt binder content, minimum stiffness at 30°C and a 10 Hz load frequency must be equal to or greater than 45,000 psi (300 MPa).
[7] At proposed asphalt binder content and 6±0.3% laboratory air voids [determined using AASHTO 209 (Method A)], minimum stiffness at 20°C and a 10 Hz load frequency must be equal to or greater than 900,000 psi (6200 MPa). At proposed asphalt binder content plus 0.5 percent and 3±0.3% laboratory air voids [determined using AASHTO 209 (Method A)], minimum stiffness at 20°C and a 10 Hz load frequency must be equal to or greater than 990,000 psi (6800 MPa).
[8] At 300×10^{-6} mm/mm. Results shall be reported for this strain level but may be obtained by extrapolation. Minimum number of repetitions required prior to extrapolation defined within test procedure.
[9] At 150×10^{-6} mm/mm. Results shall be reported for this strain level but may be obtained by extrapolation. Minimum number of repetitions required prior to extrapolation defined within test procedure.

CONSTRUCTIBILITY CONSIDERATIONS

The project specifications have been written to allow the Contractor to overlay and reconstruct from 10 p.m. Friday to 5 a.m. Monday. A construction window of 55 h has been provided with the anticipation that the project will completed during 10 or fewer weekend closures. A total of about 4.8 km (3 mi.) of the freeway will be rehabilitated, including 2.8 km (1.8 mi.) of the break, seat, and overlay alternative (BSOL), shown in Figure 3b, and 2.0 km (1.2 mi.) of the full depth asphalt concrete (AC), shown in Figure 3a. For the full depth alternative, some excavation is required of the cement treated base and aggregate subbase so that the clearance under the structures can be increased by 100 mm (4 in.).

The project schedule is shown in Figure 6. Median and shoulder reconstruction will be done during nighttime closures whereas the BSOL and full-depth AC will be constructed during weekend closures, as noted above.

Stage construction will be done by splitting the 4.8-km (3.0-mi.) project into two equally divided segments in each direction, a total of four segments as seen in Figures 7 and 8. Three lanes in one direction will be closed and traffic switched to the other side, as shown in Figure 8. As seen in this figure, the use of a movable median barrier and crossovers result in counterflow on one roadway during construction. Two or three weekend closures are planned for each segment as noted in Figure 8.

TABLE 2 Minimum Process Control Requirements (*7*, Table 39-4)

Quality Characteristic		Action Limit (Min.)	Test	Min. Sampling and Testing Frequency	Point of Sampling	Reporting Time Allowance
Data Used for Specifications						
Sand Equivalent		47	CT 217[1]	One sample per 2000 tonnes. Not less than one sample per day	Batch Plant from hot bins or Drum Plant from cold feed	24 h
Percent of Crushed Particles	Coarse Aggregate	100%	CT 205	Not less than one sample per day		24 h
	Fine Aggregate (Passing 4.75-mm, Retained on 2.36-mm)	100%				
Hveem Stabilometer	PBA-6a* (modified)	TV_{S1}[2] TV_{S2}[3]	CT 366 See 1,3,4,6, 7,8	Sample at least once per 500 tonnes. See [9] for minimum testing schedule	Mat behind paver	36 h
	AR-8000	TV_{S3}[2] TV_{S4}[3]				
Data for Report Only						
Hveem Stabilo-meter	AR-8000 (rich bottom)	TV_{S5}[2] TV_{S6}[3]	CT 366 See [1,3,4,6,7,8]	Sample at least once per 500 tonnes. See [9] for minimum testing schedule	Mat behind paver	36 h
Laboratory Percent Air-Void Content	PBA-6a* (modified)	TV_{AV1}	CT 367 See [1,4,10]	See [9] for minimum testing schedule	Mat behind paver	36 h
	AR-8000	TV_{AV2}				
	AR-8000 (rich bottom)	TV_{AV3}				

NOTES: Min. = minimum; CT = California test method

[1] Reported value shall be average of 3 test results. Samples used for the 3 tests shall be from a single split sample.

[2] Do not modify CT 304.

[3] Perform CT 304, and then apply an additional 500 tamping blows at 500 psi (3400 kPa) at 140°F (60°C).

[4] Sets of 3 briquettes must be prepared and tested to meet conditions of Note 3 and 4 separately.

[5] Limited reheat for sample preparation to 2 h. Do not place sample or briquette in oven for 15-h curve.

[6] Briquettes shall be fabricated from a single, combined sample obtained from at least 4 locations across the mat behind the paver in conformance with the requirements of CT 125.

[7] If the range of stability for the three briquettes is more than 12 points, the samples shall be discarded and new samples shall be obtained before the end of the following shift of paving and tests per Table 39-3.

[8] During production start-up evaluation, a correlation factor for cured versus uncured specimens shall be established in conformance with the requirements of Section 39-10.01A, "Production Start-Up Evaluation."

[9] AC will be sampled each 500 tonnes. Each type of AC shall be tested each day the first 5 days (or at least per 2,000 tonnes of production) and testing may be decreased to one per 5,000 tonnes thereafter unless stability falls below the action limit. Samples shall be retained to define limits of problem areas should the stability fall below the action limit. When stability falls below the action limit, testing will be increased to one test for each of the first 2,000 tonnes and may be decreased to one per each 5,000 tons thereafter. Each AC type being produced and placed shall be sampled and tested at least once per 55-h window if the quantity is less than 2,000 or 5,000 tonnes as it applies to the interval. The sequence of the first 5 test results shall not be broken by more than 7 days of non-production.

[10] Use CT 308A for determination of bulk specific gravity and AASHTO T209 (Method A) for maximum theoretical specific gravity.

TABLE 3 Minimum Quality Control Requirements (*7*, Table 39-9)

Index (i)[5]	Quality Characteristic		Specification Limits	Weighting Factor (w)	Test Method	Minimum Sampling and Testing Frequency	Point of Sampling
1	Asphalt Content		TV[1]±0.3%	0.30	CT 379 or CT 382	One sample per 500 tonnes or part thereof Not less than one sample per day	Mat behind paver
	Gradation	25.4 mm			CT 202	One sample per 500 tonnes or part thereof Not less than one sample per day	Batch plant from hot bins or Drum Plant from cold feed
2		19 mm	TV±5%	0.01			
3		12.5 mm	TV±5%	0.02			
4		9.5 mm	TV±6%	0.02			
5		4.75 mm	TV±7%	0.02			
6		2.36 mm	TV±5%	0.04			
7		600µm	TV±4%	0.05			
8		300µm	TV±4%	0.07			
9		75µm[2]	TV±2%	0.07			
10	Percent of Maximum Theoretical Density for given binder	PBA-6a* (modified)	93–97%	0.40	CT 375	One sample per 500 tonnes or part thereof Not less than one sample per day	Finished mat after final rolling
		AR-8000	93–97%				
		AR-8000 (rich bottom)	97–100%				
		AR-8000 (working platform)	91–97%				
	Maximum Theoretical Density[3] % Air Voids[4]				CT 308 AASHTO T209 (Method A) AASHTO T269		Mat behind the paver
11	Mix Moisture Content		≤1%		CT 370	One sample for 1,000 tons but not less than one sample per day	Mat behind the paver
12	Asphalt and Mix Temperature		120°C to 190°C (Asphalt) ≤ 165°C (Mix)			Continuous using an automated recording device	Plant

NOTES: [1] TV = Target Value from Contractor's Mix Design Proposal
[2] The percent passing the 75-µm sieve shall be reported to the first decimal place (tenths).
[3] California Test (CT) 375, "Density of Asphalt Concrete Using a Nuclear Gage" modified to use maximum theoretical density in accordance with ASTM D 2041 (Rice Method) in lieu of test maximum density as provided in Part 5, "Determining Test Maximum Density."
[4] Report only.
[5] Quality characteristics 1 and 10 are defined as critical quality characteristics.

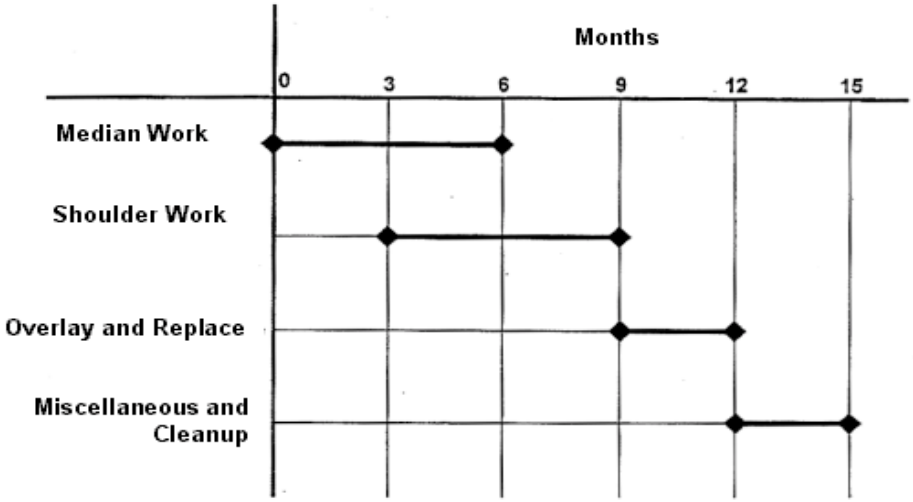


FIGURE 6 Staged rehabilitation construction schedule for I-710 project. (Source: 4th Meeting for Long-Life AC Pavement Rehabilitation Strategies with Caltrans and Southern California Asphalt Paving Association, June 11, 1999.)

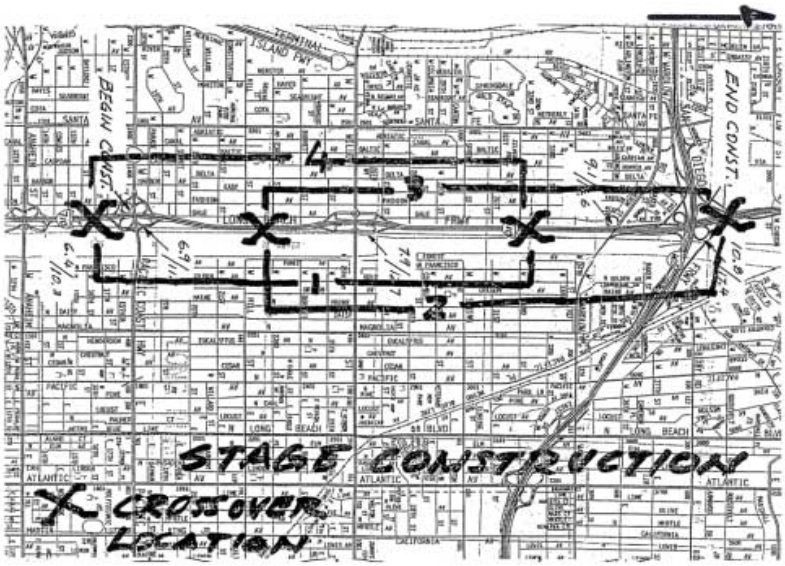


FIGURE 7 Site layout of the asphalt pavement project for I-710. (Source: 4th Meeting for Long-Life AC Pavement Rehabilitation Strategies with Caltrans and Southern California Asphalt Paving Association, June 11, 1999.)

An analysis has been made of the feasibility of one construction plan using the project management procedure described in Lee's Ph.D. dissertation at the University of California–Berkely (UC Berkeley) (*8*). This plan assumes that the BSOL section will be constructed in four lifts while the full-depth AC section will be constructed in five lifts. To assist in the analysis, the computer program CalCool (*9*), developed as a part of the CAL/APT program, was used to estimate pavement temperatures. According to Caltrans requirements the underlying layer in multi-lift construction must have cooled to 74°C (165°F) prior to placing the next lift. Since the paving sequence assumed single lane paving (3.7 m or 12 ft.) for three lanes, the program indicated that for the specific environment the lift cooling requirement would not slow construction. Figure 9 illustrates schematically the elements of the constructability analysis.

Comparison of the prediction using the procedure described by Lee (*8*) with that estimated by Caltrans is shown in Table 4. It would appear that the Caltrans estimate for the BSOL part of the construction can be completed as anticipated. On the other hand, the full-depth AC construction expectations of Caltrans may not be attainable.

A construction alternative under consideration and differing from that assumed in the analysis described herein is placing the pavement sections to the top of the second lift for the BSOL and the top of the third lift for the full-depth AC (i.e., to the top of the layer containing the AR-8000 mix). This alternative would then be opened to traffic and the PBA-6a* mix and open-graded mix would be placed in a continuous paving operation subsequently. An analysis of traffic effects on pavement performance for this approach was included in the original design considerations as noted earlier. Preliminary calculations suggest that this alternative may increase the likelihood that the full-depth AC construction would be completed in the planned time frame as compared to that shown in Table 4.

Comparison of the forecasted progress with that actually achieved during construction should provide data for improved construction guidelines for future projects. A demonstration of the potential usefulness of the program has already been accomplished during concrete reconstruction of a portion of the I-10 Freeway in Pomona, California (*10*).

SUMMARY

The I-710 project has provided an opportunity to implement approaches for mix and pavement design developed during the SHRP program (*1, 2*) and improved upon in the CAL/APT program, particularly the use of the shear test for permanent deformation evaluation and use of the flexural fatigue test and mechanistic-empirical design concepts as a part of structural thickness determination (*6*).

Results of the CAL/APT program have been incorporated in the design and construction requirements for the pavement section including improved AC compaction, the use of the "rich bottom" concept, and the incorporation of a tack coat between succeeding layers of multiple lift AC construction (*6*).

Finally, it must be emphasized that the process in arriving at the designs as well as the construction requirements was a "partnered" effort between Caltrans, the Asphalt Industry in California, and academia through UC Berkeley. This partnering provided an excellent opportunity to successfully implement new ideas and research results on this challenging project for which some of the traditional approaches were insufficient. Hopefully such working together can continue in the future to the benefit of California's traveling public.

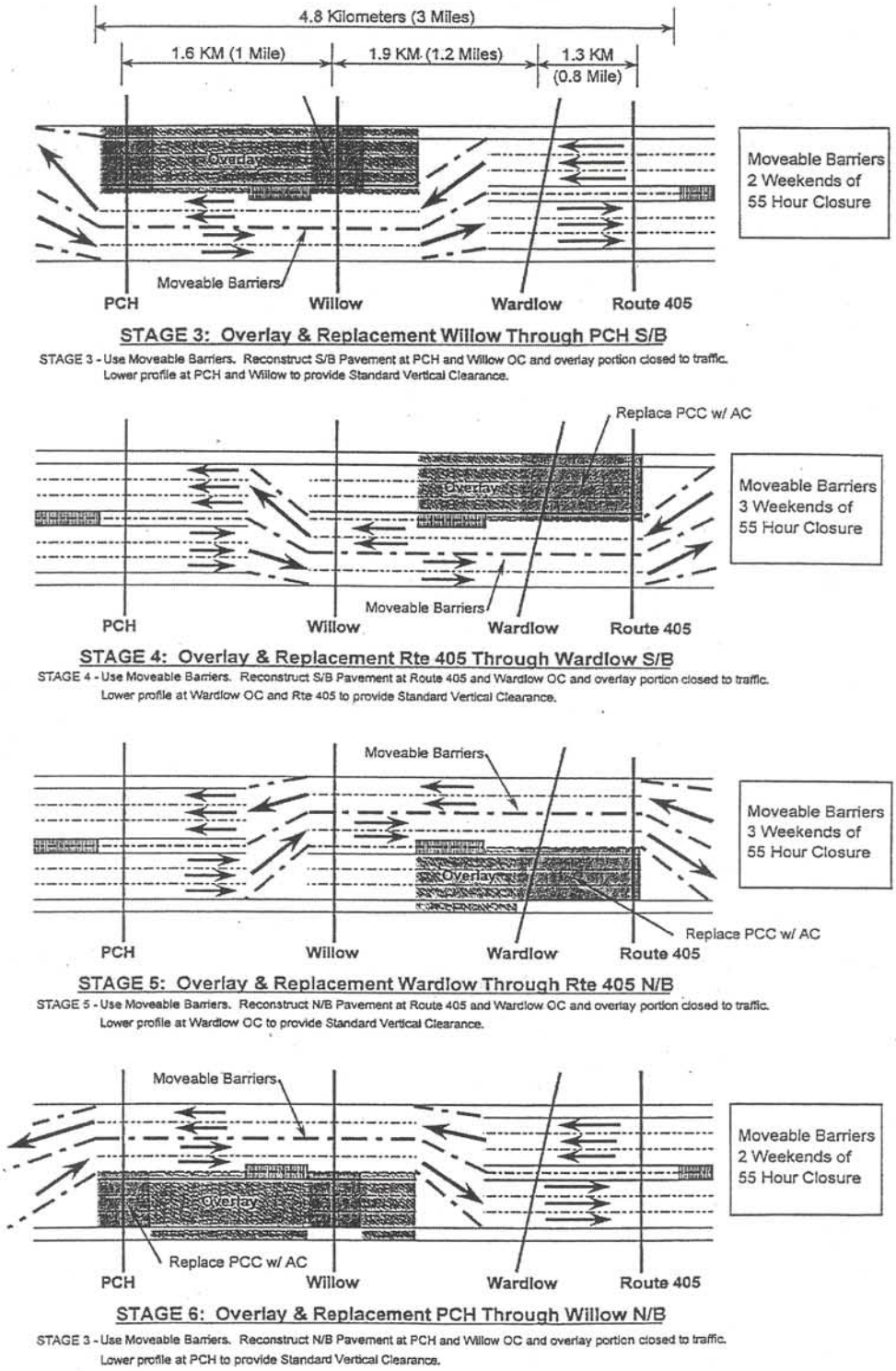


FIGURE 8 Schematic of the stage construction for the I-710 project. (Source: 4th Meeting for Long-Life AC Pavement Rehabilitation Strategies with Caltrans and Southern California Asphalt Paving Association on June 11, 1999.)

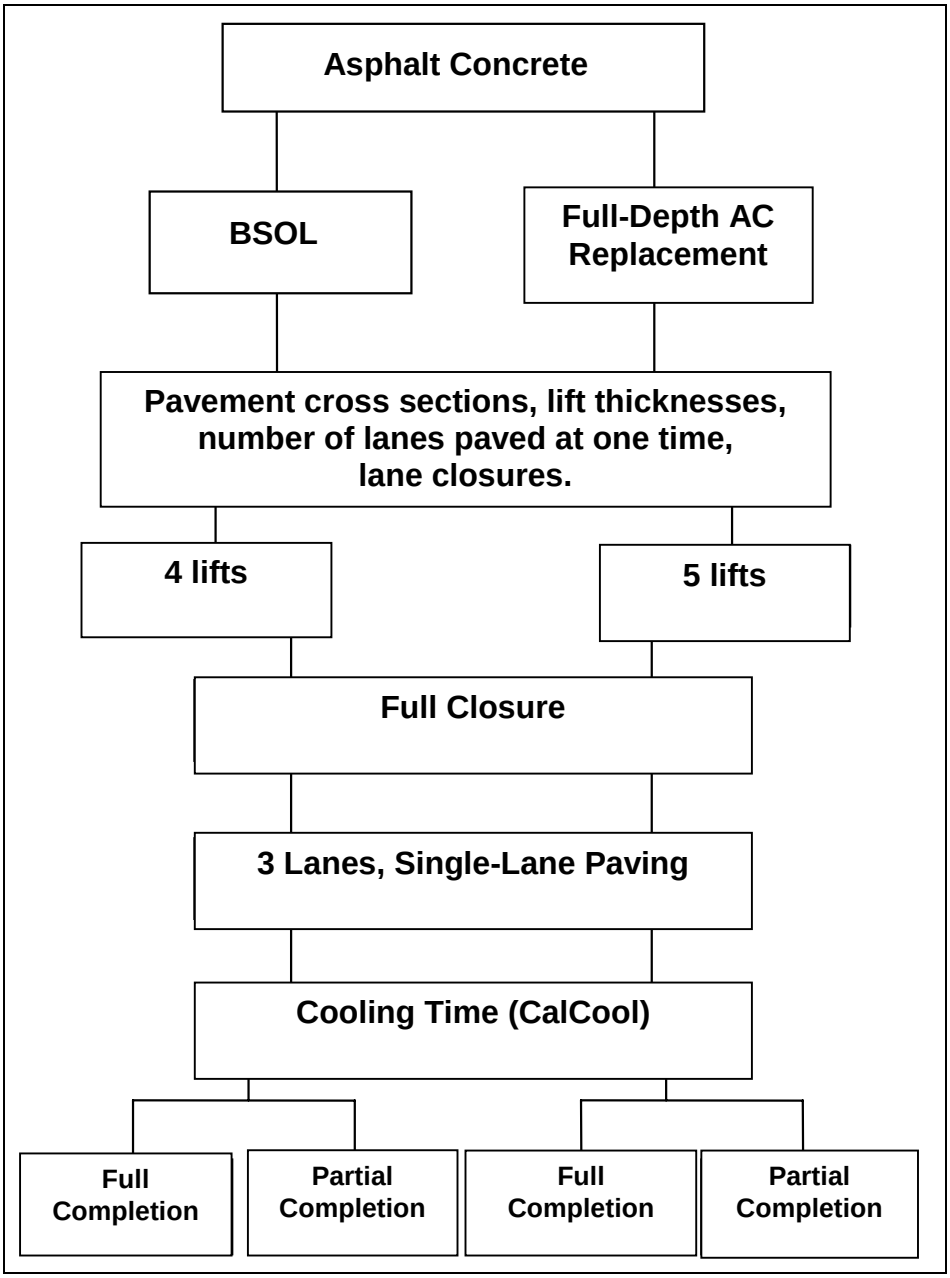


FIGURE 9 Analysis considerations, 55-hour closure constructibility program (summarized in Table 4).

TABLE 4 Productivity Estimates, I-710: Caltrans Versus University of California–Berkeley Model

Pavement Section	Section Length	Caltrans Plan (km per stage)		UC Berkeley Model (km)	Feasibility of Caltrans Plan[1]
		Stages 3, 6	Stages 4, 5		
BSOL	Centerline—km	1.60	1.10	1.55	can be achieved
	Total lane—km	4.80	3.30	4.83	can be achieved
Full Depth	Centerline—km	0.80	0.65	0.50	likely will not be achieved
	Total lane—km	2.40	2.00	1.50	likely will not be achieved

NOTE: BSOL = break, seat, and overlay. Model procedure validated for concrete pavement construction in *Case Study of Urban Concrete Pavement Reconstruction and Traffic Management for the I-10 (Pomona, Calif.) Project* (*10*).

[1] Based on model estimates (*8*).

REFERENCES

1. Monismith, C. L., R. G. Hicks, F. N. Finn, J. Sousa, J. Harvey, S. Weissman, J. Deacon, J. Coplantz, and G. Paulsen. *Permanent Deformation Response of Asphalt-Aggregate Mixes.* Report no. SHRP-A-415. SHRP, National Research Council, Washington, D.C., Aug. 1994.
2. Tayebali, A. A., et al. *Fatigue Response of Asphalt-Aggregate Mixes.* Report no. SHRP-A-404. Strategic Highway Research Program, National Research Council, Washington, D.C., 1994.
3. *Standard Specifications.* California Department of Transportation, Sacramento, July 1999.
4. Deacon, J. A., J. S. Coplantz, A. A. Tayebali, and C. L. Monismith. Temperature Considerations in Asphalt Aggregate Mixture Analysis and Design. *Transportation Research Record 1454,* TRB, National Research Council, Washington, D.C., 1994, pp. 97-112.
5. Harvey, J. T., J. A. Deacon, B. W. Tsai, and C. L. Monismith. *Fatigue Performance of Asphalt Concrete Mixes and Its Relationship to Asphalt Concrete Pavement Performance in California.* University of California, Berkeley, Institute of Transportation Studies, Asphalt Research Program, CAL/APT Program, 1995.
6. Harvey, J. T., J. Roesler, N. F. Coetzee, and C. L. Monismith. *Caltrans Accelerated Pavement Test Program, Summary Report Six Year Period: 1994–2000.* Prepared for the California Department of Transportation, Pavement Research Center, University of California, Berkeley, June 2000.
7. Special Provisions for Construction on State Highway in Los Angeles County in Long Beach from Route 1 to Route 405/710 Separation. California Department of Transportation, District 08, Route 710, Contract No. 07-1384U.4, Oct. 2000.

8. Lee, E. B. *Construction and Productivity Analysis for Long-Life Pavement Rehabilitation Strategies (LLPRS).* Ph.D. dissertation, University of California–Berkeley, Fall 2000.
9. Timm, D., V. Voller, E. B. Lee, and J. T. Harvey. *CalCool—Computer Program for Temperature Prediction in a Multi-Layer Asphalt Pavement During Construction.* Pavement Research Center, University of California, Berkeley, Aug. 2000.
10. Lee, E. B., J. R. Roesler, J. T. Harvey, and C. W. Ibbs. *Case Study of Urban Concrete Pavement Reconstruction and Traffic Management for the I-10 (Pomona, CA) Project.* Report to Innovation Pavement Research Foundation, Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley, May 2000.

Hot-Mix Asphalt Layer Thickness Design for Longer-Life Bituminous Pavements

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The performance of a hot-mix asphalt (HMA) pavement structure is dependent on the interaction between pavement responses and the strength and modulus of the different layers. Wheel loads induce stresses and strains in each layer, which can result in damage of bound and unbound materials. The accumulation of damage within the pavement layers eventually becomes visible at the surface of the pavement in the form of rutting, cracking, and surface roughness. Structural deterioration is normally associated with cracking and rutting. These two distresses, along with the cumulative or incremental damage concept, historically have been used in determining the layer thickness requirements needed to resist structurally related distresses.

An overview is presented of a procedure that has been used to design long-life HMA pavement structures for heavily traveled roadways. The design procedure is based on limiting the tensile strain at the bottom of the HMA layer and the vertical compressive strain at the top of the subgrade or embankment soil. Long life is defined in this paper as 40+ years without any major structural failures throughout the HMA layer. The design traffic used in previous design studies has exceeded 40 million equivalent single-axle loads.

The methodology applies the cumulative damage concept in the prediction of fatigue (load-related cracking) and subgrade distortion. Seasonal and other variations in material properties, including the modulus of the HMA layer, are considered in the procedure by use of the "equivalent modulus" concept. In other words, the incremental damage computed in the HMA layer for a specific modulus would be equal to the summation of the incremental damage computed allowing the modulus of the HMA layer to change with the season. Two other criteria are used for the mechanistic–empirical thickness design checks. One is based on limiting the maximum surface deflection under the design load and the other is based on limiting the modulus ratio between two adjacent unbound pavement layers.

With this methodology, three key issues related to fatigue cracking in HMA layers are addressed. The first issue is the reality of the concept of an "endurance limit" for the layer thickness design of HMA layers. The second issue is the location of where load-related cracks initiate in the HMA layer (at the bottom of the layer versus the top of the layer), and the third issue is confirmation of the fatigue characteristics of HMA layers for use in layer thickness design. Some of the Long-Term Pavement Performance data, both materials and distress data, are used to support the criteria employed in the design procedure and the methodology of that procedure.

A pavement's service life is defined as that period of time from completion (or opening to traffic) until the condition of the pavement is considered unacceptable and rehabilitation or replacement is required. The determination of pavement life is dependent upon the structural design requirements, material characteristics, layer thickness, maintenance activities and failure criteria established by the agency or owner.

For high volume roadways, designing pavements for long service lives is becoming more common in the industry. In fact, the industry is referring to these types of pavements as perpetual designs. Perpetual or long life is defined in this paper as 40+ years without any major structural failures or repairs. In other words, the repair or rehabilitation of these types of pavements is

limited to deterioration that initiates at the surface (i.e., a repair strategy of shave and pave or mill and replace the surface layer).

Pavement performance is normally predicted as a function of several factors that can be divided into the following major categories:

1. Traffic loading [normally expressed in terms of 18 kip equivalent single-axle loads (ESALs)];
2. Climatic condition (e.g., precipitation, temperature, and freezing index);
3. Subgrade parameters (e.g., subgrade soil type, resilient modulus, and other physical properties);
4. Pavement parameters (e.g., layer thickness, drainage, and age);
5. Pavement materials (e.g., resilient modulus, strength, and other physical properties); and
6. Maintenance level (e.g., amount of patching, crack sealing).

The performance of a pavement structure is dependent upon the interaction between pavement response and strength of the different layers. Wheel loads induce stresses and strains in each layer, which can result in permanent deformation or damage of pavement materials. The accumulation of permanent deformation and damage of the pavement eventually becomes visible at the surface of the pavement in the form of rutting, cracking, or surface roughness. Since pavement structural deterioration is normally associated with cracking and rutting, these two distresses, along with the accumulative damage concept, have been used in developing the pavement thickness and material combinations needed to prevent structurally related distresses.

The materials and environmental types of surface distress are generally the limiting factors in determining the service life of flexible pavements and hot-mix asphalt (HMA) overlays. Improved material and mixture selection and construction specifications are used to reduce the expected occurrence of the material and environmental related distresses. This paper presents a summary and overview of a design procedure that has been used for designing long-life HMA pavements for traffic levels exceeding 40 million ESALs.

THICKNESS DESIGN METHODOLOGY

The structural deterioration of flexible pavements is associated with cracking in the HMA surface or development of ruts in the wheel path. The methodology applies the cumulative damage concept in the prediction of these two modes of distress. Use of the cumulative damage concept permits accounting, in a rational manner, for damage caused by each load application.

Seasonal and other variations in material properties and modulus of each layer with different loads can be considered in these predictions of damage. Evaluations of design life for candidate pavement structures are based on computing of damage caused by each truck type and load (or an 18-kip ESAL) for different seasons of the year, and then summing the results to obtain the total damage to the pavement structure.

The objective of the design effort is to provide pavement structures that will serve the design traffic levels projected through a design period before experiencing failure. Failure of flexible pavements is defined as alligator cracking over 10 percent of the area subjected to wheel loads or one-half inch of rutting. The failure of a pavement system under this concept is assumed to occur when the damage index reaches a fixed amount, generally 1.0. It should be understood that a damage index of one does not necessarily imply a functional failure, but is instead that level of damage selected as sufficient to warrant maintenance or rehabilitation.

For this study, a damage index of one means the pavement has been subjected to a sufficient number of wheel loads to cause 10 percent alligator cracking or 0.5 inches of rut depth (as defined by the SHRP pavement distress manual) (*1*). These values of 10 percent cracking and 0.5-inch rut depth were selected because previous studies of in-service pavements have indicated that these levels will usually trigger some type of pavement rehabilitation.

Two other criteria were used for the mechanistic–empirical thickness design checks. One is based on limiting the maximum surface deflection and the other is based on limiting the modulus ratio between two adjacent unbound pavement layers. These two criteria and the rutting and fatigue criteria used for the design checks are discussed and defined in the following paragraphs.

STRUCTURAL RESPONSE DESIGN CRITERIA

Limiting Modulus Ratio Criteria for Unbound Aggregate Layers

The long term in place modulus of unbound base and subbase layers are dependent on the modulus of the supporting layer because of potential de-compaction in the lower portion of these layers. The U.S. Army Corps of Engineers developed criteria to limit the modulus of unbound aggregate layers based on the thickness of an unbound aggregate layer and the modulus of the layer supporting the unbound aggregate layer (*2*). These limiting modulus ratio criteria are graphically shown in Figure 1 and were used in determining the maximum layer modulus of unbound aggregate base and subbase layers.

Subgrade-Embankment Protection Criteria

Rutting or surface distortion is considered to occur primarily in the subgrade and has been related to the vertical compressive strain at the top of the subgrade by the following empirical functional form.

$$\log N_{fv} = b_3[\log(M_{R(\text{soil})})] - b_2[\log(\varepsilon_{vs})] - [\beta_v b_1] \quad (1)$$

where

N_{fv} = number of load repetitions for subgrade distortions that cause surface distortions exceeding 0.5 inches in depth,
$M_{R(\text{Soil})}$ = design resilient modulus of the subgrade soil or foundation (psi),
(ε_{vs}) = vertical compressive strain at the top of the subgrade soil or foundation;
b_1, b_2, b_3, = soil properties from repeated load triaxial tests [b_1 is also adjusted to correlate the laboratory test results to field observations ($\beta_v\, b_1 = 10.90$, $b_2 = 4.082$, and $b_3 = 0.955$)], and
β_v = field calibration factor for subgrade distortion–vertical strain.

This assumption implies that the structural layers above the subgrade or foundation will be constructed such that only negligible rutting will occur within those layers. Figure 2 shows the relationship between vertical compressive strain and number of wheel load applications for various subgrade stiffness values that were utilized to approximate a failure level of subgrade rutting or distortion (*2, 3*).

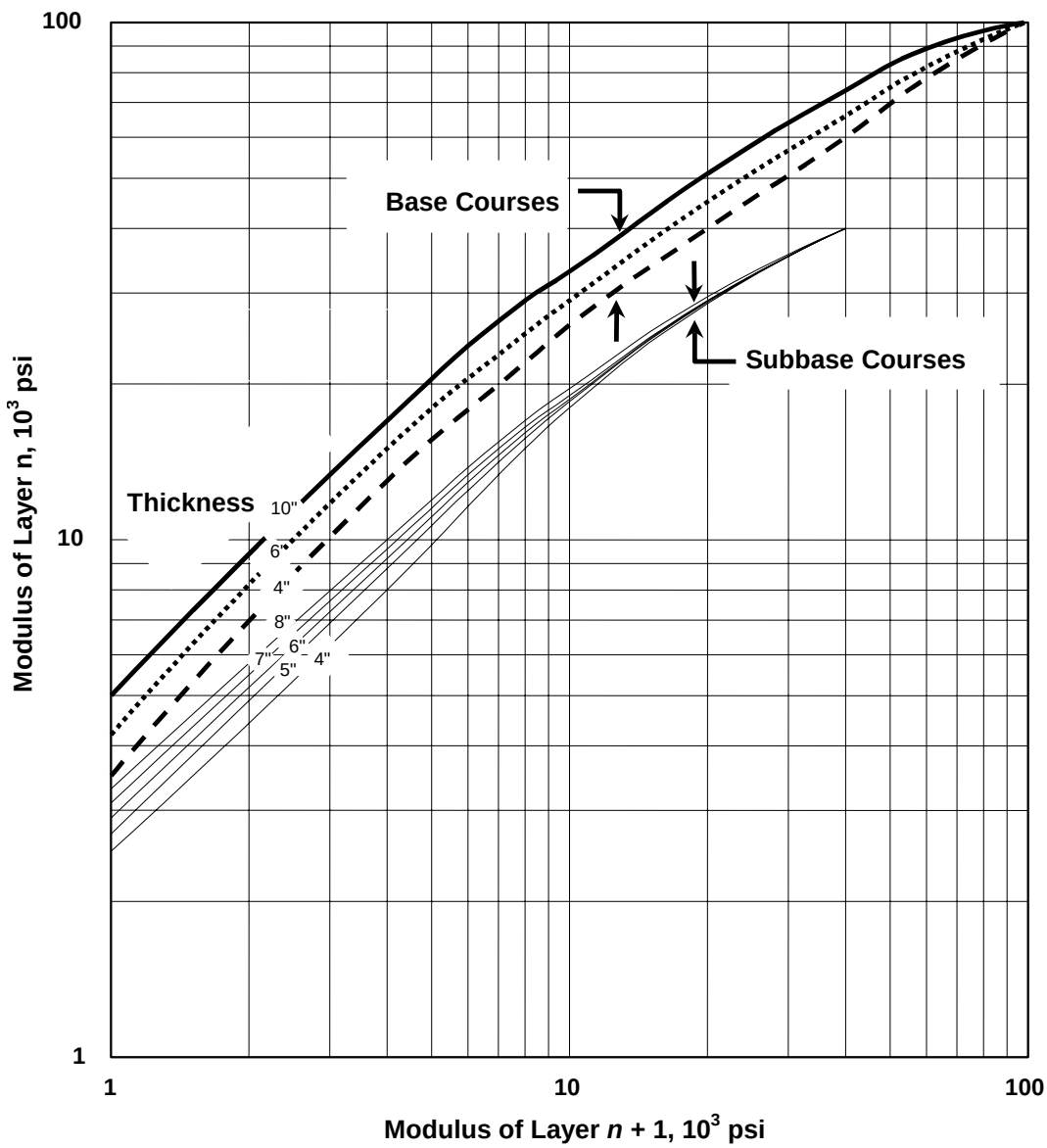

FIGURE 1 Limiting modulus criteria of unbound aggregate base and subbase layers (*2*) (1 in. = 0.0254 m; 10^5 psi = 698 MPa).

The soil properties b_1, b_2, and b_3 are obtained through triaxial repeated load permanent deformation tests in the laboratory. The regression coefficient b_1 is shifted or adjusted to correlate laboratory results to a specific level of subgrade distortion. This assumes that the materials placed above the foundation will be properly compacted and of sufficient strength so that significant permanent deformation will not occur in those layers.

Fatigue Cracking Criteria

Fatigue or alligator cracking in flexible pavements results primarily from repeated wheel loads and has been related to the horizontal tensile strain at the bottom of the HMA layer by the following empirical functional form (*4*).

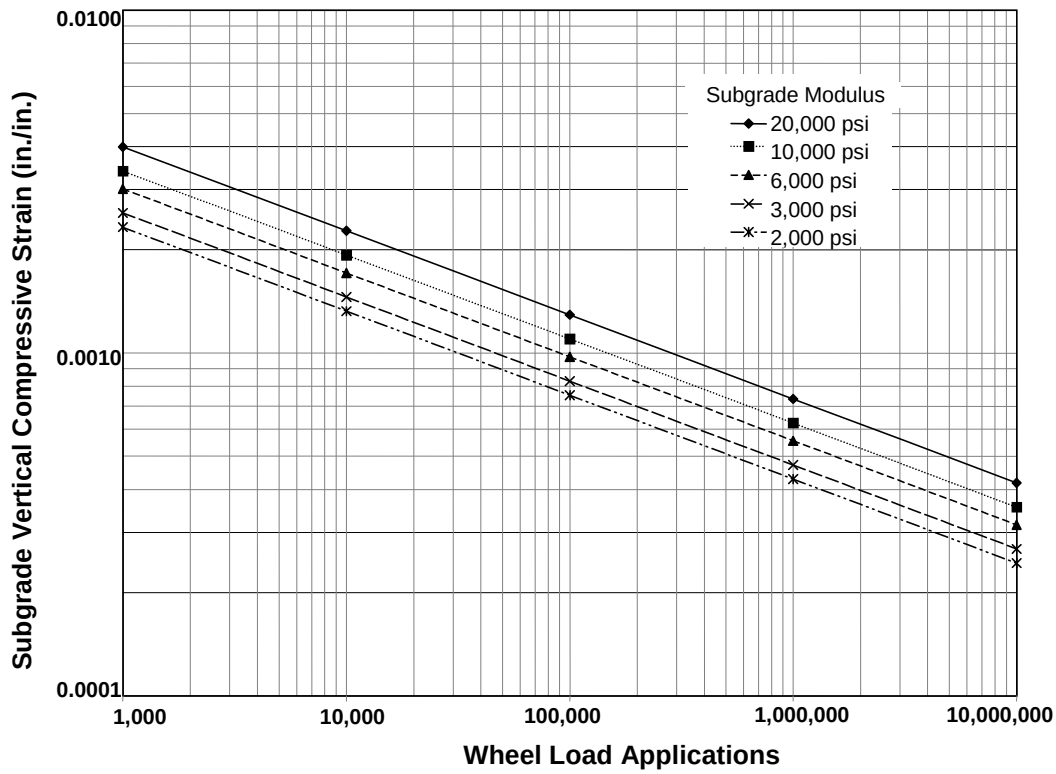

FIGURE 2 Relationship between subgrade vertical compressive strain and wheel load applications for the deformation failure criteria (*2*).

$$\log N_{ft} = \beta_t k_1 - k_2[\log(\varepsilon_t/10^6)] - k_3[\log E/10^3)] \quad (2)$$

where

N_{ft} = number of load repetitions to specific level of fatigue cracking;
ε_t = tensile strain at the bottom of the HMA layer;
E = modulus for the HMA mixture (psi);
k_1, k_2, k_3 = HMA material properties determined from beam fatigue tests ($\beta_t k_1$=14.820 for crack initiation, $\beta_t k_1$=15.947 for less than or equal to 10 percent area cracking, and $\beta_t k_1$=16.086 for more than 45 percent area cracking; k_2=3.291; k_3=0.854); and
β_t = field calibration factor for fatigue cracking–tensile strain.

The material properties k_1, k_2, and k_3 are obtained through fatigue beam testing in the laboratory. The shift factor β_3 is used to correlate laboratory results to actual field behavior at different levels of load-related cracking. The shift factor varies by the failure criteria (i.e., the extent of fatigue cracking) and can be dependent on the HMA mixture composition. Von Quintus and others found that the field calibration factor was highly dependent on the indirect tensile strain at failure and on the total resilient modulus (*5, 6*). Figure 3 shows the relationship used in this study between HMA tensile strain and number of load applications to produce 10 percent alligator cracking for various HMA modulus values, assuming that the load-associated cracks initiated at the bottom of the HMA layer (*4*).

Total fatigue cracking in the field is predicted using the cumulative damage approach. Miner's law is used to accumulate the fatigue cracking damage for the different wheel loads on a seasonal basis over the analysis period using the following relationship:

$$D_k = \sum_{j=1}^{k} \sum_{i=1}^{m} \frac{n_{ij}}{N_{fij}} \tag{3}$$

where

D_k = fatigue damage through season k,
m = number of load classes,
n_{ij} = actual number of load repetitions for load class i (i = 1,…, m) during season j, and
N_{fij} = number of load repetitions for load class i and season j to reach failure.

Load-related fatigue cracks also initiate at or near the surface and propagate downward. This type of fatigue cracking is believed to occur more commonly on pavements with thick HMA surface layers where large stiffness or modulus gradients exist. Surface-initiated fatigue cracks generally start as longitudinal cracks near the edge of the wheels. The mechanisms that cause these types of cracks to develop at the surface are believed to be a combination of the tensile and shear modes adjacent to the wheel loads and are discussed in a later section of this paper. Extensive validation of the mechanisms has yet to be finalized.

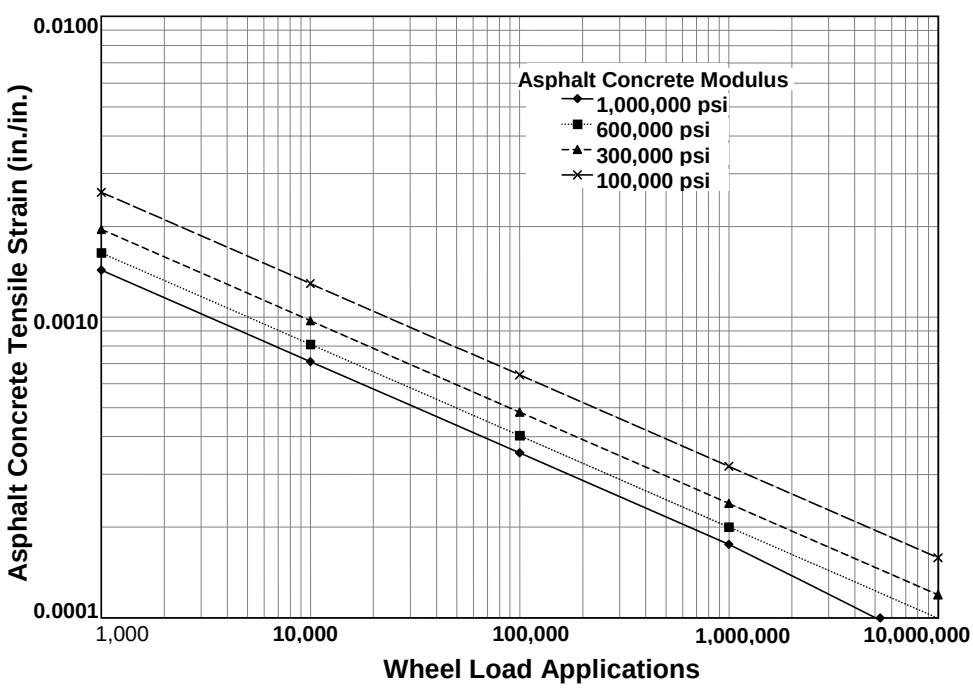

FIGURE 3 Relationship between asphalt concrete tensile strain and wheel load applications for the alligator cracking failure criteria (*4*).

Maximum Surface Deflection Criteria—Overall Structural Adequacy Check

The maximum surface deflection under a 9-kip gear load or 18-kip axle load has been used for pavement design and evaluation. Various design criteria have been developed by different agencies, but the critical deflection relationships defined from the AASHO Road Test or Transport Research Laboratory are the ones most commonly used (*7, 8*). The critical deflections are used to judge the acceptability of the pavement design cross section and not to predict the occurrence of specific distresses.

MATERIALS RESPONSE CRITERIA

Permanent Deformation

Although the subgrade protection criteria assumes that no permanent deformation will occur within the pavement layers, surface distortions or rutting may be the consequence of permanent deformation (or plastic strains) within HMA layers, unbound aggregate layers, or both. The design methodology for long life HMA pavement should consider both contributions to rutting. The empirical permanent deformation model for both bound and unbound materials is a power law function of the following form.

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_i a N_b \tag{4}$$

where

ε_p = accumulated plastic strain in layer h_i after N load repetitions,
ε_r = resilient strain at the mid-depth of layer h_i,
N = number of load repetitions,
a, b = material properties, and
β_i = field calibration factor for permanent deformation in the pavement layers.

To calculate total pavement rutting, the pavement resilient strains are estimated at representative locations (e.g., midthickness of layers/sublayers). The rut depths for each layer are estimated for different wheel loads on a seasonal basis over the analysis period using the following relationship (*6*).

$$RD = \Sigma[\varepsilon_{pi}(N)h_i] \tag{5}$$

where

RD = total rut depth for the pavement layers and subgrade (inches),
$\varepsilon_{pi}(N)$ = accumulated plastic strain at N load repetitions for layer i, and
h_i = thickness of layer i (inches).

Thermal Cracking

The thermal cracking model is based on the original work done by Roque and Hiltunen during the SHRP program (*9*). This model was subsequently enhanced as part of the on-going work under NCHRP Project 9-19 (Superpave Support and Performance Models Management). The thermal cracking predictive model is founded on mechanics, specifically fracture mechanics, rather than on empirical relationships between laboratory-field performance and various predictor variables. The extent of cracking in the field is predicted from the model using an assumed relationship between a probability distribution for crack length and the percent of cracking:

$$\begin{aligned} C_f &= \beta_5 * \text{Prob}(\log C > \log h_{AC}) \\ &= \beta_5 * \text{N}\left(\frac{\log\left(C / h_{AC}\right)}{s_{\log a}}\right) \end{aligned} \tag{6}$$

where

C_f = observed amount of thermal cracking (expressed as the length of thermal transverse cracks occurring in a pavement length of 500 ft.),
β_5 = field calibration coefficient,
Prob( ) = probability,
C = predicted crack depth (from the Roque-Hiltunen mechanistic thermal cracking model),
h_{AC} = thickness of the asphalt layer,
N( ) = standard normal distribution, and
$s_{\log a}$ = standard deviation of the log of crack lengths in the pavement.

This thermal cracking model was calibrated on a combined set of 22 original SHRP thermal cracking sections, plus 14 C-SHRP and 5 MnRoad sections under NCHRP Project 1-37A (Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures).

DETERMINATION OF DESIGN LAYER MODULUS—EQUIVALENT DAMAGE CONCEPT

Most pavement designs are completed well in advance of the actual selection and use of specific materials and mixtures. When the actual materials and mixtures are unavailable for the structural design process, determination and use of the design layer modulus based on the equivalent damage concept is used instead of breaking a typical year into different seasons and calculating damage for each season and wheel load. In other words, determination is made of an equivalent layer modulus for the entire year that will result in the same total damage when calculating and summing the damage for each season of the year. The following provides an overview of determining the equivalent or design layer modulus for bound and unbound materials.

HMA Materials—Equivalent Temperature Concept

The equivalent annual modulus or the design modulus for HMA mixtures can be determined in accordance with the following equation when using the fatigue relationship of Equation 2 (*3, 7, 10*).

$$E_{\text{design}} = [\Sigma E(T)_i \times DF_i]/(\Sigma DF_i) \quad (7)$$

where

$$DF_i = 7.4754 \times 10^{10}[E(T)_i]^{-1.908} \quad (8)$$

E_{design} = The equivalent or annual design modulus for the HMA mixture (psi),
$E(T)_i$ = The HMA modulus for the average middepth pavement temperature (°F) for season i (measured in the laboratory or backcalculated from deflection basins and adjusted to laboratory conditions) (psi), and
DF_i = Fatigue cracking damage factor in season i.

Unbound Materials—Equivalent Seasonal Modulus

Damage factors for unbound aggregate base and subbase layers (based on fatigue cracking of the HMA layers) can be used to ensure that there is sufficient cover or surface thickness to prevent overstressing the base and subbase materials and inducing high tensile strains in the HMA surface layers during periods of increased moisture. The equivalent annual aggregate base or subbase modulus can be determined in accordance with Equation 9 (*10*).

$$M_{R(\text{Aggregate})} = \Sigma[(M_{RA})_i \times (UF)_i]/[\Sigma(UF)_i] \quad (9)$$

where

$$(UF)_i = 1.885 \times 10^3(M_{RA})_i^{-0.721} \quad (10)$$

$M_{R(\text{Aggregate})}$ = the equivalent annual resilient modulus for unbound aggregate base and subbase materials (psi),
$(UF)_i$ = damage factor for unbound aggregate base and subbase materials in season i, and
$(M_{RA})_i$ = resilient modulus of the unbound aggregate base or subbase measured in the laboratory for a moisture content in season i (psi).

Permanent deformation damage factors (based on distortion in the subgrade) are used to ensure that there is sufficient cover to prevent overstressing and excessive permanent deformation in the subgrade during periods of increased moisture. The following equations can be used to calculate an equivalent annual or design resilient modulus for the subgrade soil (*10, 11*).

$$M_{R(\text{Soil})} = \Sigma[(M_{RS})_i(US)_i]/\Sigma(US)_i \quad (11)$$

$$(US)_i = 4.022 \times 10^7(M_{RS})_i^{-1.962} \quad (12)$$

where

$M_{R(\text{Soil})}$ = equivalent or design resilient modulus of the subgrade soil (psi),
$(M_{RS})_i$ = resilient modulus for the subgrade soil measured in the laboratory for a moisture content or physical condition within season *i* (psi), and
$(US)_i$ = damage factor for the subgrade soil in season *i*.

Backcalculated Versus Laboratory-Determined Layer Modulus

The design modulus for each layer within the pavement structure represents the modulus of that layer determined from laboratory tests. If the layer modulus is backcalculated from deflection basin data, the layer modulus is reduced according to the types of materials in accordance with the procedure established by Von Quintus et al. (*3, 12*). The reason for reducing the backcalculated layer modulus is that the structural response and calibration factors were based on laboratory-measured modulus, rather than on backcalculated elastic layer modulus.

KEY DESIGN ISSUES

This section of the paper identifies and briefly discusses some of the key issues related to the design of long-life HMA pavements. These issues are three—the applicability of an endurance limit for HMA mixtures, the applicability of the criteria for surface initiated fatigue cracks, and site factors and conditions affecting design.

Endurance Limit for HMA Mixtures

The concept of an endurance limit is used in other disciplines but has not been accepted or used for designing HMA pavements. The endurance or fatigue limit is defined as the horizontal asymptote of the relationship between the applied stress or strain and the number of load repetitions, such that a lower stress or strain will result in an infinite number of load repetitions.

For long-life HMA pavements, the applicability of an endurance or fatigue limit has become a key issue in determining the HMA layer thickness requirements. With the fatigue equations that have been developed over the years, the greater the design traffic, the thicker the pavement. Whether HMA has an endurance limit as a material/mixture property is debatable. However, there is a limit for which the allowable number of load applications become so large that laboratory beam fatigue tests confirming the fatigue strength of the mixture become impractical. This value seems more of a “practical” limit than an endurance limit.

There have been only a few studies or papers suggesting values for the endurance limit. Most values used or suggested are tensile strains less than 0.000100 in./in. The author has used a value of 0.000065 in./in. at the equivalent annual pavement temperature for this “practical” limit for HMA layers. This value was obtained from a limited amount of data based on a limiting strain ratio rather than a specific value, and it is believed to be more realistic. The strain ratio is defined as the initial tensile strain applied to the HMA layer divided by the tensile strain at failure, as measured from the indirect tensile test. Figure 4 shows the relationship between the indirect tensile strain at failure and total resilient modulus (*6*). In a few limited studies, HMA mixtures were found to have no area fatigue cracks when the ratio is less than 10 percent (*13*). This criterion has been used for determining the HMA layer thickness required for long-life pavements but should be confirmed through extensive laboratory and field studies.

Surface-Initiated Fatigue Cracks

Most load related fatigue analyses for determining the HMA layer thickness assume that the cracks initiate at the bottom of the HMA layer and propagate upward to the surface of the pavement. For thick HMA layers, however, there is more and more evidence and studies that suggest these load-related cracks can actually initiate at the surface and propagate downward. There are various opinions on the mechanisms that cause these types of cracks, but there is no conclusive data to suggest one is more applicable than the others. Some of the more common opinions are:

- Tearing of the HMA surface mixture from radial tires with high contact pressures near the edge of the tire causes the cracks to initiate and propagate in shear and in tension.
- Severe aging of the HMA mixture near the surface (large modulus gradient) resulting in high stiffness that, when combined with high contact pressures adjacent to the tire loads, causes the cracks to initiate and propagate in shear. And
- A combination of thermal strains and stresses and of tensile strains near the surface and adjacent to the tire loads causes the cracks to initiate and propagate in tension, a process accelerated by the aging of the HMA mixtures near the surface.

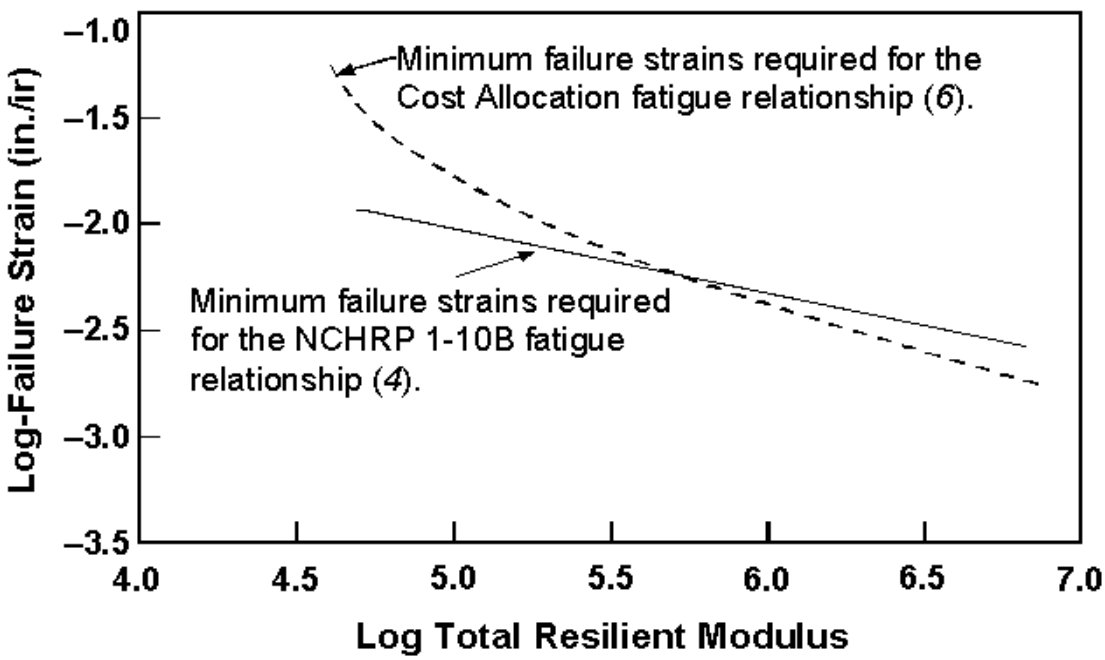

FIGURE 4 Relationship between the failure strain and total resilient modulus as measured using the indirect tensile test (*6*).

Site Factors and Conditions Affecting Design

Climate

One of the important although complex factors that affect pavement performance is the climate. This is especially true in those areas of the United States that experience large variations in temperature and moisture. Air temperatures and other climatic parameters are used to calculate the temperature throughout the pavement structure to determine the HMA modulus and pavement response characteristics. Temperatures are especially important in predicting the level of transverse cracking for a particular HMA mixture.

Moisture can also be important in determining the modulus of unbound pavement materials and soils, but is extremely difficult to predict. However, moisture has an important effect on the resilient modulus of the unbound materials and soils. Historical data should be used to determine the seasonal or monthly variations in the underlying unbound layers and supporting soils.

Subsurface and Drainage Investigations

It is critical that adequate subsurface investigations be performed for both new and rehabilitated HMA pavements to identify response characteristics of the supporting soils and subsurface water flow that can have a detrimental impact on the long-term performance of the pavement. Adequate drainage layers should be designed to prevent ground water or subsurface water flow from infiltrating and significantly reducing the strength of the unbound pavement materials and soils (*14*).

Subgrade Soil

The properties of the roadbed soil are essential inputs in the pavement design process. Methods characterizing soil properties include strength, resilient modulus, shrink-swell potential and frost susceptibility, as well as other types of properties. It is suggested that the unbound pavement layer supporting the HMA mixtures have a minimum equivalent modulus of 25,000 psi. Moderate to high frost-susceptible soils (as classified using the Army Corps of Engineers procedure) should be protected from frost penetration and freeze-thaw weakening. Expansive soils can be very destructive to the service life of all pavements, but this is difficult if not impossible to simulate this in design studies. When expansive soils are encountered along the project site, special precautions should be made to eliminate or significantly reduce the effect of differential volume change in the supporting soils.

SUMMARY

The methodology presented in this paper is believed to represent the state of the art in pavement design for long-life HMA pavements. Although the methodology offers capabilities unavailable through more conventional procedures and should be considered more accurate, its predictions of damage and/or distress may be claimed to be as good as the state of the art allows only if the traffic forecast (both in magnitude, axle weight, and tire pressures) is reasonably accurate. The point of this discussion is that these damage predictions offer valuable information for planning and design purposes, but that they are approximate, as are most engineering analyses involving soils and pavement materials.

REFERENCES

1. *Distress Identification Manual for the Long-Term Pavement Performance Project.* Report No. SHRP-P-338, SHRP, National Research Council, Washington, D.C., 1993.
2. Barker, W. R., and W. N. Brabston. *Development of a Structural Design Procedure for Flexible Airport Pavements.* Report No. FAA-RD-74-199. U.S. Army Engineer Waterways Experiment Station, FAA, Sept. 1975.
3. Von Quintus, H. L., and B. Killingsworth. *Analyses Relating to Pavement Material Characterizations and Their Effects on Pavement Performance.* Publication No. FHWA-RD-97-085. FHWA, Jan. 1998.
4. Finn, F. N., K. Nair, and C. Monismith. *Minimizing Premature Cracking of Asphalt Concrete Pavements.* NCHRP Report 195. NCHRP, National Research Council, Washington, D.C., June 1973.
5. Von Quintus, H. L., J. A. Scherocman, C. S. Hughes, and T. W. Kennedy. Asphalt-Aggregate Mixture Analysis System: AAMAS. NCHRP Report No. 338. NCHRP, National Research Council, Washington, D.C., March 1991.
6. Rauhut, J. B., R. L. Lytton, and M. I. Darter. Pavement Damage Functions and Load Equivalence Factors. Report No. FHWA/RD-84/018. FHWA, June 1984.
7. Kingham, R. I. *Development of the Asphalt Institute's Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements.* Report 69-3. Asphalt Institute Research, June 1969.
8. Nunn, M. Design of Long-Life Roads for Heavy Traffic. Presented at the Australian Asphalt Pavement Association Industry Conference, Transport Research Laboratory, Crowthorne, U.K., 1998.
9. Lytton, R. L., et al. *Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixes*. Report No. SHRP-A-357. SHRP, National Research Council, Washington D. C, 1993.
10. Von Quintus, H., and B. Killingsworth. Design Pamphlet for the Determination of Layered Elastic Moduli for Flexible Pavement Design in Support of the 1993 AASHTO Guide for the Design of Pavement Structures. Publication No. FHWA-RD-97-077. FHWA, Sept. 1997.
11. Von Quintus, H., and B. Killingsworth. Design Pamphlet for the Determination of Design Subgrade in Support of the 1993 AASHTO Guide for the Design of Pavement Structures. Publication No. FHWA-RD-97-083. FHWA, Sept. 1997.
12. Von Quintus, H., and B. Killingsworth. Design Pamphlet for the Backcalculation of Pavement Layer Moduli in Support of the 1993 AASHTO Guide for the Design of Pavement Structures. Publication No. FHWA-RD-97-076. FHWA, Sept. 1997.
13. Review of Life-Cycle Costing Analysis Procedures. Ministry of Transportation of Ontario, Dec. 1998.
14. Yu, H. T., L. Khazanovich, S. P. Rao, H. Ali, M. I. Darter, and H. Von Quintus. Guidelines for Subsurface Drainage. Draft Final Report. NCHRP, TRB, Washington, D. C., 1998.

Evaluation of Top-Down Cracking in Thick Asphalt Pavements and the Implications for Pavement Design

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Surface-initiated longitudinal wheelpath cracks (or top-down cracks) had been observed in both cores and trench sections removed from asphalt concrete highway pavements. Cracking was documented in both thin and thick pavement sections and the mechanisms for surface crack propagation were explained. An approach was developed using a combination of fracture mechanics and finite element modeling to analyze a cracked pavement and predict the response of the pavement near the crack tip and throughout the depth of the asphalt concrete layer. Prediction of pavement response indicated that the mechanism for crack propagation was primarily tensile and that the influence of pavement structure and load spectra (magnitude and position) is significant. Load positioning and temperature-induced stiffness gradients in the asphalt concrete were shown to have the most effect on crack propagation, along with asphalt and base layer stiffness. The mechanism provided an explanation for crack propagation that confirmed observations of crack growth in the field. Crack growth was divided into stages according to crack length and depending on various structural characteristics. A concept called "time of low crack-growth activity" was formulated that considers the magnitude of tension and crack length as a function of time. Most importantly, the mechanism and details of top-down cracking were described and may be considered before selecting an appropriate perpetual pavement system.

Over the last several years, surface-initiated longitudinal wheelpath cracking has been plaguing Interstate highways as the predominant mode of failure in Florida. A similar problem has been reported in Europe and is apparently more prevalent in other parts of the United States as well. Several different combinations of cracks have been observed, such as single cracks in either wheelpath, both wheelpaths (as seen in Figure 1), between the wheelpaths, and even multiple longitudinal cracks in the wheelpaths. Extensive work has been done by researchers at the University of Florida and the Florida Department of Transportation to identify the causes and solutions to this problem.

In the past, cracking has always been assumed to start at the bottom of asphalt concrete and propagate upward through the layer. Various factors have been evaluated with respect to the response at the bottom of the pavement layer (at a location centered underneath load) and structure was shown to have a strong effect. However, visual inspection of cores and trench sections clearly showed that cracks initiate at the surface of the asphalt concrete and propagation downward in the layer, as seen in Figure 2.

FIGURE 1 Lane exhibiting surface-initiated longitudinal cracks in both wheelpaths.

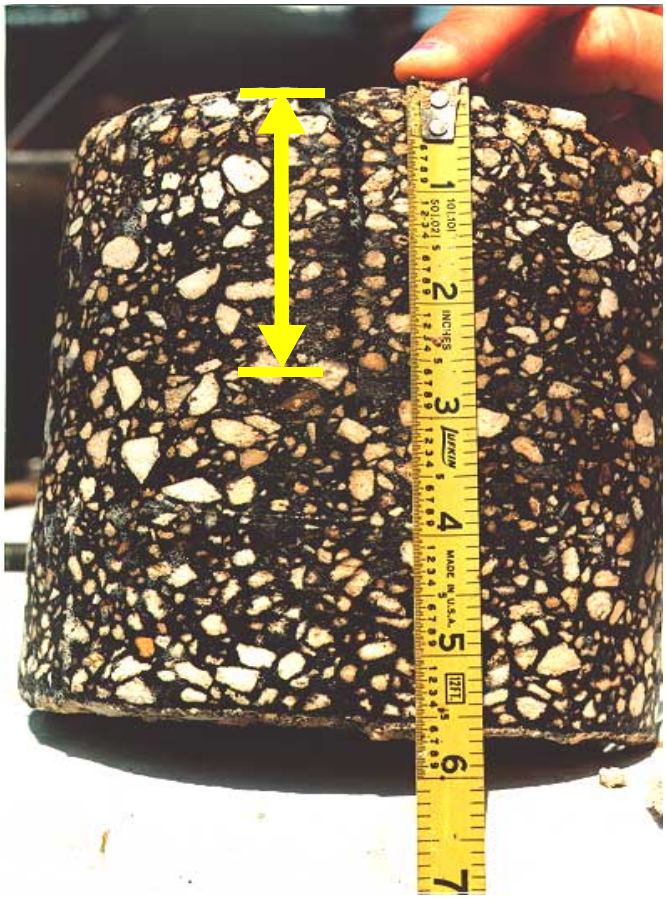

FIGURE 2 Core extracted from wheelpath illustrates crack opening at surface.

Detailed investigation of this distress has been published in other forums and the objective of this paper is to summarize the findings related to top-down cracking. The findings are presented particularly as they relate to perpetual pavements in the following areas:

- Mechanisms for initiation and propagation,
- Key factors dominating the mechanisms, and
- Implications for mitigation and design.

FINDINGS FOR CRACK INITIATION

During the study, particular attention was paid to pavement surface stresses and their input to the distress mechanism. Tire-pavement interface stresses, including lateral stresses, thermal stresses, and induced stiffness gradients, were all analyzed. Tire-pavement interface stresses measured on an instrumented steel-bed device were obtained for a variety of truck tire types and of tire-inflation pressures and loads, and the resulting stresses were applied to the pavement models. Of particular interest were the transverse stress reversals that are induced under the ribs of radial truck tires. Analysis of thermal stresses revealed that a critical condition for high tension exists over a brief period of time, particularly during winter evening hours.

Pavement structure was also evaluated as a contributing factor for surface-initiated longitudinal wheelpath cracking. Both layer stiffness and thickness were varied to values typically found on Interstate highways. The sensitivity analysis showed that the magnitude of surface stresses was greatest in thicker pavements, as a result of decreased bending, under the widest tire rib.

Therefore, the primary contributors to surface-initiated longitudinal wheelpath cracking are transverse contact stresses induced by radial truck tires; thermal stresses may also have some influence. In some cases, cracks may be induced by differences in paving equipment (as suggested by researchers in Illinois DOT).

From preliminary findings that showed that pavement structure (thickness and stiffness) had little effect on surface stresses, it appears that this problem may be addressed through the use of improved asphalt mixtures with higher fracture resistance. In addition, measured tire-pavement interface stresses are instrumental in the proper evaluation of pavement cracking performance and must be considered.

MECHANISMS FOR CRACK PROPAGATION

The potential causes for crack initiation were discovered; however, an explanation for propagation of top-down cracks past the initial stage was needed. One reason was that neither tire contact nor thermal surface stresses could be used to describe the propagation of top-down cracks. A relatively small zone of tension, approximately 1 cm deep, was predicted at the surface of the pavement. Therefore, research work was conducted to define the damage mechanisms and further identify factors that control propagation.

A parametric study was conducted to isolate the effects of various parameters on surface crack propagation. Some of the parameters evaluated include: load spectra (positioning with respect to crack and measured vertical and lateral tire contact stresses), cracks and discontinuities, crack depth, asphalt pavement thickness (Interstate highway pavements are typically thick and stiff), and surface and base-layer stiffness. The range of values for these

factors is illustrated in Figure 3. Stiffness gradients induced in the asphalt concrete layer, due to daily temperature and environmental fluctuations, were also analyzed for their influence on crack growth.

Fracture mechanics was the analytical procedure selected for the prediction of crack growth. Stress intensity factors were computed to define the response behavior at the crack tip. That is, a local description of the crack tip region was provided and the stresses in the process zone (or contours) ahead of the crack tip were predicted. The three parameters used in the evaluation are defined as follows:

Mode I Stress Intensity Factor, $K_I \approx \lim \sigma_{xx} * (2\pi r)^{1/2}$

Mode II Stress Intensity Factor, $K_{II} \approx \lim \tau_{yx} * (2\pi r)^{1/2}$

Fracture Energy Release Rate, $J = (K_I^2 + K_{II}^2) * (1 - \nu^2)/E$

where

σ_{xx} = transverse stress
τ_{yx} = shear stress
ν = Poisson's ratio
r = vertical distance away from crack tip
E = stiffness modulus

Results of the parametric study indicated the effects of pavement structure. Tension was found to be the overwhelming contributor to failure and greatly exceeded the magnitude of shear at the crack tip, indicating that crack growth is primarily through Mode I opening. Figure 4 illustrates the difference in magnitude between the tension and shear stress intensity factors. The figure also indicates that a higher stiffness ratio (asphalt to base-layer stiffness) increases tension at the crack tip. The most critical factor for tension at the crack tip was found to be load position. The magnitude of tension then depended on the crack length, as shown in Figure 5.

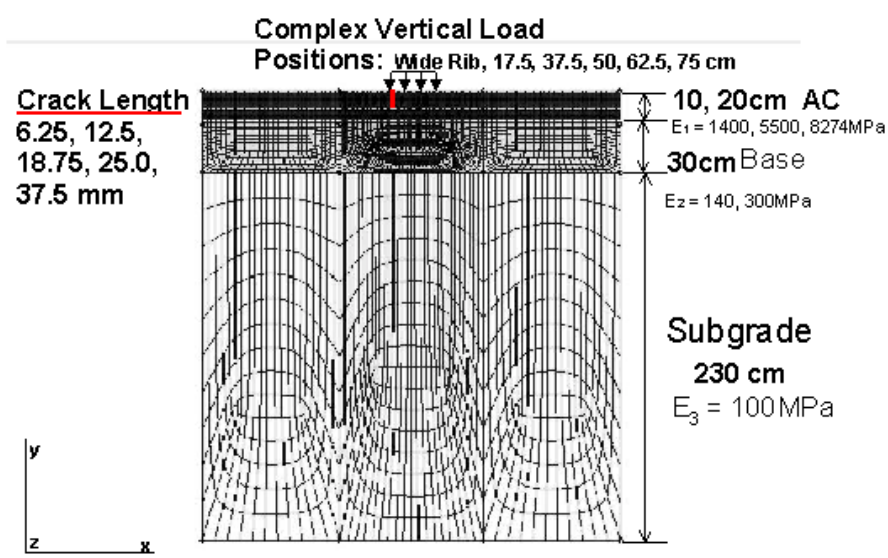

FIGURE 3 Pavement model showing parameters evaluated for identification of factors critical to propagation of surface cracking (AC = asphalt concrete, *E* = stiffness modulus).

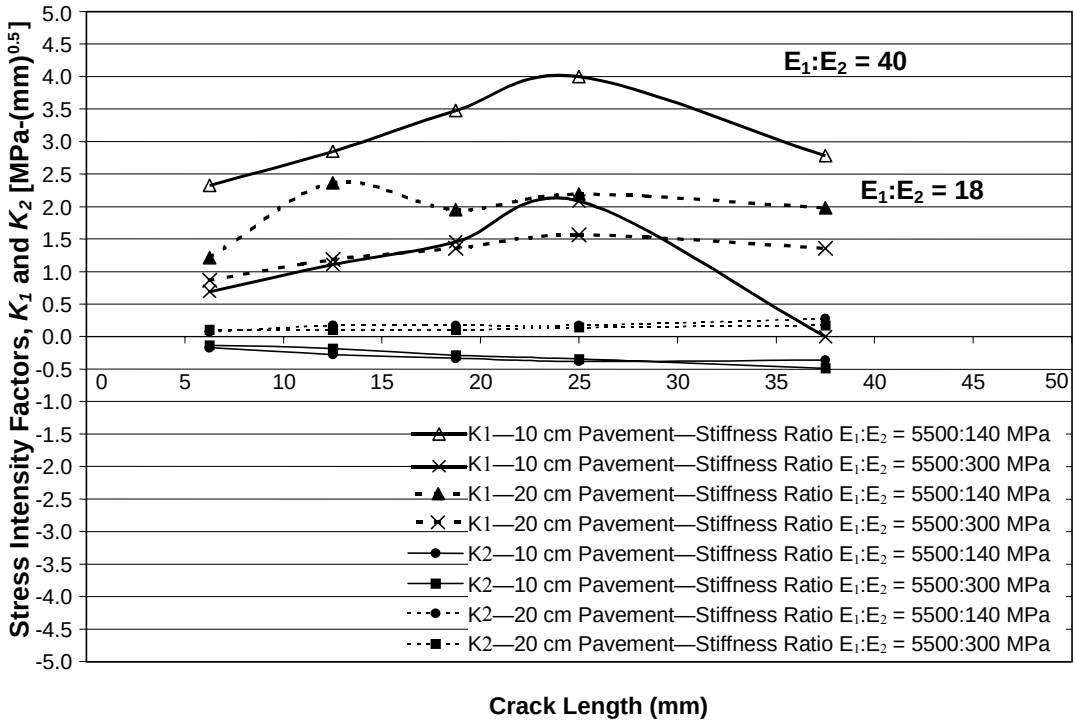

FIGURE 4 Comparison of failure modes: magnitude of tensile fracture (K_I) versus shear fracture (K_{II}) for one loading case (62.5 cm from crack).

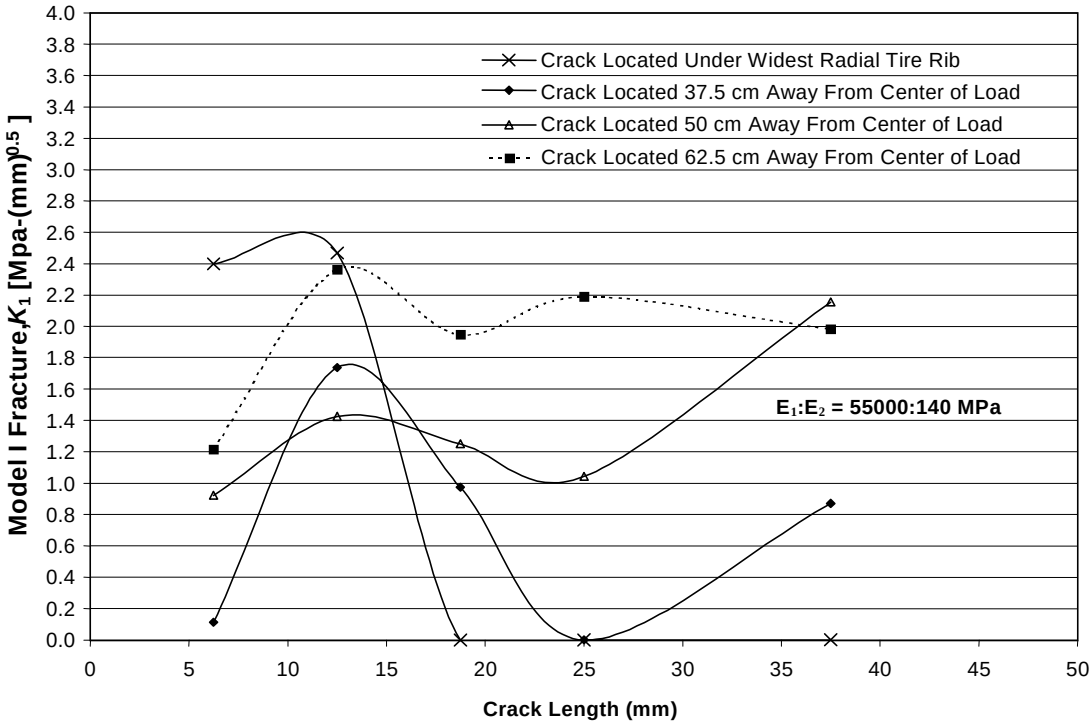

FIGURE 5 Effect of load positioning on opening at the crack tip for 20-cm surface layer with a high stiffness ratio.

The parametric study also included identifying the influence of stiffness gradients on crack growth. The stiffness gradients selected for this study were based on measurements taken from a FHWA study in northcentral Florida. Temperature differentials, age-hardening, and sudden rains all contribute to the inducing of sublayers of variable stiffness within asphalt concrete. The four gradient configurations determined for the evaluation are defined as follows:

- Case 1: Uniform temperature distribution (i.e., no stiffness gradient) mean pavement temperature computed at 1/3-depth when temperature conditions are warm.
- Case 2: Sharpest temperature gradient near surface—temperatures at 7 p.m. represent this condition.
- Case 3: Highest temperature differential between surface and bottom of asphalt concrete layer—temperatures at 5 a.m. represent this condition.
- Case 4: Rapid cooling near the surface represents case of sudden rain showers.

Finite element analysis of the pavement models indicated a major increase in tension at the crack tip. Figure 6 illustrates the tensile stress intensity at the crack tip at each stiffness gradient combination for a given pavement structure and load position. Observation of the figure shows that tension predicted in the stiffness gradient cases was sometimes as much as seven times greater in magnitude than in the case of a pavement with uniform stiffness. However, an asphalt concrete of uniform stiffness would never truly predict maximum crack growth. For this reason, analyzing the pavement as a layer of variable stiffness is a key factor in the prediction of surface crack propagation.

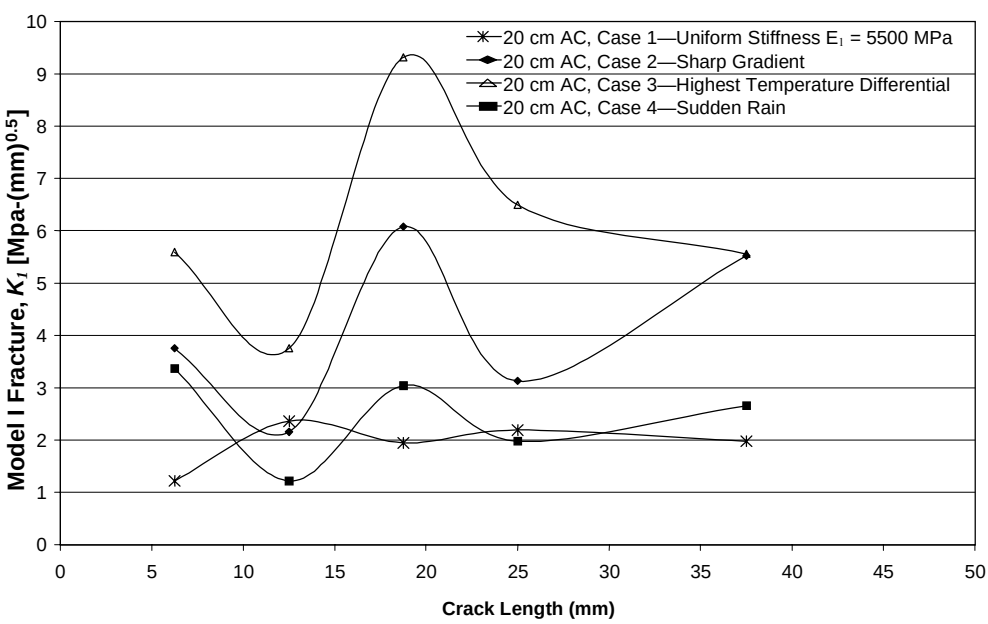

FIGURE 6 Effect of temperature-induced stiffness gradients on stress intensity K_1 predicted at crack tip for 20-cm AC (load centered 75 cm from crack; base stiffness E_2 = 140 MPa; AC = asphalt concrete).

STAGES OF CRACK GROWTH

Interpretation of the findings from the parametric study led to defining of stages of crack growth. The two stages of crack growth for this study were defined as short cracks (6.25- to 12.5-mm depth) and intermediate cracks (18.75- to 37.5-mm depth). Evaluating the problem from this perspective helped to isolate the implications of various load spectra. Analyses indicated that both load wander and magnitude are instrumental in the cracking mechanism. For example, further observation of Figure 5 reveals that the critical load position is not always directly in the wheelpath on top of the crack. In fact, the location of the critical load position will depend on the crack length, the pavement structural characteristics, and even the type of stiffness gradient induced. For this reason, a need exists to determine how many loads actually induce tension at the crack tip.

It was also observed that the magnitude of tension varied depending on crack length, as a function of loading position and the other factors as seen in Figure 7. From extensive observation of all data, a conceptual idea was formulated that would define a "time of low crack-growth activity." Within this concept, cracks would be allowed to develop to a certain intermediate length. A time of "low crack-growth activity" would be defined as the time period where the stress intensities are relatively low and cracks are not inclined to propagate. Then, at the first indications of increased cracking, pavement rehabilitation could be applied before the crack exceeded the predetermined intermediate length. It is critical to rehabilitate the pavement early in the cracking process, depending on the time available before the crack rate speeds up.

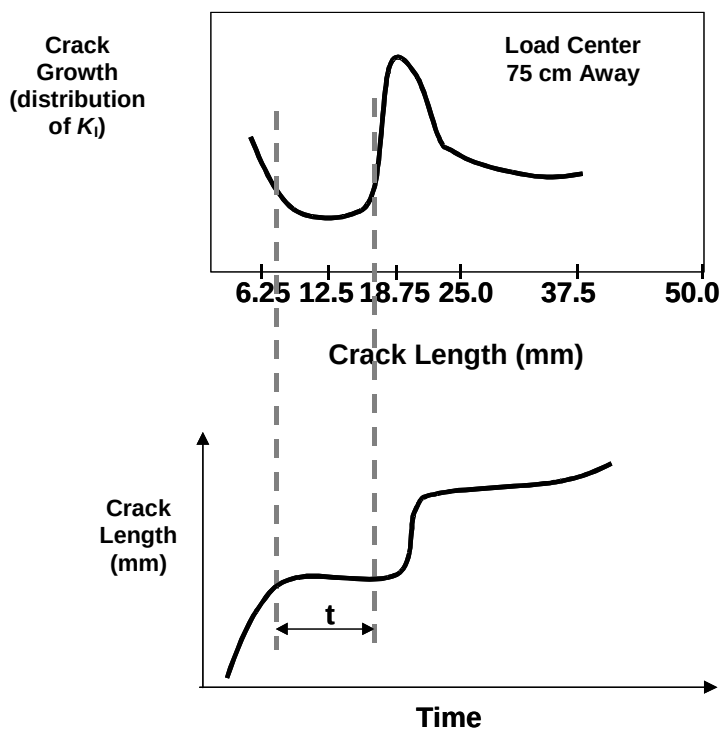

FIGURE 7 Potential for crack growth and time available for identification and rehabilitation for 20-cm pavement and given load spectrum. (*t* = time available for identification and rehabilitation.)

Analyses indicated that for given load spectra, the cracking potential varies with crack length and apparently slows at intermediate crack lengths. The distribution of K_1 can then be translated to a crack growth potential as the crack length increases, as shown in Figures 7 and 8. Knowing the crack growth potential may then lead to the development of a pavement management approach defining a time period in which to perform rehabilitation. The conceptual "Time of Crack Growth Activity" can be applied to individual field sections depending on recorded load wander, pavement structure, and seasonal temperatures.

IMPLICATIONS FOR DESIGN

Comprehensive analyses showed that top-down cracks develop even in thick pavements that may not be susceptible to other forms of damage. In addition, the mechanisms for top-down cracking cannot be captured without considering critical factors such as realistic contact stresses, temperature-induced stiffness gradients, load spectra (magnitude and wander), and the presence

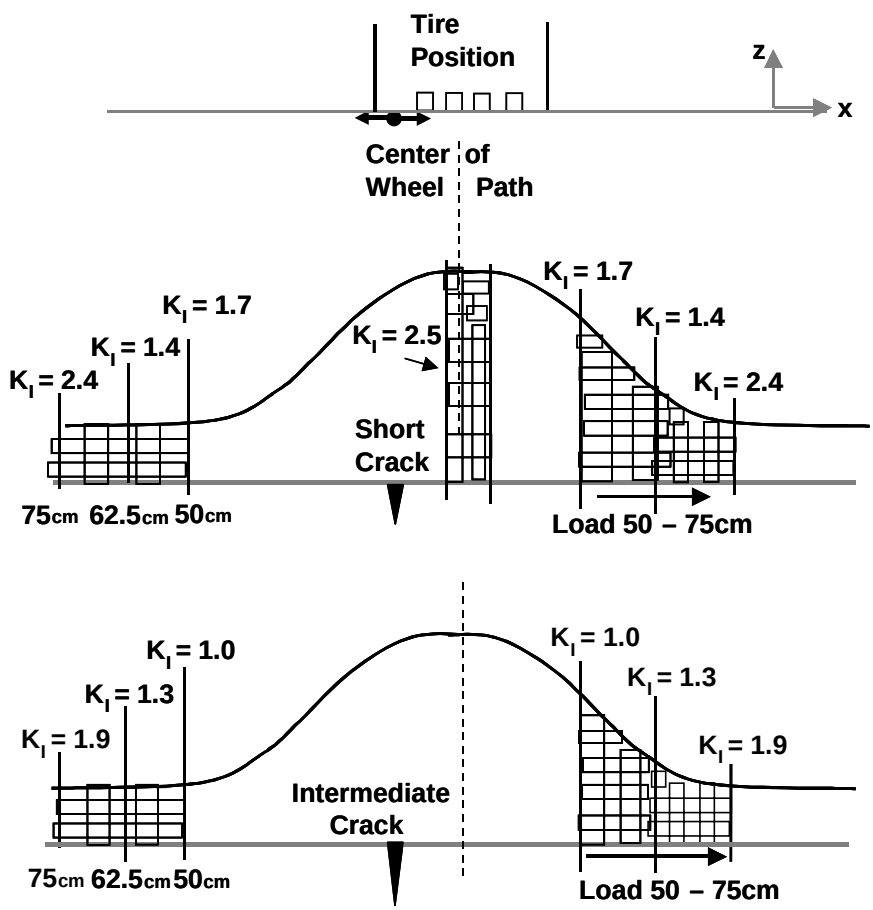

FIGURE 8 Transverse distribution of load within wheelpath and critical load positions for cracking in a 20-cm asphalt concrete layer of uniform stiffness (E_1). [AC = asphalt concrete; fracture stress intensity (K_1) in MPa $(mm)^{0.5}$]

of cracks and discontinuities as described through use of fracture mechanics. Consideration of these factors would indicate that employing a design approach using averaged pavement and load conditions is inadequate and will not predict cracking since critical conditions must be identified. It appears that fracture mechanics is then necessary for predicting the cracking mechanism through its description of stress redistributions at the crack tip and its identification of local effects. Likewise, stiffness gradients were shown to significantly intensify the cracking mechanism, which implies that analyzing with a uniform layer stiffness would be unrealistic and would not capture the true magnitude of tension generated in the pavement.

Evaluation of all data suggests that mitigation of top-down cracking should be addressed primarily through the implementation of more crack-resistant surface materials and use of less damaging truck tires. Thus, a focus for prevention should concentrate on improving surface materials, communication with tire designers, and pavement design-management techniques. Since cracking potential was shown to increase with asphalt concrete thickness, the use of multiple overlays may actually exacerbate crack growth, indicating that mill-replace techniques may be a better alternative for mitigation of top-down cracks.

CONCLUSIONS

Based on the findings of the comprehensive analytical and field study, propagation of surface-initiated longitudinal wheelpath cracks is a Mode I tensile failure mechanism and occurs only under critical conditions. Finite element analysis of a cracked pavement was conducted to define the critical design conditions at which crack growth will occur, as a function of several pavement structural factors that were varied to determine their individual effects on crack propagation. Results indicated that it is necessary to model cracks and discontinuities and predict stress redistributions at the crack tip, as well as compute the direction of crack growth. The analyses showed that crack growth must be analyzed using realistic load spectra to adequately predict pavement performance since load positioning was found to be the overriding contributor to crack growth. In particular, to predict failure and to determine future design conditions, load wander must be considered along with the use of measured tire contact stresses.

It may also be concluded that additional factors must be included in a future approach to pavement design, such as crack length and temperature-induced stiffness gradients in asphalt concrete, because of their significant effect on the tensile response of surface cracks. Therefore, a sensitivity analysis should be developed for determining the rate of crack growth relative to time. Once the crack-growth rate relative to time is a defined parameter, it may be adopted as a pavement management strategy. It is suggested that the period of time when the crack rate slows down should be defined; however, additional field and mixture information is needed to define this length of time in order to establish guidelines for pavement management.

ACKNOWLEDGMENTS

The authors would like to acknowledge the support of the Florida DOT and FHWA in the on-going study of top-down longitudinal wheelpath cracking.

Study of Long-Lasting Pavements in Washington State

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A combination of three studies has allowed an examination of long-lasting pavements in Washington State and specifically those pavements on the Washington State Department of Transportation (WSDOT) route system. The findings of these studies are summarized in this paper. The major findings support the view that thick asphalt concrete pavements (greater than 160 mm) should have long lives. The performance of WSDOT wearing courses, though good, can be improved. Specific focus to improve performance should be on construction practices. Generally, these results support the view that "mill and fill" surface rehabilitation is very effective. Finally, the current in-service WSDOT thick asphalt concrete pavements come close to meeting the goals associated with perpetual pavements.

A combination of three studies has allowed an examination of long-lasting pavements in Washington State and specifically those pavements on the Washington State Department of Transportation (WSDOT) route system. These studies examined pavements with superior and inferior performance on the WSDOT route system, studied surface initiated-cracking of asphalt concrete (AC) wearing courses, and reviewed pavement management system data for Interstate 90 (I-90). Each of these studies will be reviewed with specific emphasis on I-90 performance results.

SUPERIOR-PERFORMANCE PAVEMENTS

The first study assessed factors that influence superior and inferior performing pavements (*1*, *2*). All WSDOT pavements were sorted by surface type [portland cement concrete (PCC), AC, and bituminous surface treatments], highway type (Interstate or non-Interstate), and construction (new versus rehabilitated or resurfaced).

Based on measures of cracking, the International Roughness Index (IRI), and rutting, the following observations were made:

- For superior-performance pavements, the dominant surface distress was longitudinal cracking.
- For inferior-performance pavements, the dominant surface distress was fatigue cracking.
- Typically, inferior-performance pavements were thicker than superior performing sections.
- A comparison of superior- and inferior-performance sections revealed:
  - Inferior sections were about one-half the age of the superior sections.
  - The area of cracking for the inferior sections was three times greater.
  - The inferior sections had experienced one-half the equivalent single-axle loads (ESALs) of the superior sections.
  - The IRI was 14 percent higher for the inferior sections—not a large difference.

– Rutting for the inferior sections was 40 percent higher than for the superior sections.

The broad conclusion drawn from that study is that well-constructed flexible pavements can perform adequately for long periods of time. Construction related effects (including mix variations) are the likely cause of most inferior performance in pavement sections. Further, WSDOT thickness design practices appear to be adequate and are working well for the design of both AC and PCC pavements.

Lastly, superior-performance Interstate pavements carry the highest ESALs in Washington State (not a surprise)—up to 2.5 million ESALs per year in the design lane.

SURFACE-INITIATED CRACKING

For some time, longitudinal cracking has been commonly observed on WSDOT pavements. Research performed in the United States and several other countries revealed a substantial amount of such cracks initiated at the pavement surface. Acting on this information, WSDOT undertook an extensive coring program in Eastern Washington. The coring was done in conjunction with traditional site investigations for upcoming rehabilitation projects. Further, correspondence with personnel in the Minnesota Department of Transportation revealed the occurrence of surface initiated cracking at the Minnesota Road Test Facility. The Minnesota Road Test Facility sections have been in service since 1994. Cores revealed that the initial longitudinal cracks started at (or near) the pavement surface.

The WSDOT work included not only cores but also falling-weight deflectometer testing (*3*). The bottom line was that most flexible pavements with AC thicknesses greater than 160 mm exhibited only surface-initiated cracking (if any cracking was present). This is an important finding in that it appears (based on that study and others) that there are minimum AC thicknesses that “resist” the formation of fatigue cracking. Knowing this will aid the systematic design of long-lasting pavements.

Prior studies and currently available finite element modeling suggest tensile strains exist at the pavement surface that are sufficient to aid the development of surface-initiated cracking. Asphalt binder aging is the other most likely contributor to such cracking. Surface-initiated longitudinal cracks can be confined to the wearing course; hence, “mill and fill” should aid long AC structural section performance.

INTERSTATE 90

The field performance data for I-90 were reviewed with the 1999 version of the Washington State Pavement Management System (WSPMS). The purpose was to examine all pavement segments on the 480 km of I-90 within Washington State. Specifically, the pavement segments fit into three categories (based on original construction): flexible, cement-treated base (CTB) with AC wearing course, and PCC pavements. Various statistics were generated and are shown in Tables 1 through 6. The percentages of each of the three pavement types on I-90 are flexible (47 percent), CTB/AC (33 percent), and PCC (20 percent).

An examination of the complete length of I-90 within Washington State spans two very different climate zones caused in part by the Cascade Mountain range and, to some extent, traffic levels. Western Washington has a mild marine dominated climate with high ESAL levels. Eastern Washington (east of the Cascade Mountains) has cold winters and warm to hot summers. Portions of I-90 have low ESAL levels (375,000 ESALs/year in the design lane) and other

portions very high (2.5 million/year in the design lane). Data from the 1999 WSPMS were grouped into uniform segments defined as pavement structures with the same structural design, constructed at the same time, and in the same vicinity. Individual pavement sections within a uniform segment could have different performance as measured by rutting, cracking, IRI, and other factors. The individual sections ranged in length from 0.02 to 17.0 km, and the uniform segments ranged from 0.15 to 24.4 km. A limited number of sections were eliminated because of questionable data in the WSPMS. Only data in the eastbound direction were used. The assumption was that the westbound data would be essentially the same.

Flexible Pavements

Tables 1 and 2 provide an overview of flexible pavement performance on Interstate 90. Table 1 shows that, on average (average weighted by segment length), the time since original construction for flexible segments ranged from about 26 years for Western Washington to 29 years for Eastern Washington. The original thickness of AC was 370 mm in Western Washington and 240 mm in Eastern Washington. The time from original construction to the first resurfacing ranged from 18.5 years in Western Washington to 12.4 years in Eastern Washington. For Eastern Washington the times to the first resurfacing ranged from 6 to 21 years. A range of such width is significant and suggests that something other than traditional pavement performance factors, such as thickness and traffic, may be influencing performance—at least for the underperforming segment. An inspection of the WSPMS data suggests that the most likely cause is stage construction.

TABLE 1 Summary of Performance of I-90 Flexible Pavements

Location	Time Since Original Construction (years)	Thickness of Original AC (in.)	Time from Original Construction to First Resurfacing (years)	Age of Current Wearing Course (years)	Current IRI (m/km)	Current Rut Depth (mm)
Western Washington						
Weighted Average	25.8	14.5	18.5	7.4	1.0	5
n	9	9	9	9	9	9
Range	23–29	13.8–18.6	17–22	4–12	0.7–1.3	2–7
Eastern Washington						
Weighted Average	29.3	9.5	12.4	4.7	0.8	5
n	27	27	25	25	25	25
Range	6–35	6.0–13.9	6–21	2–10	0.6–1.2	1–9

NOTES: Weighted Average = values weighted by length of individual uniform segments; *n* = number of uniform segments; Range = smallest to largest values; 1 in. = 25.4 mm.

TABLE 2 Summary of Resurfacings for I-90 Flexible Pavements

Location	Percent of Segments Resurfaced				Wearing Course Life (years)	
	First Resurf	Second Resurf	Third Resurf	Fourth Resurf	Original AC	First Resurf
Western Washington (total no. of segments = 9)	100	0	0	0	Mean = 18.5 *n* = 9 Range 17–22	N/A
Eastern Washington (total no. of segments = 27)	93	85	11	0	Mean = 12.4 *n* = 25 Range 6–21	Mean = 12.2 *n* = 21 Range 8–17

NOTE: Mean = weighted average (weighted by length of individual uniform segments); *n* = number of uniform segments; Range = smallest to largest values.

For the current in-service wearing courses with ages of about 7 years (Western Washington) to 5 years (Eastern Washington), the IRI mean value fit into the "good" category as defined by Long-Term Pavement Performance (LTPP) (IRI <1.4 to <1.5, depending on age). (The LTPP rating categories are good, average, or poor.) The ranges of segment IRIs for Western Washington (0.7 to 1.3 m/km) and Eastern Washington (0.6 to 1.2 m/km) all fit within the LTPP "good" category. For rutting, the mean value was at the boundary of the "good-average" categories (5 mm). Thus, in general, the performance of this pavement type was good as defined by criteria developed by LTPP. Furthermore, none of the originally constructed flexible pavement structures on I-90 have been reconstructed to date.

Table 2 summarizes the percentage of flexible pavement segments on Interstate 90 that have been resurfaced (AC overlays). In Western Washington all of the segments have been resurfaced once since original construction; however, none have been resurfaced twice. Eastern Washington is different. Most of the segments have been resurfaced twice since original construction. Additionally, the data reveal that the first resurfacing (first AC overlay) has served about as long as the original wearing course (12.4 versus 12.2 years, respectively). This implies that the basic pavement structure has survived well, since virtually all of the overlays have rarely exceeded 45 mm. Furthermore, many of the AC overlays have been "mill and fill," and thus there was no net gain in pavement thickness.

CTB Pavements

Tables 3 and 4 overview the pavement performance of CTB with AC wearing course on Interstate 90. Table 3 shows that, on average, the time since original construction for these segments was about 38 years for Eastern Washington. No CTB pavements were built on Interstate 90 in Western Washington. The original thickness of AC and CTB (combined) was 230 mm—about the same as that of flexible pavements for that part of the state (240 mm). The mean time from original construction to the first resurfacing was 13 percent less than that for flexible pavements at 10.8 years. The range of times to the first resurfacing ranged from 3 to 16 years.

The current in-service wearing courses had a mean age of about seven years. The associated IRI mean value of these segments fit into the "good" category as defined by LTPP. The range of

TABLE 3 Summary of Performance of I-90 CTB Pavements

Location	Time Since Original Construction (years)	Thickness of Original AC and CTB (in.)	Time from Original Construction to First Resurfacing (years)	Age of Current Wearing Course (years)	Current IRI (m/km)	Current Rut Depth (mm)
Western Washington	N/A	N/A	N/A	N/A	N/A	N/A
Eastern Washington						
Weighted Average	38.2	9.1	10.8	7.1	0.9	7
n	21	21	21	21	21	21
Range	33–42	8–10	3–16	1–10	0.6–1.2	1–11

NOTES: N/A = There are no segments of CTB construction on I-90 in Western Washington; Weighted Average = values weighted by length of individual uniform segments; *n* = number of uniform segments; Range = smallest to largest values.

TABLE 4 Summary of Resurfacings for I-90 CTB Pavements

Location	Percent of Segments Resurfaced				Wearing Course Life (years)		
	First Resurf	Second Resurf	Third Resurf	Fourth Resurf	Original AC	First Resurf	Second Resurf
Western Washington	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Eastern Washington (total no. of segments = 21)	100	100	71	10	Mean = 10.8 *n* = 21 Range 3–16	Mean = 11.9 *n* = 21 Range 7–21	Mean = 11.5 *n* = 15 Range 8–14

NOTES: Mean = weighted average (weighted by length of individual uniform segments); N/A = no CTB segments on I-90 in Western Washington; *n* = number of uniform segments; Range = smallest to largest values.

segment IRIs (0.6 to 1.2 m/km) all fit within the LTPP "good" category (IRI<1.5 m/km). For rutting, the mean value and range both fit into the LTPP "average" category (5 to 11 mm). It is important to note that most of the pavement reconstruction done on Interstate 90 to date has involved this pavement type.

Table 4 summarizes the percentage of the originally constructed CTB pavement segments on I-90 that have been resurfaced. Most of the segments have been resurfaced three times since original construction; however, these segments are, on average, the oldest on I-90. Additionally, the first and second resurfacings have served longer than the original wearing course (10.8 versus 11.9 and 11.5 years). For this pavement type, there were wider ranges of resurfacing treatments and thicknesses. A number of the resurfacings involved granular overlays (crushed stone base material plus AC wearing course) placed directly on the original pavement structure. Furthermore, many of the AC overlays were thicker (75 to 105 mm) than the traditional 45 mm

thickness. This is not unexpected because most of the CTB was constructed before the completion of Interstate 90 and the overlays represented needed structural upgrading.

PCC Pavements

Tables 5 and 6 provide an overview of PCC pavement performance on I-90. Table 5 shows that, on average, the time since original construction for uniform segments was about 20 years for Western Washington and 31 years for Eastern Washington. The original mean thickness of the PCC slabs was 230 mm. The mean time from original construction to the first resurfacing was 19 years for Western Washington and 18 years for Eastern Washington. The range of times to the first resurfacing was large. All of the PCC slabs in Western Washington were placed on asphalt treated base. In Eastern Washington, the base type was typically crushed stone.

The current pavement surfaces have mean ages of about 18 years for Western Washington and 12 years for Eastern Washington. The associated IRI mean values of these segments fit into the "good" LTPP category (<1.7 m/km for an age of 18 years) for Western Washington and "average" for Eastern Washington (1.3-2.4 m/km for an age of 12 years). The current mean wheelpath wear depths for Western and Eastern Washington were 2 and 3 mm, respectively (note that Washington allows the use of studded tires during the winter months).

Table 6 summarizes the percentage of the originally constructed PCC pavement segments on

TABLE 5 Summary of Performance of I-90 Rigid Pavements

Location	Time Since Original Construction (years)	Thickness of Original PCC (inches)	Time from Original Construction to First Rehabilitation (years)	Age of Current Wearing Course (years)	IRI (m/km)	Rut Depth (mm)
Western Washington						
Weighted Average	20.0	9.3	19.0	18.3	1.5	2
n	19	19	3	19	19	19
Range	5–49	8–12	13–28	4–24	0.9–1.9	1–6
Eastern Washington						
Weighted Average	30.6	9.1	29.5	11.6	1.6	3
n	21	21	12	21	21	21
Range	4–42	8–11	18–42	0–42	1.1–2.4	1–4

NOTES: Weighted Average = values weighted by length of individual uniform segments; *n* = number of uniform segments; Range = smallest to largest values.

TABLE 6 Summary of Resurfacings for I-90 Rigid Pavements

Location	Percent of Segments Resurfaced				Wearing Course Life (years)		
	First Resurf	Second Resurf	Third Resurf	Fourth Resurf	Original PCC (only segments that have been rehabilitated)	First Resurf	Second Resurf
Western Washington (total no. of segments = 19)	16	5	0	0	Mean = 19.0 *n* = 3 Range 13–28	N/A	N/A
Eastern Washington (total no. of segments = 21)	57	0	0	0	Mean = 29.5 *n* = 12 Range 18–42	N/A	N/A

NOTES: Mean = weighted average (weighted by length of individual uniform segment); *n* = number of uniform segments; Range = smallest to largest values.

I-90 that have been "resurfaced." Only 16 percent of the Western Washington slabs have been resurfaced, whereas 57 percent have been resurfaced in Eastern Washington. Resurfacing is generally defined as retrofitted dowel bars followed by grinding or an AC overlay (typically 90 mm thick). The original PCC slabs that had been resurfaced survived about 19 years in Western Washington and 30 years in Eastern Washington.

IMPLICATIONS FOR WSDOT

The broad goals of the perpetual pavement concept is to achieve:

- Wearing course life of 20 years,
- Structural section with a life of 40 or more years,
- Primary surface rehabilitation through mill and fill, and
- Pavement distress within only the top few centimeters of the surface.

The current in-service WSDOT thick AC pavements (as illustrated by the results from Interstate 90) come close to meeting the goals associated with Perpetual Pavements.

Some of the specific implications of the performance assessment for WSDOT are summarized as follows:

- Pavement Design Period: WSDOT uses the AASHTO *1993 Pavement Guide* for the design of new or reconstructed pavements and a design period of 40 years for both flexible and rigid pavements. Based on this information from Interstate 90, the structural sections for flexible and rigid pavements are all intact (no significant reconstruction to date) with most of the segments approaching 30 years of service. This is supportive of the WSDOT design life assumption.

- Life-Cycle Cost Analyses (LCCA): The I-90 data generally support the following assumptions that WSDOT uses in its LCCA:
  - Analysis period of 40 years,
  - AC resurfacing following 10 to 15 years of service, and
  - Grinding of PCC slabs following 20 years of service to restore smoothness.
- CTB pavements: CTB/AC pavements have not been constructed on the WSDOT system since the 1960s. The I-90 data generally support that decision made long ago.
- Overall performance: The WSDOT pavements, as represented by those on Interstate 90, generally fall into the LTPP "good" performance category (recall the other possibilities being "average" and "poor"). The IRIs of the current AC wearing courses all fall into the "good" category. The PCC slabs are rougher than the AC-surfaced pavements but have been in service more than twice as long.

REFERENCES

1. Baker, M. J., J. P. Mahoney, and N. Sivaneswaran. Engineering Application of Washington State's Pavement Management System. In *Transportation Research Record 1643*, TRB, National Research Council, Washington, D.C., 1998, pp. 25–33.
2. Baker, M. J., and J. P. Mahoney. *Identification and Assessment of Washington State Pavements with Superior and Inferior Performance*. WSDOT Research Report WA-RD 437.1. Washington State Department of Transportation, Olympia, 2000.
3. Uhlmeyer, J. S., K. Willoughby, L. M. Pierce, and J. P. Mahoney. Top-Down Cracking in Washington State Asphalt Concrete Wearing Courses. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1730*, TRB, National Research Council, Washington, D.C., 2000, pp. 110–116.

Development of Long-Life Overlays for Existing Pavement Infrastructure Projects with Surface Cracking in New Jersey

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Existing pavements represent significant capital expenditure on infrastructure. Appropriate maintenance insures that the existing pavement infrastructure is maintained in the most cost-effective manner. Pavements in New Jersey have been the subject of extensive evaluation to determine the best treatment options while making maximum use of the existing structure.

The interstate I-287 in Morris County, New Jersey was showing considerable signs of distress in the early 1990s. The rehabilitation of this highway followed an evaluation of the structure (*1*) that revealed both a hardened binder in the existing pavement layers and surface cracking. The degree of surface cracking was evaluated by extensive coring and materials testing and then confirmed through falling-weight deflectometer testing of all pavement travel lanes.

The existing pavement materials provided an excellent stiff-base structure utilizing the existing pavement infrastructure. The surface-cracked layers were removed to a depth consistent with the crack depth and replaced with a polymer modified asphaltic overlay. The pavement is currently performing in an excellent manner and is expected to give many years of surface before maintenance is required to restore riding quality.

The Interstate route 287 (I-287) at Morristown, New Jersey, is a north–south highway that provides a western bypass around New York City and local traffic movements through New Jersey along a very busy highway corridor. The road also intersects with three major east–west highways at this location: I-80, Route10, and Route 24. Due to the poor condition of the pavement and to the opening of I-287 north of I-80, linking to the New York Thruway (with increasing traffic volumes), this route became a priority for maintenance. The road was initially constructed in 1968 and was considered for rehabilitation design in 1993. The roadway geometry in the highway corridor in the Morristown vicinity was restricted in width due to numerous structures and the presence of the town of Morristown. The road typically carries 24,000 trucks/day (2 way) and the limits of the project were from Milepost 35.5 to 38.8 (Figure 1).

The reconstruction project included the construction of high occupancy vehicles (HOV)

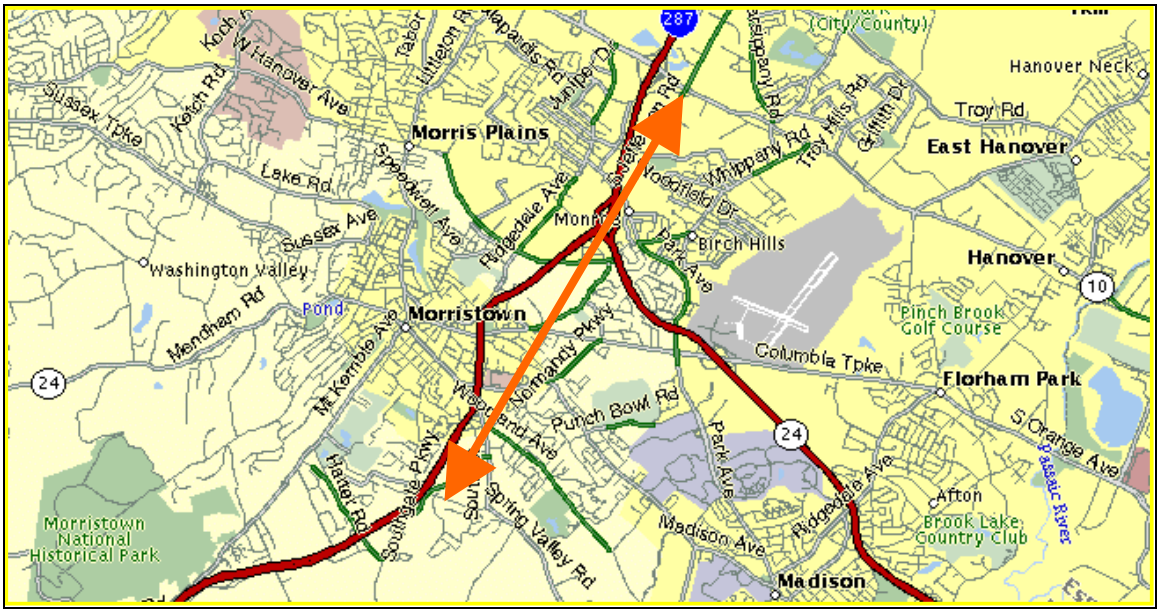

FIGURE 1 Project location.

lanes in both directions to increase capacity and coincide with the opening of I-287 to the north. In addition, it was a requirement that rehabilitation of the pavement structure must have minimal impact on roadway profile, provide a durable wearing surface, and maintain the existing number of travel lanes in peak traffic hours during the construction.

CONSTRUCTION AND DESIGN DATA

The original pavement section was of asphaltic materials that consisted of layers as follows:

- 3-in. hot-mix asphalt (HMA) surface course;
- 7-in. HMA base course (stone mix);
- 8-in. crushed stone base (dense graded);
- 10-in. subbase (graded sand); and
- Subgrade soil (silty sand).

No rehabilitation of the pavement was performed between construction and 1993 (26 years) with the exception of small patches in local areas required to maintain a safe running surface.

The traffic design data used for the pavement rehabilitation was as follows:

- 1993 average daily traffic$_2$ (ADT) = 110,190;
- 2013 ADT$_2$ = 170,830;
- 22 percent total trucks, 9 percent heavy trucks; and
- 20-year equivalent single-axle loads (ESALs) = 50,000,000.

Due to the heavy traffic volumes in this area, there is often the occurrence of slow or standing loads.

The depth of frost penetration at this location is estimated to be 36 in. during the winter months.

The project volumes for HMA were approximately 75,000 tons HMA surface mix and 225,000 tons HMA base mix.

PAVEMENT TESTING AND EVALUATION

A visual condition survey of the pavement was conducted during the design phase, concluding that all lanes had cracking, apparently typical bottom-up fatigue cracking (Table 1). The cracking was more evident in Lanes 2 and 3, which carry the higher truck volumes. The typical surface appearance at that time is illustrated in Figure 2.

TABLE 1 Results of Visual Condition Survey

Lane 1 (Fast Lane)	Lanes 2 and 3 (Middle and Slow Lane)
Low-severity cracking Rutting <1 in.	Moderate- to high-severity fatigue cracking Wheelpath longitudinal cracking Some high-severity patching Rutting > 1 in.

FIGURE 2 Typical surface appearance, 1993.

The initial indications were that cracks penetrated through all bound layers. Subsequently, a pavement coring survey was performed, which indicated that the cracks originated at surface with the majority stopping at the base layer (3-in. depth) (Figure 3). No cracks penetrated through entire layer. This was in contrast to the traditional concept used to design and evaluate pavements, the assumption that cracks start at the bottom of the pavement structure.

As a consequence of this finding it was decided to perform additional testing to determine the likely cause of the pavement distress and to evaluate the pavement for overlay design. The additional work consisted both of materials testing and pavement testing with the falling weight deflectometer (FWD).

Materials Testing

A total of 25 pavement cores were taken from the pavement structure. For each of these cores the gradation, binder content and properties were determined. The binder contents and grading indicated that the mixtures were consistent with the job mix formula for the original asphalt pavement. However, the binder properties, as measured by penetration and viscosity, indicated that significant hardening had taken place, particularly in the surface layers. The mean binder results (penetration test) and void content are illustrated versus their approximate position in the pavement structure in Figure 4.

The data from all the testing appeared to suggest that the degree of hardening in the binder (viscosity data) is related to the void content of the mixture with a regression coefficient r^2 of 0.6618. However, it should be noted that this data set is fairly limited with only five test results (Figure 5).

The data would suggest that the binder in the base has hardened partially due to the higher void content of these materials. The harder binder in the surface course is probably related to the exposure of the binder to oxidative aging due to its location at the surface.

REC. PEN	VOIDS
16	5.5%
40	4.9%
22	6.8%
25	9.3%

FIGURE 3 Example of surface cracking (REC. PEN = recovered penetration in units of mm/10, ASTM D5-97).

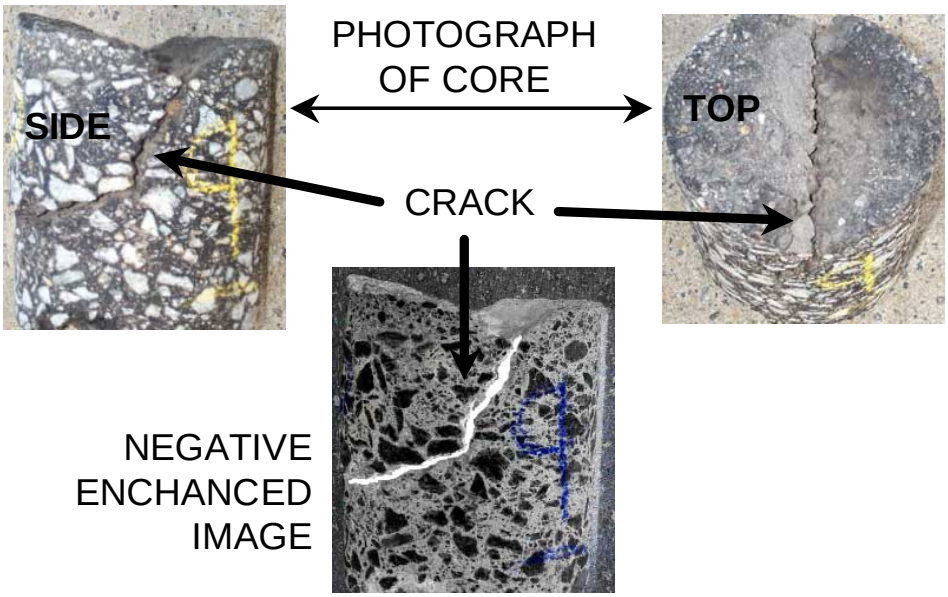

FIGURE 4 Penetration and air voids shown versus approximate depth of core

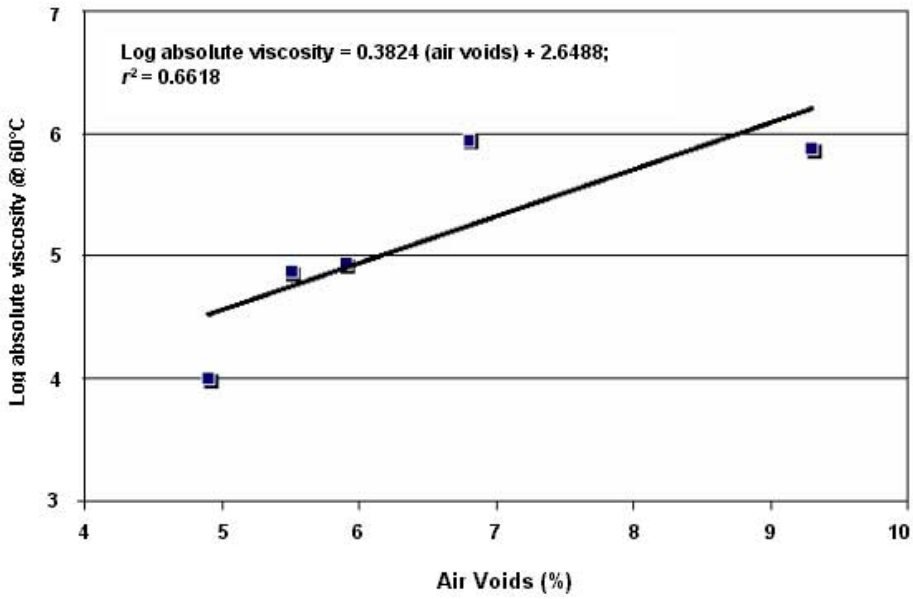

FIGURE 5 Log absolute viscosity versus air voids.

Falling-Weight Deflectometer Testing

The pavement was tested using a Dynatest FWD in the fall of 1993. The testing was conducted in all wheelpaths for the entire length of the project. After testing, the deflection data was analyzed (*2*) to determine the stiffness of the various pavement layers. The stiffness of the combined asphalt layer corresponding to the 85-percentile deflection values are presented in Table 2. Lane 1, which carries less traffic, had a considerably higher stiffness compared to lanes 2 and 3 which carry all the truck traffic.

TABLE 2 Stiffness Results from FWD Analysis

Lane	Northbound	Southbound
1—Fast	7,100 MPa	7,600 MPa
2—Middle	5,600 MPa	5,300 MPa
3—Slow	4,600 MPa	4,900 MPa

Using a simple layer equivalence approach it was demonstrated that the test results for lanes 2 and 3 would suggest that the lower stiffness is consistent with a cracked surface layer and a sound base. The results from the FWD testing thus supported the view that the base was in a sound condition while the reduced stiffness was associated with surface cracking as illustrated in Figure 6. Consequently, the pavement still contained a sound base of high stiffness that could be incorporated in the rehabilitated pavement structure.

The mode of pavement distress identified on this project is currently not supported by design methods. No classical models exist to enable the prediction of this form of distress. The occurrence of the surface cracking is considered to be a function of the hard binder in the surface combined with load-associated effects since the cracking was more evident in heavily traveled lanes 2 and 3.

With stiff pavement layers, surface cracking can be predicted to occur at the edge of the loaded area using sophisticated pavement models that are based upon dissipated energy or damage mechanics. The example shown in Figure 7 illustrates the case of a thick pavement structure made with a dense asphaltic material similar to that laid on I-287 and for which the highest value of dissipated energy occurs at the top of the pavement structure (*3*). This type of analysis will show that the dissipated energy will also be highest at the surface for thick pavement even if a wearing course with higher binder content is used. The analysis concluded that to limit cracking in the wearing course it is necessary to design the surface course mixture for high fatigue life, with as little age-hardening susceptibility as possible.

As a consequence of the analysis, a polymer-modified asphalt surface course was selected for the final surface. The surface cracks were removed by milling out the top 3 in. of pavement structure and then laying down two 2-in. layers, consisting of an HMA base course followed by a polymer-modified surface course.

TESTING DURING CONSTRUCTION

A planned program of testing was organized during the construction project, which had the objectives of (1) assessing the as-built pavement structure and (2) determining how the proposed mixtures compare to the Superpave asphalt specifications.

During the construction period a test area with 1200 tons of New Jersey's first Superpave mix was identified. This area was chosen for both mixture and pavement structural evaluation.

FWD Testing

The section was tested with the FWD after milling (December 1996) and after the final resurfacing (April 1997). The stiffness modulus values corresponding to the 85-percentile deflection values were the transposed into structural numbers using the procedures contained within the AASHTO Pavement Design Manual. This resulted in structural numbers for the slow lane of 5.7 in. before construction, 4.5 in. after milling, and 7.6 in. after final overlay. The existing structural capacity before overlay, using this approach, is a remaining pavement life of

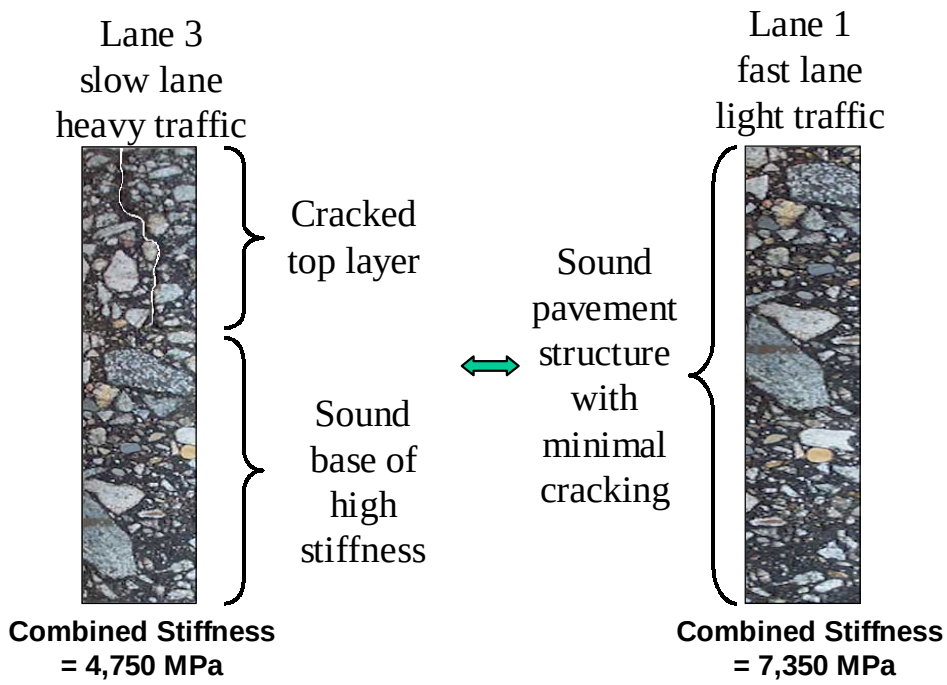

FIGURE 6 Average FWD results. [Combined stiffness refers to the stiffness modulus obtained from back analysis when the asphaltic layers (base and wearing course) have been combined into a single layer.]

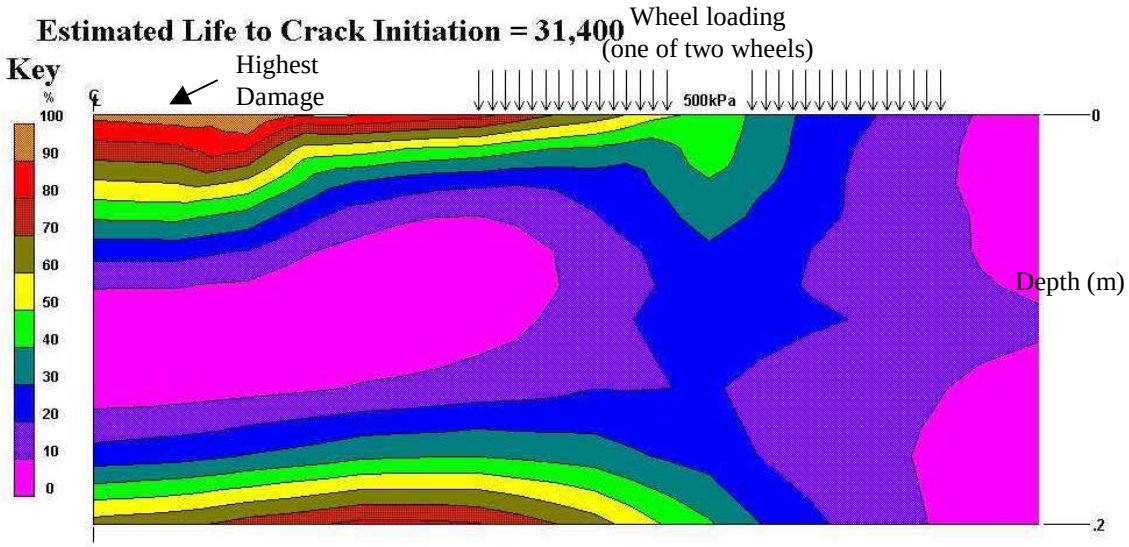

FIGURE 7 Surface-cracking calculations: pavement of 8 in. dense bitumen macadam with applications of a 40-kN duel wheelload; contours of relative (percent) fatigue damage; cross section from center line of loaded area with position of one wheel indicated (*3*).

7,000,000 ESALs compared to the rehabilitated structural capacity of 69,000,000 ESALs. The rehabilitated structure essentially offers an indeterminate pavement life that will need only periodic resurfacing to restore the ride quality.

Mix Testing

The objective of the mixture testing was to test the materials being used in accordance with the Superpave standards to determine what changes may be needed to NJDOT's standard I-4 surface course mix for compliance with the new standards. When this mixture was compared to the Superpave requirements it was observed that only a very minor change to the gradation was required to convert existing mixture to meet 12.5 mm Superpave requirements (*4*). The majority of NJDOT's standards for consensus properties were previously specified at the same or at higher quality values than used in the Superpave specifications.

The binder used in the surface course corresponds to a PG76-22 (see Table 3). This would compare to a PG58-22, which would be used for the region as per the standard Superpave procedures. The PG76-22 represents 3-grade bumps (e.g. PG58-22 to PG76-22) to consider heavy and slow traffic loads.

CURRENT PERFORMANCE

The project was inspected in January 2001 and is illustrated in Figure 8. Currently, after 4 years of traffic, the project is performing to acceptable standards with little signs of distress.

The most recent pavement management data available for this project is presented in Figures 9 and 10 that illustrate the pavement rating and skid properties.

The pavement rating data was obtained on December 11, 1998 (approximately 2 years after surfacing). The pavement rating is presented as both a Ride Quality Index (RQI) and Surface Distress Index (SDI) as measured with the ARAN profile device. (ARAN is the trade name for the equipment manufactured by Roadware of Paris, Ontario.) The pavement ratings (RQI and SDI) are related to the condition as indicated in Table 4.

TABLE 3 Superpave Binder Test Results

Test Parameter	Results	Specification Requirement
Safety/Handling		
Flash Point (Cleveland Open Cup)	318°C	>230°C
Viscosity @ 135°C	1.55 Pa-s	<3 Pa-s
Loss on heating	0.25%	<1%
High Temperature		
$G^*/\sin\delta$, Original	1.56 @ 76°C	>1 kPa
$G^*/\sin\delta$, RTFOT	3.04 @ 76°C	> 2.2 kPa
Intermediate Temperature		
$G^*/\sin\delta$	3550 kPa @ 22°C	<5000 kPa
Low Temperature		
Creep Stiffness, *S*	220 MPa @ -12°C	<300 MPa
m-value	0.315 @ -12°C	>0.300

Based on the above definitions the pavement rating is in a “good” to “very good” condition with few (if any) visible signs of deterioration.

The average skid number obtained for the site on September 6, 2000, was 49.6 and with a standard deviation of 3.3 (see Figure 10). This would meet the requirements for a traffic speed of approximately 70 mph and is considered acceptable.

Further long-term monitoring of the project is planned during years ahead to determine the effectiveness of the treatments used.

The value of the engineering approach to this pavement structure has been demonstrated via the efficiencies in the pavement structural design compared to a full reconstruction option that would have been used based upon a limited visual survey.

CONCLUSIONS

The study of the pavement structure at Morristown, New Jersey, illustrated the need to consider the occurrence of surface cracking in the design and execution of pavement rehabilitation projects in New Jersey. The specific findings obtained during this study are as follows:

- This project illustrated the need to develop pavement solutions with adequate consideration to the mode of distress occurring on site.
- The surface cracking appears to be associated with a hard binder in the surfacing materials.
- FWD tests confirm that the base is of adequate stiffness for the overlay design.
- The developed solution involved evaluation with a FWD and the use of polymer-modified binders in the surface course.

FIGURE 8 Current condition.

TABLE 4 Ride Quality and Surface Distress Categories

RQI/SDI	Visual Category	Remarks
0.0 to 1.0	Very Poor	Pavements that are in an extremely deteriorated condition. The facility is passable only at reduced speeds, and with considerable ride discomfort. Large potholes and deep cracks exist. Distress occurs over 75 percent or more of the surface.
1.0 to 2.0	Poor	Pavements that have deteriorated to such an extent that they affect the speed of free flow traffic. Flexible pavements may have large potholes and deep cracks. Distress includes raveling, cracking, and rutting that occurs over 50 percent or more of the surface. Rigid pavement distress includes joint spalling, faulting, patching, cracking, and scaling, and may include pumping and faulting.
2.0 to 3.0	Fair	The riding qualities of pavements in this category are noticeably inferior to those of new pavements, and may be barely tolerable for high-speed traffic. Surface defects of flexible pavements may include rutting, map cracking and extensive patching. Rigid pavements in this group may have a few joint failures, faulting and cracking, and some pumping.
3.0 to 4.0	Good	Pavements in this category, although not quite as smooth as those described above, give a first class ride and exhibit few, if any, visible signs of surface deterioration. Flexible pavements may be beginning to show evidence of rutting and fine random cracks. Rigid pavements may be beginning to show evidence of slight surface deterioration as well as minor cracks and spalling.
4.0 to 5.0	Very Good	Only new (or nearly new) pavements are likely to be smooth enough and sufficiently free of cracks and patches to qualify for this category. All pavements constructed or resurfaced during the date year would normally be rated very good.

- Adequate service life was achieved with only a relatively thin overlay.
- This approach had significant cost benefits (savings) for the project.
- Superpave mixes were not significantly different from New Jersey's current I-4 HD (Heavy Duty) mixes.
- The pavement management information collected to date indicates that the surface is performing adequately.

Planned monitoring of performance over the following years will further assist with the development of a rational approach to the design of pavements which exhibit severe surface cracking.

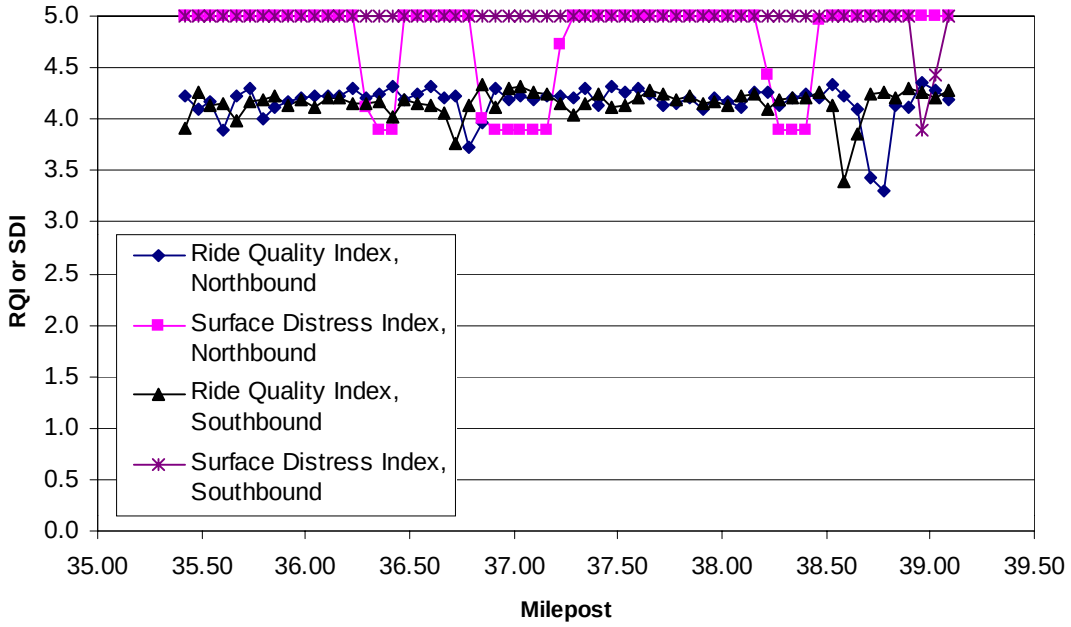

FIGURE 9 Pavement ratings (RQI and SDI) versus Milepost I-287.

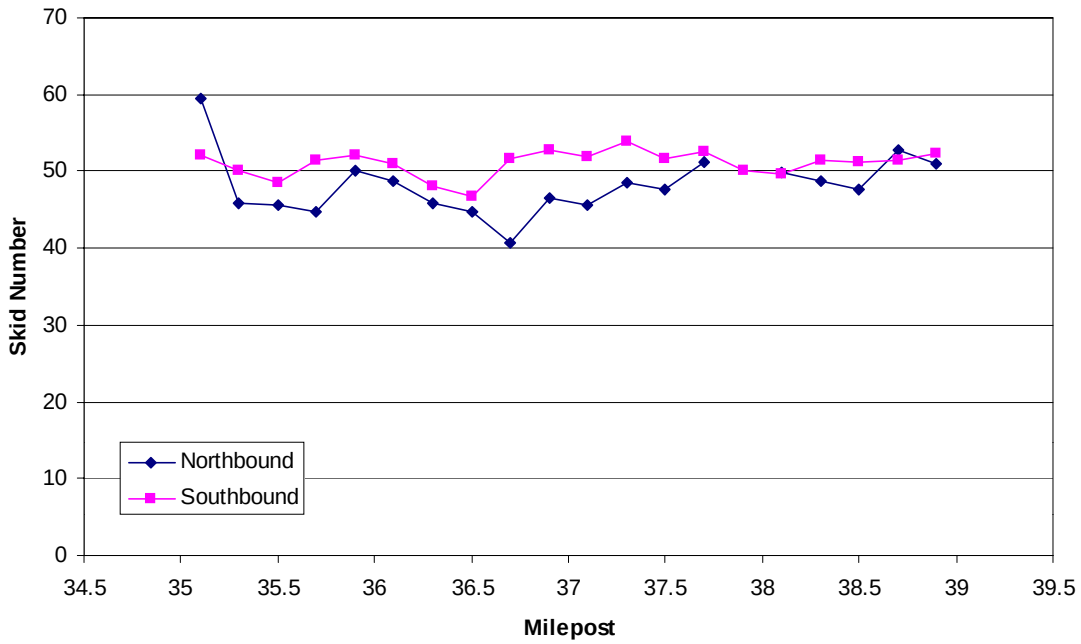

FIGURE 10 Skid numbers versus Milepost I-287.

REFERENCES

1. Parsons Brinckerhoff. *Pavement Design, Route I-287 Section 9S + 10V*. Dec. 1993.
2. Brown, S. F., W. S.Tam, and J. M. Brunton. Structural Evaluation and Overlay Design: Analysis and Implementation. *Proc., Sixth International Conference on the Structural Design of Asphalt Pavements,* Vol. 1, Ann Arbor, Mich., 1987, pp. 1013–1028.
3. Rowe, G. M., and S. F. Brown. Fatigue Life Prediction using Visco-Elastic Analysis. *Proc., International Society of Asphalt Pavements*, Seattle, Wash., 1997.
4. SWK Pavement Engineering. *Evaluation of SHRP Tests and Mixture Design Procedures and Verification of Structural Improvement Following Overlay Construction on Route I-287 (Sections 9S and 10V).* Report Submitted to New Jersey Department of Transportation, July 1998.

ADDITIONAL RESOURCE

Deacon, J. A., A. A. Tayebali, J. S. Coplantz, F. N. Finn, and C. L. Monismith. Fatigue Response of Asphalt-Aggregate Mixes: Part III Mix Design and Analysis. SHRP Project A-003, Asphalt Research Program, Institute of Transportation Studies, University of California, Berkeley, Nov. 1992.

Illinois Extended-Life Hot-Mix Asphalt Pavements

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At the request of the hot-mix asphalt (HMA) industry, the Illinois Department of Transportation (IDOT) initiated efforts to develop contract provisions for the design and construction of "perpetual" HMA pavements. A "perpetual" pavement for this effort is defined by the Asphalt Pavement Alliance as a pavement structure consisting of a rut-resistant, impermeable, and wear-resistant top structural layer, with a rut-resistant and durable intermediate layer, and a fatigue-resistant and durable base layer. IDOT believes a more appropriate term is extended-life HMA pavement.

This paper describes (1) the approach taken by Illinois and its hot mix industry, and (2) the issues involved in developing the contract provisions.

PARTNERSHIP APPROACH

The approach taken was in the form of a partnership of the Illinois Department of Transportation (IDOT), academia, contractors, asphalt suppliers, and national experts. This approach ensured the best information, knowledge, and experiences were available to develop an appropriate specification for Illinois conditions.

The following were represented on the task group:

- IDOT
- Contractors
- Asphalt suppliers
- Aggregate suppliers
- Academia
- Industry association representatives
- National experts

In the planning stages for this effort, it was agreed that a specification could be developed through a series of four meetings spaced 1 to 2 months apart, with various subcommittees working on specific issues between meetings. The first meeting was held June 14, 2000, with the goal of having a draft document by December 31, 2000. This ambitious goal was substantially accomplished.

At the first meeting, the task group determined that the main topics to be addressed could be broken down into the following four distinct areas.

- Pavement substructure,
- Material durability and stripping prevention,

- Fatigue-layer design analysis and bottom-layer mixture design, and
- 20-year pavement surface.

The task group divided itself into subgroups representing these four areas. Issues, questions, and concerns discussed during the general meetings were in many instances assigned to the appropriate subgroup for consideration, with recommendations presented at the next general meeting.

PERPETUAL PAVEMENT DESIGN CONCEPT

The perpetual pavement concept of a three-layered system, renewable surface, middle rut-resistant layer, and lower fatigue resistant layer was explained by representatives of the Asphalt Pavement Alliance. After discussing and fully understanding the concepts, the task group agreed they were reasonable for use in Illinois.

Thickness Issues

The first challenge was to determine appropriate total thickness of the pavement structure and thicknesses for each of the three individual layers. Since 1989, IDOT has been using a mechanistically based pavement-thickness design procedure. It includes limiting the maximum tensile strain at the bottom of the lower lift, as well as limiting the subgrade stress. The IDOT design typically results in design strain levels at or below what pavement design experts are suggesting (i.e., 0.60 microstrain for perpetual pavement). Discussions with the design that procedure authors indicated that the fatigue algorithms used to characterize mixtures used in Illinois were quite conservative. As such, the existing thickness design would already have taken into account any type of mixtures the task group would recommend. To meet the 6-month goal of having a specification, it was decided to use the current design thickness procedure with no change. A long-term goal would be to review the thickness design to recognize a more fatigue-resistant lower layer than currently assured in the design procedure.

Prepared Subgrade

Currently, IDOT’s standard cross section for full-depth hot-mix asphalt (HMA) has the pavement built on top of 12 in. of lime modified soil with added provisions for very weak subgrade soils. The department proposed to replace the 12 in. of lime-modified soil with 12 in. of aggregate subbase. This subbase is used typically in the Chicago area when reconstructing pavements. The gradation is such that a wide range of sized material can be used, but does result in a drainable layer. A review of the reduction in strain at the bottom of the HMA pavement produced a microstrain reduction of 2. Questions were raised as to the benefit of a 2-microstrain reduction compared to the cost of incorporating 12 in. of granular subbase, especially in areas of the state where such aggregate is not readily available. Estimated cost increases ranged from 6 to 14 percent of a typical cross-section cost, depending on the specific project. This issue could not be resolved by the group and was forwarded to IDOT management to resolve.

Underdrains

Current IDOT design standards require longitudinal underdrains to be placed under the shoulder/pavement joint on both sides of the pavement. The pipe is located low enough to drain the aggregate layer it if exists, or 24 in. below the surface of the pavement when the pavement is built on top of lime-modified soil. The question was raised as to the value of underdrain in the cross section use, especially with clay-type soils in Illinois. After some discussion, it was decided that not enough performance data existed to eliminate the underdrain at this time.

Fatigue-Resistant Layer

Although the pavement thickness design procedure was felt to be conservative in regards to fatigue using existing IDOT mixtures, it was decided to develop a “rich bottom base” to improve the fatigue resistance. Various different mixes were discussed in terms of appropriateness in meeting the characteristics of a highly fatigue-resistant HMA. From a mix design perspective, a very specialized mixture could be designed, but its practicality and resulting cost was questioned. To reduce confusion with additional mix designs, differing aggregate requirements, and varying production criteria during construction, it was deemed most appropriate to modify the mixture used in the middle layer

The selected rich-bottom base used the middle-layer mix design with an asphalt content that provided 2.5 percent air voids at the same gyration level. Adding one-half percent more of asphalt to the 4 percent design wa suggested, but the task group agreed that designing for 2.5 percent voids was more realistic in defining the mixture characteristics desired in the layer. The additional asphalt should also improve moisture damage resistance, provide additional fatigue life, and be easy to place and compact. Also, a higher in-place density requirement of less then 6.0 percent voids for this layer was included. All these characteristics would ensure a durable fatigue-resistant layer. This layer would be constructed in one 4-in. lift.

Intermediate Pavement Layer

After reviewing performance data and the members’ review of the mix design requirements, the task group determined that the department’s current specified binder mixture and mix design procedure was appropriate for the intermediate pavement layer. The design procedure and ingredient requirements follow the intent of Superpave, and historically Illinois has provided rut-resistant mixtures. The appropriate gyration level would be utilized for the expected traffic over a 20-year period, as recommended by Superpave.

Renewable Surface Layer

To achieve the goal of a 20-year surface life, the use of stone matrix asphalt (SMA) was chosen to resist both rutting and durability problems for the renewable surface layer. The use of IDOT’s dense-graded mixtures was discussed, but ultimately not pursued. The task group felt SMA could provide a higher level of long-term durability due to its mixture characteristics.

To determine appropriate thickness of the surface layer, a shear stress analysis of the pavement structures was conducted and the results of rutting and surface cracking studies were reviewed. In most cases, the studies indicate that rutting and cracking occur in the top 4 in. Based on the analysis and studies, the following top layer thicknesses were selected for the SMA:

Traffic	*Total SMA Thickness (Inches)*
Low	2
Medium	4
High	6

For the very high equivalent single-axle load (ESAL) pavements, a 6-in. total thickness of SMA was felt to give a factor of safety to help resist rutting. (Although the specific definitions of low, medium, and high have not been finalized, high may ultimately be defined as 25 million ESALs or greater, based on a 20-year traffic analysis.) For the lower-volume pavements, no need was recognized for the high rut-resistant SMA in the lower part of the surface layer.

However, the task group agreed that the top 6 in. of the pavement would require modified binder to minimize surface cracking. For those pavements with either 2 or 4 in. of SMA, the remaining 4 or 2 in., respectively, would be of the intermediate layer mixture with modified asphalt.

Asphalt Penetration Grade Selection

The use of polymer-modified asphalt cement in both the base layer and intermediate layer was not felt to be cost-effective. It was decided to use modified asphalt in the 6 in. of the surface layer to help resist surface cracking and rutting. The department's current penetration grade selective requirement for full-depth pavements for northern Illinois was felt appropriate for statewide use. It is recognized this is a conservative approach for pavements built in the southern parts of the state. However, the approach would help minimize low-temperature cracking and the polymer would improve durability. For the 6-in. surface layer, "grade bumping" to accommodate slow and standing traffic will use the department's current guidelines.

Use of Hydrated Lime

Illinois uses liquid additives when it determines a mixture is susceptible to moisture damage (stripping). Based on the Illinois version of AASHTO T-283, there were suggestions to make the existing test procedure more stringent. Ultimately, the task group elected to require the use of hydrated lime for all mixtures used in the extended-life pavement. Along with the mandatory use of lime, the mixtures will still have to pass the department's current tensile strength–ratio requirements. Any future changes to the test procedures would be considered for all mixtures, not just for perpetual pavements.

Mixture Durability Enhancements

The task group agreed on the following items to enhance the durability of the mixtures and so increase the life of the pavement structure.

- Increase minimum lift thickness to 3 to 6 times nominal maximum aggregate size to allow higher in-place density.
- Require positive dust control on HMA plants for better mixture production control.
- Require priming with a polymer material between each lift to promote bonding.

- Require all-virgin aggregate in the bottom layer. This eliminates variability of mixture stiffness due to reclaimed asphalt pavement.
- Require the use of a material-transfer vehicle or a paving machine capable of re-mixing the bituminous mixture for all lifts of all mixtures to reduce the potential for segregation. To eliminate premature pavement fatigue from the vehicle, its use is permitted only on constructed lifts 10 in. or greater in thickness.
- Revise in-place field density testing to promote uniform levels of density across the mat.
- Increase the in-place density requirements for the middle layer to less than 7 percent air voids.

Longitudinal Joint Construction Practices

The task group reviewed the need to improve longitudinal joint durability. It was felt the longitudinal joints would be the portion of the pavement prone to long-term durability problems. The task group discussed procedures such as full-width paving to eliminate longitudinal joints, and cutting of the joint prior to the adjacent lane being paved to improve the density at the joint.

Three recommendations were ultimately made to improve durability of longitudinal joints. First, improve the quality of the material near the joint by requiring mainframe screed extensions and utilizing good paving practices. Second, change the field density–testing procedures to put more emphasis on nuclear tests 2 ft from the joints. Third, require the use of a spray-on polymer-modified tack coat on the vertical face of all lifts that will become longitudinal joints. This tack coat will extend on existing lifts for a width of 6 in. to the side of the existing joint. In combination, all three procedures should improve the life of the longitudinal joints.

Individual Layer Thickness Tolerance

Due to the importance of each of the three layers—the bottom fatigue layer, the intermediate layer, and the surface layer—the concern for construction thickness tolerances of each layer was discussed. The pavement thickness design is already conservative, with the lower fatigue layer given the same fatigue resistant as the intermediate layer, and with modified asphalt binder being required in the upper 6 in., which fully encompasses the shear zone of 3 to 5 in. Therefore it was decided that existing department construction tolerances and total pavement thickness requirements would be used.

Pavement Smoothness Requirements

The department has an ongoing pavement smoothness initiative to improve the smoothness of newly constructed pavements. The department believes that smoother constructed pavements address the public’s desire for smoother pavements. Also, pavements built smooth will remain smoother throughout their service life. The task group concurred with including the department’s most stringent pavement smoothness specifications for the extended-life HMA pavements.

CONCLUSION

The task group substantially met the goal of having draft documents ready by December 31, 2000. This was accomplished by taking current IDOT pavement and mixture designs, as well as construction practices, and reviewing them for appropriateness when using the three-layered perpetual pavement concept, and then making adjustments where needed. Most of the effort was in improving the durability of mixtures to ensure that they last indefinitely for the bottom and intermediate layers and at least 20 years for the renewable surface layer, with no rutting, surface cracking, or durability issues.

One major item yet unresolved is the need for a 12-in. granular subgrade under the three-layered HMA pavement. An approach being considered is to require the aggregate subbase only for very high levels of traffic.

Two other goals currently being addressed by IDOT and the HMA industry are to develop specifications for SMA mixtures for medium- and low-traffic situations to ensure durability, and specifications for incorporating hydrated lime in HMA mixtures.

Future efforts will include reviewing the thickness-design procedure to account for the different mixtures used in the three layers, as well as maximizing the benefits of such a design.

Currently, the HMA industry in Illinois is pursuing projects where the extended-life HMA pavements specifications can be included.

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