Guidelines for Evaluation of Highway Pavements for Rehabilitation

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Abstract. Although all State DOT agencies are engaged in pavement rehabilitation, fairly few of them have any more than the most simple and general guidelines for selection of rehabilitation strategies. This paper presents an overview of the project-level evaluation process described in the *Guide for Selection of Pavement Rehabilitation Strategies*, developed under NCHRP Project 1-38.

The guidelines for pavement data collection address pavement section inventory, traffic analysis, distress surveying, nondestructive deflection testing, materials sampling and testing, profile and roughness measurement, friction measurement, drainage inspection, and other nondestructive testing. The guidelines for project-level pavement evaluation address distress evaluation, structural evaluation, and functional evaluation, drainage evaluation, and identification of uniform sections. Trigger values are suggested for key condition levels at which a pavement is generally considered to need a structural improvement. Similarly, trigger values are suggested for key condition levels at which a pavement is generally considered to need a functional improvement.

The results of the distress, structural, functional, and drainage evaluations lead directly into the selection of appropriate rehabilitation techniques for each of the three main pavement types considered, combination of the individual rehabilitation techniques into one or more feasible rehabilitation strategy alternatives, life-cycle cost analysis of the alternatives, and selection of a rehabilitation strategy.

APPROACH

The *Guide for Selection of Pavement Rehabilitation Strategies* was developed after a review of the pavement rehabilitation practices of State DOTs and the literature available on pavement evaluation, rehabilitation techniques, and selection of rehabilitation strategies. The rehabilitation strategy selection procedures used by different highway agencies differ in their details, but typically consist of the following principal activities:

1. Data collection,
2. Pavement evaluation,
3. Selection of rehabilitation techniques,
4. Formation of rehabilitation strategies,
5. Life-cycle cost analysis, and
6. Selection of a rehabilitation strategy.

The first two steps in this process, data collection and pavement evaluation, are summarized in this paper.

PROJECT-LEVEL DATA COLLECTION

The purpose of project-level data collection is to gather all of the information necessary to conduct an evaluation of the pavement´s present condition and rehabilitation needs, develop one or more rehabilitation strategies, predict the performance of each strategy, and estimate the cost of each strategy.

Roadway Section Definition

This involves identifying the location of the project by route name or number, direction, county, nearby city or town, milepost limits, and/or station limits – all information that will be needed to locate the project and estimate rehabilitation costs over its length. Information such as the locations of bridges, underpasses, and interchanges, station equations, etc., should also be noted.

Pavement Section Inventory

This involves examining pavement management files, construction records, and reports from past evaluation and rehabilitation activities for the purpose of determining the pavement type, pavement age, pavement layer materials and thicknesses, number of lanes, widths of lanes and shoulders, predominant subgrade soil type, and subdrainage features.

Traffic Analysis

The current traffic volumes and axle loadings and anticipated traffic growth rates should be determined. Recommendations for traffic data collection is provided in References 7, 8, and 9. With this information, traffic volumes and axle loadings may be forecasted for the design traffic lane (usually the outer lane in one direction) over whatever design periods are later selected for the rehabilitation strategy alternatives considered.

For the purposes of pavement rehabilitation strategy selection, the current and projected future traffic should be characterized in terms of whatever traffic input is used in the resurfacing and reconstruction design procedures used by the agency. In the 1993 AASHTO[1,2] methodology, which is used by many State DOTs, the mixture of anticipated axle loads is expressed in terms of an equivalent number of 18-kip single-axle loads (ESALs). Guidelines for calculation of ESALs are provided in Appendix D of the 1993 AASHTO Guide.[1] The equations for calculating load equivalency factors are given in the Highway Research Board's report of the proceedings of the 1962 St. Louis Conference on the AASHO Road Test.[10] The Asphalt Institute procedures for asphalt pavement design[3] and overlay design[4] also use ESALs as the traffic input. The Portland Cement Association procedures for concrete pavement design[5] and concrete overlay design[6] use axle load data directly.

Distress Survey

Rehabilitation of a pavement is most likely to be successful – that is, provide satisfactory performance and cost-effectiveness – if it is selected on the basis of knowledge of the types of distresses occurring in the pavement and the causes for those distresses, and it effectively repairs those distresses. A good understanding of the types of distress which may occur in different types of pavements, and the causes for those distresses, is therefore essential to the success of pavement rehabilitation. A field survey is required to accurately determine the types, quantities, severities, and locations of distress present.

The *LTPP Distress Identification Manual*[11] is widely used to guide field technicians in identifying distress types, rating distress severities, and measuring distress quantities on highway pavements. Guidelines for distress identification and measurement are also given in References 12, 13, and 14. Similar distress identification manuals have been developed by several State DOTs.

Automated devices are also available for use in conducting distress surveys. These devices operate at highway speeds without disrupting traffic, and thus are particularly well suited to high-traffic-volume situations. Information on the capabilities of some automated distress survey devices is summarized Reference 15.

Nondestructive Deflection Testing

While some agencies may not be equipped for nondestructive deflection testing, such testing is always highly desirable, especially when the distress survey indicates that the pavement requires a structural improvement. A Falling Weight Deflectometer (FWD) or other device capable of applying loads comparable in magnitude to truck wheel loads is recommended for this purpose. Overviews of the different devices available for nondestructive deflection testing are provided in References 14, 16, 17, and 18. General guidelines for deflection testing are given in ASTM D4695, and guidelines for testing with falling weight deflectometer devices are given in ASTM D4694. Guidelines for deflection testing are also given in References 1, 14, 18, and 19.

Purposes of Deflection Testing

Deflection testing is conducted on asphalt pavements for the purposes of backcalculating the stiffnesses of the subgrade and pavement layers, assessing the remaining life of the pavement, and/or determining the overlay thickness required to satisfy a structural deficiency. Asphalt highway pavements should be tested in the outer wheel path of the outer traffic lane, which is just one to two feet from the lane edge, for the purpose of attempting to assess the extent of fatigue damage. The assumption of infinite horizontal layers is thus violated, but this is generally ignored.

Deflection testing on concrete pavements is conducted at slab interiors, to backcalculate the stiffnesses of the subgrade and pavement layers; at transverse and longitudinal joints and cracks, to measure deflection load transfer and differential deflection, and at slab corners, to detect voids under the slabs.

On concrete highway pavements, slab interiors are usually tested at the middle of the outer lane, for the purpose of backcalculating the dynamic modulus of subgrade reaction (k value) and concrete elastic modulus. A concrete slab of highway lane width (typically 12 ft) is narrower than that required to comply with the infinite horizontal layer assumption, so adjustments for the finite slab size are required when analyzing the deflection data. Interior deflections are not measured on concrete slabs with the goal of directly assessing the fatigue damage, so testing at the midwidth of the slab is no different than testing in the outer wheelpath, and may be preferable from the standpoint of keeping the load plate as far away as possible from the lane/shoulder edge. Testing in the outer wheel path may be more convenient, however, if interior tests and joint load transfer tests are to be combined in one pass down the traffic lane.

Deflection load transfer is measured for use in estimating the distribution of stress between adjacent slabs, which may be used in a mechanistic analysis of the fatigue life of the pavement. Deflection load transfer is also considered to be related to the development of faulting at joints and cracks. The deflection load transfer at transverse joints may also be used to select a load transfer coefficient (J factor) for use in the 1993 AASHTO method of overlay design. For all of these purposes, however, deflection load transfer measurements need to be adjusted for slab temperature in order to be meaningful.

One set of deflection measurements can be used to calculate both differential deflection (loaded side deflection minus unloaded side deflection) and deflection load transfer (ratio of loaded side deflection to unloaded side deflection). Differential deflection is more relevant than the deflection load transfer to the rate of deterioration of joints and cracks, and to the likelihood of reflection cracking in asphalt overlays.

Corner testing for void detection is the least commonly conducted type of deflection testing. It may be warranted if the pavement already has some corner breaks, has very poor transverse joint load transfer, or manifests other signs of loss of support (e.g., pumping of fines or water at joints).

On asphalt-overlaid concrete pavements, deflections are measured at slab interiors to backcalculate layer and foundation stiffnesses. To measure deflection load transfer and differential deflection, deflections are measured at transverse and longitudinal joints and cracks. Deflection load transfer is conceivably useful in a mechanistic analysis of the asphalt-overlaid concrete pavement's remaining life, although this is not a common or straightforward analysis. The differential deflection is useful in identifying the joints and cracks deteriorating most rapidly. No accepted procedures have yet been established for void detection testing on asphalt-overlaid pavements.

Deflection Testing Interval

Typical testing intervals for highway pavements are between 100 and 500 ft (between about 50 and 10 points per mile, respectively). Deflections in asphalt pavements, and to some extent, in concrete pavements, tend to become more variable with time. Thus, a longer testing interval is appropriate for younger pavements, and a shorter interval is more appropriate for older pavements.

Measurement and Consideration of Temperature

Asphalt mix temperature measurements are required when testing asphalt and asphalt-overlaid pavements because the resilient modulus of asphalt concrete varies substantially with temperature. It is not uncommon for the AC mix temperature to vary by 30ºF or more during a typical day of deflection testing. This magnitude of temperature variation could easily correspond to a variation of 500,000 psi in asphalt concrete modulus. Failure to account for this variation will result in incorrect moduli being used for the asphalt layers.

The temperature at the middepth of the asphalt mix should be measured at least three times during each day of testing, to establish a curve of mix temperature versus time that may later be coordinated with the times recorded in the deflection data file. The air and surface temperatures are usually measured at the same time. Mix temperature is influenced by sunshine as well as air temperature. If parts of the pavement are shaded and others are not, temperatures should be measured in both shaded and unshaded areas, and it should be noted at each deflection location whether the location is shaded or unshaded. If it is not possible to obtain mix temperature measurements, the mix temperature may be estimated from air and surface temperatures, using procedures described in References 20, 21, 22, 23, and 27.

Backcalculation of asphalt pavement layer stiffnesses becomes difficult when the asphalt concrete modulus is greater than 2 million psi or less than 200,000 psi. Therefore, testing should be done when the temperature (measured in the asphalt concrete) is between 40^0F and 100^0F. When testing in sun and hot temperatures, the temperature of the asphalt concrete may be much higher than the air temperature. If mix temperature measurement is not possible, the pavement surface temperature (measured with an infrared gun) is better than the air temperature as an approximate indicator of whether the asphalt concrete temperature is too high for testing.

Temperature measurement is required when testing concrete pavements to monitor the temperature gradient in the concrete, and to relate the load transfer measurements to the temperature. The temperature gradient is monitored by measuring the temperature at three depths (for example, one quarter, one half, and three quarters of the slab thickness), at least three times during each day of testing, to establish a curve of temperature gradients versus time that may later be coordinated with the times recorded in the deflection data file.

Careful thought should be given to the allowable temperature range for testing concrete pavements for load transfer measurement purposes. Load transfer is highly dependent on temperature. Deflection load transfer (the ratio of unloaded side deflection to loaded side deflection, expressed as a percentage) follows an S-shaped curve, asymptotically approaching 100 percent at high temperatures, and a minimum percentage (greater than 0) at low temperatures. The full temperature-load transfer curve cannot be extrapolated from load transfer measured at only one temperature. This curve can be established, however, using measurements at a few selected reference points, at two significantly different temperatures, e.g., 20ºF or more apart.

Load transfer testing should be avoided when slab temperatures are so warm that the joints and cracks are closed. At what temperature this will occur depends on the joint/crack spacing. When testing in hot weather, measured load transfers should be checked in the field to see if they are very close to 100 percent. If they are, further load transfer testing should be postponed to some later time when the slabs are cooler and the joints have opened somewhat.

For concrete pavements, it is traditionally recommended to avoid testing for backcalculation purposes when a significant temperature gradient exists through the slab thickness. This is often taken to mean avoiding testing in the slab interiors during certain hours of the afternoon when the top of the slab is hotter than the bottom and the slab is curled downward, and avoiding testing at the edges and corners during certain hours of the night when the top of the slab is cooler than the bottom and the slab is curled upward. Whether or not a significant temperature gradient exists in the slab should be determined by measurement, as described earlier.

The real concern, however, is not merely whether the slab is curled, but whether the slab is curled out of contact with the underlying foundation. A slab resting on a soft foundation may be curled upward or downward and still be in full contact with the foundation at the location at which the deflections are measured. In many cases when the slab rests on a soil, gravel, or weakly stabilized subbase, the slab will not curl out of contact. The backcalculated slab and foundation moduli will be the same as if the deflections were measured while the slab was flat. However, when the slab rests on a high-strength stabilized base, the potential for curling out of contact is a concern. If the slab is curled out of contact with the foundation at the location where the deflections are measured (e.g., curled up and tested at the edge or corner, or curled down and tested at the midslab interior), then the backcalculated foundation modulus will be erroneously low. Procedures have been developed for analyzing a series of deflections measured at different load levels, to determine when the slab is in contact with the foundation.[24]

It is not necessarily true that a slab is flat when it has no temperature gradient through its thickness. It is conceivable that a slab may be curled with a zero temperature gradient. This may occur if a temperature gradient existed in the slab during its initial set when it was constructed. Although multidepth temperature measurement is always recommended when testing concrete slabs, the more reliable way to ascertain whether or not a slab is in contact with the foundation at a given location is to conduct a load sweep, just as is done for void detection at corners. A series of loads of inccreasing magnitude are applied, and the relationship of load magnitude to deflection at the center of the load plate is examined. A straight-line relationship between more than two load-deflection points indicate that an elastic response has been achieved, signifying that at those load levels the slab is in contact with the foundation. Deflections measured at sufficiently high load levels to insure slab-foundation contact may indeed be used in backcalculation of the slab and foundation moduli.

The traditional method of testing for voids has been to use a load sweep at slab corners and assess the linearity and intercept of the load-deflection plot. This has been done on many projects without regard to whether or not the slabs were curled at the time of testing. References 24 and 25 provide guidelines for evaluating corner deflections to distinguish curling from loss of support.

Coordination of Deflection Testing with Visible Distresses

Asphalt pavements with alligator cracking in the wheelpaths may show significant variability in deflections and also in the degree of distress along the length of the project. A correlation can usually be observed between the severity of the alligator cracking and the magnitude of the maximum deflection. Assuming that one of the primary purposes of the deflection testing of an asphalt pavement is to assess its structural condition, it is useful to test at locations with various degrees of cracking. Even severely alligator-cracked areas can usually be tested.

It is more difficult to relate deflection magnitude to cracking in a concrete or asphalt-overlaid concrete pavement. One possible option is to measure deflections at the preselected interval and test in both cracked and uncracked areas (an area in the interior of the slab, away from joints or edges, without linear cracks or localized failures within the deflection basin). If deflection basins are measured this way in both cracked and uncracked areas, the concrete modulus backcalculated from the deflections should be considered an "effective" modulus, which represents not the true stress-strain behavior of intact concrete but rather the condition of the slab in its current state of cracking. An example of this approach to structural evaluation is found in the work done by Rollings,[26] in which a relationship was established between the "E ratio" (intact slab modulus versus cracked slab effective modulus) and the Structural Condition Index determined from cracking data. However, measuring concrete or asphalt-overlaid concrete pavement deflections near cracks poses several practical difficulties. Different measurements will be obtained depending on where the FWD load plate is with respect to the crack.

The alternative is to backcalculate the concrete modulus only from deflection basins measured away from cracks. This modulus should not then be considered an indicator of the degree of structural damage in the slab. The backcalculation results and the distress survey results must then be considered together to form an overall assessment of the structural condition of the slab.

Target Load Levels

At least two and often three target load levels are used in deflection testing. One of the reasons for testing over a range of load magnitudes is to analyze whether or not the foundation exhibits a nonlinear response to load. Another reason is to be confident of obtaining at least one deflection basin of sufficient curvature for successful backcalculation. As a rule of thumb, a target load of sufficient magnitude to produce a mean maximum deflection of 6 mils is needed to obtain deflection basins of sufficient curvature to lend themselves to successful backcalculation.

For highway pavements, at least one of the target load levels should be 9000 pounds, to facilitate an analysis of the pavement's structural capacity using the 1993 AASHTO Guide method.[1] Suggested target load levels for highway pavements are 6000, 9000, and 12000 pounds.

Number of Drops per Load Level

After the seating drop, it is common practice when testing asphalt pavements to apply multiple load drops for each load level at each station testing. ASTM D4694 recommends that if significant permanent deformation under the loading plate occurs, the FWD should be moved to a different position and the applied force should be reduced "until the permanent deformation is of no significance to the first test at a test location."

For concrete and asphalt-overlaid concrete pavements, there is generally little or no significant change in deflections between load drops at the same load level. Two drops per load level are sufficient for these pavement types, the second one serving as a safeguard against a deflection sensor malfunctioning.

Sensor Configuration

Different sensor configurations yield different backcalculation results. Specifically, two configuration issues that significantly influence the magnitudes of the modulus values obtained from backcalculation are the outer radius to which the deflection basin is measured, and whether or not the maximum deflection (d_0 measured at the center of the load plate) is used in the backcalculation.

The main reason for measuring as far out as possible is to obtain one or more deflections far enough away from the load plate to estimate of the subgrade modulus independent of the effects of the overlying pavement layers. In selecting the distance for the farthest measurement point, consideration should be given to the fact that deflections decrease with distance but measurement error remains essentially constant. Thus the influence of measurement error on backcalculation becomes greater at greater distances from the load plate.

Load Transfer Measurement

When testing on concrete or asphalt-overlaid concrete pavement, one of the transducers in front of the load plate can be moved behind the load plate to measure load transfer on both the approach and leave sides of transverse joints and/or cracks. Similarly, to facilitate measurement of longitudinal load transfer (e.g., across the lane/shoulder joint on a concrete pavement with a tied concrete shoulder), another transducer can be mounted to one side of the load plate. When using the SHRP configuration (0, 8, 12, 18, 24, 36, and 60 inches), the transducers at 8 and/or 18 inches can be moved for these load transfer measurement purposes. This leaves in place the transducers at 0, 12, 24, and 36 inches for backcalculation, plus a distant sensor (60 inches) for an independent estimate of the subgrade modulus. Load transfer measurement across the longitudinal centerline joint or other longitudinal joints between highway traffic lanes is not recommended, for safety reasons.

At any given joint or crack in a concrete pavement or asphalt-overlaid concrete pavement, it is very possible that the load transfer measurements on the approach and leave sides will be unequal, because the crack in the slab rarely propagates completely vertically. However, on any given project, it may or may not be true that one side has consistently and significantly lower load transfer than the other side. If paired *t* tests show a significant consistent difference, the lower of the two should be used to compute the mean load transfer for use in slab stress analysis. It should also be kept in perspective that deflection load transfers are usually measured over a narrow temperature range during just a few days or hours out of the year and that all other load transfer levels for other temperatures are estimated.

Load transfer measurement is one of the more time-consuming aspects of deflection testing. It requires the operator to carefully position the load plate and sensors across the joint or crack, using either cameras mounted under the FWD or the help of an assistant. Measuring load transfer on both the approach and leave sides of transverse joints significantly increases the total testing time, and therefore is not recommended unless a specific objective of the project is to investigate differences in approach side and leave side load transfer. Otherwise, it is recommended to measure only the approach side load transfer (with the load plate behind the joint or crack), because it is then not necessary to move a deflection transducer to behind the load plate.

Materials Sampling and Testing

Any rehabilitation strategies involving overlay options will require information about the existing pavement materials and subgrade, for purposes of overlay thickness design. Depending on the design procedure used, the information required may include:

- Thicknesses of the pavement layers,
- Condition of the pavement layer materials,
- Elastic moduli of the pavement layers, and/or
- Elastic modulus or k value of the subgrade.

The stiffnesses of the pavement layers and subgrade may be determined from nondestructive deflection testing, as described previously. Layer thicknesses and stiffnesses may also be determined from laboratory testing of materials samples, or in some cases, from field tests. Materials sampling and testing is described in this section. Subgrade stiffness (elastic modulus or k value) may also be estimated from correlations with other soil properties, as described subsequently.

Layer Materials and Thicknesses

The material types and layer thicknesses should be determined from inventory records before deflection testing is conducted. Coring to check layer thicknesses may be done before deflection testing, but it is preferable to do coring after deflection testing, so that any unanticipated changes in the deflection magnitudes can be investigated during the coring. It is not usually feasible to conduct coring and deflection testing simultaneously, because the coring operation is slower and cannot keep pace with the deflection testing operation.

Layer thicknesses are important to the analysis of the deflection data. If a sound, full-thickness core cannot be obtained for a layer of asphalt or concrete (because the material is extensively deteriorated), the thickness can usually still be measured by probing in the core hole for the underside of the layer. Obviously, this condition should be recorded. If there are no plans to perform laboratory testing on the pavement layer materials and only the layer thicknesses are needed, a small-diameter (e.g., half-inch) drill bit may be used to determine asphalt, concrete, and stabilized base layer thicknesses.

It is also useful to note from observations of cores whether or not the layers are bonded together. However, layers that come out unbonded in the core may not have been unbonded in place; it is conceivable that the layers were separated by torsion during the coring operation. Examination of the interface may indicate whether the layer samples were separated during coring or had been unbonded for some time.

Asphalt Concrete Resilient Modulus Testing

Diametral resilient modulus testing may be conducted on cores from asphalt concrete and asphalt-treated base layers. Guidelines for this test are given in ASTM D4123. The test method involves repetitive loading along one diameter of core, and then along the perpendicular diameter. Resilient modulus testing usually involves tests at three temperatures, each at one or more loading frequencies. The resilient modulus of the asphalt concrete is calculated as a function of the applied load, the thickness of the test sample, the measured recoverable horizontal deformation, and the Poisson's ratio (which may be calculated from the measured recoverable horizontal and vertical deformations, or assumed to be 0.35). The average resilient modulus is reported for each temperature, load duration, and load frequency used.

The elastic modulus of the asphalt concrete mix at any given temperature may be estimated using the equation developed by Witczak for use in the Asphalt Institute's MS-1 Design Manual.[27] This is a refinement of work originally done for the Asphalt Institute by Kallas and Shook.[28] It is considered highly reliable for dense-graded asphalt concrete mixes with gravel or crushed stone aggregates.[29] Witczak's equation can be reduced to a relationship between the asphalt concrete elastic modulus and the mix temperature for a particular loading frequency by measuring or assuming values for the asphalt cement and mix parameters in the equation. It should be noted, however, that the equation applies to new mixes. Asphalt concrete materials that have been in service for some time may have either a higher modulus (due to hardening of the asphalt) or a lower modulus (due to deterioration of the mix, from stripping or other causes) at any given temperature. Therefore, it is recommended that the results of diametral resilient modulus testing on cores at two or more temperatures be used to calibrate the above equation for the particular mix being evaluated.

It should also be remembered that the elastic modulus that an asphalt concrete mix exhibits in the field, i.e., under traffic loading or during nondestructive deflection testing, is typically about 2 to 2.5 times higher, at any given temperature, than the elastic modulus that would be measured for this same mix in the laboratory.[19] For example, a mix for which an elastic modulus of 400,000 psi is measured in the laboratory at 70°F may be expected to exhibit an elastic modulus of between about 800,000 and 1 million psi in the field at the same temperature. The reason for this is the difference between the load frequency in the laboratory test and the load frequency of nondestructive deflection testing, which simulates high-speed traffic loading. Laboratory diametral resilient modulus testing is typically conducted at frequency of about 1 to 2 Hz. The load duration of the falling weight deflectometer is about 25 to 30 milliseconds,[30] which corresponds to a loading frequency of between 15 and 20 Hz.

For a given set of asphalt concrete mix parameters, laboratory testing frequency, and deflectometer impulse load duration, Witczak's equation may be used to calculate the ratio of field modulus to laboratory modulus at any given temperature. Whether the laboratory modulus or field modulus is used in analysis depends on the input required for the analysis model being used. For example, the laboratory modulus is the correct input to asphalt concrete fatigue models developed from laboratory testing, while the field modulus is the correct input to fatigue models developed from full-scale field testing.

Asphalt Concrete Indirect Tension Testing

Indirect tension testing uses the same test setup as diametral resilient modulus testing but involves applying a single load, at a constant rate of deformation, to failure of the sample. Guidelines for this test are given in ASTM D 4123. The indirect tensile strength is calculated as a function of the applied load, the length of the sample, and the diameter of the sample. Indirect tensile strength testing is faster than diametral resilient modulus testing, but may be less useful: the two have not been demonstrated to correlate well.

Marshall Stability and Flow Testing

Marshall stability and flow testing may be conducted on asphalt concrete cores to determine the stability and flow of the mix. Guidelines for this test are given in ASTM D5581. The stability is the maximum load resistance in pounds that a specimen exhibits at 60°C. The flow is the strain (in units of 0.25 mm) measured during loading. The mix density, air voids, and maximum theoretical specific gravity must also be determined to relate the results of the stability-flow test to potential or observed problems such as excessive rutting.

Asphalt Extraction Testing

Extraction testing may be conducted on asphalt concrete and asphalt-treated base layer cores to separate the asphalt cement from the aggregate and subsequently determine the asphalt cement content and the aggregate gradation. The stiffness of the recovered asphalt cement may be determined from penetration or viscosity testing, as described in ASTM D2171. This testing is especially important if consideration is being given to recycling some or all of the existing asphalt materials.

Concrete Indirect Tenstion Testing

Indirect tensile testing may also be conducted on cores from concrete layers. A widely used correlation between concrete indirect tensile strength and third-point modulus of rupture is that developed by Hammitt.[31]

Soil Sampling

Samples of unbound base materials and subgrade materials may be obtained in the field for laboratory testing. Fine-grained soil samples may be obtained by split-spoon sampling, as described in ASTM D1586. Granular material samples may be obtained by augering or trenching.

Resilient Modulus Testing of Soils

Resilient modulus testing may be conducted on fine-grained and coarse-grained soil samples. Guidelines for laboratory resilient modulus testing are given in AASHTO T294-92. A compacted soil sample is placed in a triaxial test apparatus, subjected to an all-around confining pressure, and further subjected to repeated axial load, while the resulting vertical deformation is measured. The resilient modulus of the soil is calculated as the ratio of the deviator stress (the total vertical stress minus the all-around confining pressure) to the resilient strain (that portion of the total strain recovered when the load is removed).

Fine-grained soils tend to exhibit stress-softening behavior, that is, the resilient modulus decreases with increasing deviator stress. Coarse-grained soils, on the other hand, tend to exhibit stress-hardening behavior: higher resilient modulus at higher deviator stress levels. The resilient modulus that a material exhibits in the laboratory may be considerably lower than the resilient modulus that the same material exhibits in the field, due to differences in the magnitudes of deviator stress, all-around confining pressure, and loading rate. Field resilient modulus values for fine-grained soils, obtained by backcalculation from falling weight deflectometer deflections, have been reported in a number of studies to exceed laboratory resilient modulus values by factors between about three and five.[1] Less information is available about the relationship of field-to-lab resilient modulus for coarse-grained soils, but in general, field modulus values are expected to be higher than laboratory modulus values for these materials as well.

Whether the laboratory modulus or field modulus of the subgrade soil is used in analysis depends on the input required for the analysis model being used. For example, the original AASHO Road Test model for fexible pavement performance was calibrated to the laboratory resilient modulus of the soil at the AASHO Road Test site. Therefore, when using the 1993 AASHTO overlay procedure to determine the required asphalt overlay thickness for an in-service asphalt pavement, the appropriate input for the subgrade soil is the laboratory resilient modulus.

California Bearing Ratio Testing

The California Bearing Ratio (CBR) test is a simple laboratory test that measures the resistance of a soil sample to the penetration of a piston at a constant rate. Guidelines for CBR testing are given in AASHTO T 193. The CBR of the soil is the ratio, expressed as a percentage, of the load corresponding to a given penetration, to the load corresponding to the same penetration for a standard well-graded crushed stone.

Stabilometer Testing

The stabilometer test is a simple laboratory test that measures horizontal stress in a soil sample as a result of a constant vertical pressure. The resistance value (R value) is calculated as a function of the applied vertical pressure, the transmitted horizontal pressure, and the displacement of stabilometer fluid necessary to increase the horizontal pressure from 5 psi to 100 psi.

Soil Testing in the Field

The CBR test may also be conducted in the field, as described in AASHTO T 193. An efficient and inexpensive way to estimate the in-place CBR is with a Dynamic Cone Penetrometer (DCP). This device is a graduated rod with a metal cone on one end and a mass repeatedly lifted and dropped to drive the cone into the soil. The DCP's penetration rate (mm/blow) correlates well to CBR for fine-grained soils (CBR up to about 15 percent).

Plate load testing of subgrade soils is not commonly done because it is slow, labor-intensive, and in the case of existing pavements, requires removing segments of the surface and base layers. It is nonetheless the direct method for determining the static modulus of subgrade reaction (k value), a required input to concrete pavement overlay design procedures.

Guidelines for repetitive static plate load testing are given in ASTM D1195 and in AASHTO T221. In the repetitive test, the static elastic k value is calculated as the ratio of the applied pressure to the elastic deformation (the recoverable portion of the total deformation measured). Guidelines for nonrepetitive static plate load testing are given in ASTM D1196 and in AASHTO T222. In the nonrepetitive test, the pressure-deformation ratio at a deformation of 0.05 inch is considered to represent the static elastic k value. A 30-inch-diameter plate should be used to determine the static elastic k value from either repetitive or nonrepetitive plate load testing. Smaller-diameter plates will yield higher k values that are inconsistent with the subgrade response to full-size slab loading.[32]

The dynamic k value obtained by backcalculation from deflections measured on concrete slabs, using a falling weight deflectometer, is about double the static elastic k value obtained from plate load testing of the same soil.[33] This is due to the difference in the soil's response to dynamic loading and static loading.

Correlations have been developed to estimate soil k values as a function of CBR, density, and soil class.[34, 35, 36, 37, 38] Several of these correlations are summarized in Reference 41. Additional correlations between soil properties (gradation, density, moisture content), soil classification, CBR, DCP penetration rate, and resilient modulus, are given in the Illinois Department of Transportation's guidelines on subgrade inputs and subgrade stability requirements for local road pavement design.[39] More information on soil classification and soil properties is available from the Portland Cement Association[36] and the Asphalt Institute.[40]

Profile and Roughness Measurement

Roughness may be characterized by indices based on either the measured profile of the measured surface, or the output from a roughness meter installed in a vehicle. At the project level, roughness measurements can be useful in locating areas of excessive roughness, deciding whether or not a nonoverlay rehabilitation strategy should include some treatment for reducing roughness (such as an overlay or diamond grinding), and assessing the effectiveness of such treatments. In general, however, roughness measurement plays a larger role in network-level pavement management (i.e., identifying projects in need of maintenance or rehabilitation) than in project-level evaluation.

The measured profile may also be used to simultaneously produce, by simulation, the outputs of other roughness devices measuring devices as if those devices had been used to measure the surface. Simulation of vehicle responses from profile measurements is described in ASTM E1170. Devices that can be simulated include the BPR Roughometer, the CHLOE Profilometer, the Mays Ride Meter, the PCA Road Meter, and various straightedge devices.

Pavement profile measurement is described in ASTM E950. Profile measurement is most efficiently done using a high-speed non-contact profilometer such as the K. J. Law profilometer or the South Dakota profiler. Profile measurement may also be done using survey rod and level equipment or a Dipstick device. Descriptions of profile and response-type roughness measurement devices are given by Shahin.[14]

The International Roughness Index (IRI) is a roughness parameter obtained from a mathematical model applied to a measured profile. The model simulates a quarter-car (one wheel) system travelling at 80 km/hr. The IRI is computed as the cumulative movement of the suspension of the quarter-car system divided by the travelled distance. Detailed guidelines on accurate measurement of longitudinal pavement profile for the purpose of calculating International Roughness Index (IRI) are given in Reference 42.

Present serviceability index (PSI) may be estimated from International Roughness Index using equations developed by Hall and Correa,[43] derived from AASHO Road Test data.[44] Similar equations have been for estimation of present serviceabilty index have been developed by Dujisin and Arroyo,[45] and models for estimation of present serviceability rating (PSR) have been developed by Paterson,[46] Al-Omari and Darter,[47] and Gulen et al.[48]

Friction Measurement

Friction testing, like roughness testing, is less a project-level evaluation activity than a network-level pavement management activity. At the project level, friction measurements can be useful in deciding whether or not a nonoverlay rehabilitation strategy should include some treatment improving surface friction (such as an overlay, diamond grinding, or grooving), and assessing the effectiveness of such treatments. Friction testing may be done using testing wheels, in locked-wheel mode, slip mode, or yaw mode, or using smaller laboratory devices.

The friction parameter obtained from locked-wheel testing is the Skid Number, or 100 times the friction coefficient measured. This test method is described in ASTM E274. Slip mode testing involves measuring the change in angular wheel speed during braking of a free-rolling wheel. Yaw-mode testing involves turning the testing wheel (without braking) to some angle away from the direction of motion, and measuring the sideways friction factor.

Among the smaller devices available for measuring surface friction, the most commonly used is the British Portable Tester, the use of which is described in ASTM E303. Pavement surface texture can be measured by methods such as the sand patch method. Laser devices such as the TRRL texture meter may also be used to measure surface texture. Descriptions of these and other friction and texture measurement methods are given by Shahin.[14] Additional information on pavement friction is given in Reference 49.

Drainage Inspection

The most obvious signs of inadequate subsurface drainage will be notable during the distress survey: pumping of water and/or fines at transverse and/or longitudinal joints, blowholes along the lane/shoulder joint, and localized settlement of an asphalt concrete shoulder near blowholes. D-cracking in a concrete pavement may also indicate a drainage deficiency. A third major moisture-related problem is stripping in asphalt and asphalt-overlaid concrete pavements, which may be investigated by visual examination of cores after splitting.

The following additional indications of inadequate drainage should also be noted during the field survey:

- Standing water in the ditches,
- Cattails or other water-loving vegetation in the ditches,
- Inadequate height of subdrain outlets or daylighted base above the ditchline,
- Clogging or obstruction of subdrain outlets, or
- Clogging of daylighted base by soil and/or vegetation.

If visual observations suggest a significant drainage deficiency may exist, more intensive inspection may be conducted. The effectiveness of both longitudinal edgedrains and daylighted bases may be evaluated by applying water (either to the pavement surface or to the base through a core hole in the surface) and observing the outflow, or by observing the outflow during or immediately after a rainfall. Localized clogging, obstruction, and crushing of longitudinal edgedrain pipes can be investigated using video inspection equipment.

Other Nondestructive Testing

Ground-Penetrating Radar

Ground-penetrating radar (GPR) is used to estimate pavement layer thicknesses, joint deterioration, moisture contents in base layers, and stripping in asphalt concrete layers. Ground-penetrating radar has been tested on asphalt, concrete, and asphalt-overlaid concrete pavements, as well as bridge decks.

Short-pulse, ground-penetrating radar works on the principle of wave propagation and reflection and transmission of electromagnetic waves. A brief pulse of electromagnetic energy is directed into the pavement. Dielectric discontinuities in the pavement (for example, changes in material type, moisture content, or density) cause part of the incident wave to be reflected and part to be transmitted into the next layer. This reflected energy is recorded by devices at the surface, and analyzed to determine pavement properties (layer thicknesses, voids, moisture contents, etc.). Vehicles equipped with ground-penetrating radar equipment (radar systems, transducers, antennae, and on-board recording devices such as magnetic tapes, oscilloscopes, and computer hardware and software) operate at speeds from 3 to 70 mph, which may require a moving lane closure. Ground-penetrating radar testing is described in ASTM D4748.

The major advantages of ground-penetrating radar testing are its speed and accuracy. It continues to be the only technology that can provide meaningful subsurface information at close to highway speed. Its disadvantages include the complexity of the radar output and the lack of good software to convert the signals into information meaningful to pavement engineers. Current data analysis methods are labor intensive and require considerable expertise for interpretation of the raw data. Some coring is required with ground-penetrating radar, for calibration purposes.[50]

Infrared Thermography

Infrared thermography is used to locate reinforcing steel and detect concrete delaminations in reinforced concrete pavements. Infrared thermography has also been used to detect debonding at asphalt/concrete interfaces, and to measure temperature differentials in newly placed asphalt overlays. Developed initially for application to bridge deck inspection, infrared thermography has been used to survey pavements as well.

Infrared thermography is the process of detecting temperature differences associated with defective areas within a pavement. Various types of infrared scanners have been used to detect both delamination and debonding. Temperature differences indicative of defects, such as a thin delamination heating faster than the thicker, sound pavement around it, are detected by scanners and recorded on videotape. Often, real-image video recording equipment is mounted together with the scanner to record surface defects such as potholes and patches that may otherwise be interpreted incorrectly when viewed on the infrared output. The infrared scanning equipment can be van mounted and operated at speeds of 15 mph.

The major advantages of infrared thermography are its speed and accuracy, relative to destructive methods such as coring, for subsurface data collection. Disadvantages of infrared thermography include its sensitivity to non-pavement-related conditions such as time of day and recent weather conditions. Also, the two-dimensional output cannot indicate the depth of the distressed area. Perhaps the greatest practical disadvantage of infrared, however, is the complexity of the infrared outputs and video images.

Wave Propagation/Spectral Analysis

Wave propagation is a technique for monitoring the dispersion (change in velocity with frequency or wavelength) of surface waves in a pavement, to predict pavement condition. In a layered system, the dispersion of surface waves is indicative of the relative stiffnesses of distinct layers.

Surface waves may be produced using drop weight devices, vibratory devices, or strike hammers. This third method is employed in the testing technique known as spectral analysis of surface waves (SASW). This technique involves using a series of progressively larger hammers to produce waves of increasing wavelength, which tend to propagate through the deeper layers of a pavement. The waves generated in the pavement by the strike hammers, and their dispersions, are monitored by two transducers acting as receivers. The data are collected by a spectral signal analyzer and passed to a computer for processing. The wave velocities can be transformed into representations of modulus versus depth.

Spectral analysis of surface waves has been applied to asphalt, concrete, and asphalt-overlaid concrete pavements, over both fine-grained and coarse-grained subgrades. SASW analysis results have been shown to compare well with backcalculation results from deflection analysis. An advantage of SASW over deflection-based backcalculation is that it is capable of predicting pavement layer moduli without advance knowledge of layer thicknesses or material types. A disadvantage of SASW is the difficulty and time involved in data collection and interpretation. More automated data acquisition and processing methods are needed to make this testing technique more practical.[50]

Sonic/Ultrasonic/Seismic Wave Analysis

The use of sonic, ultrasonic, and seismic waves to evaluate internal concrete conditions has also been applied to pavements. These techniques involve emission of stress waves from a source (a transducer or high-speed, low-mass projectile) at the pavement surface, and detection of direct or refracted wave characteristics by very precise sensors. Compression and shear waves are used to determine modulus and strength. Horizontal and seismic waves are used to detect voids beneath a concrete slab. Analysis of the reflected wave data can provide information on pavement layer thicknesses and delaminations.[50] The usefulness, speed, accuracy, advantages and disadvantages sonic/ultrasonic/seismic wave analysis for pavement evaluation are not as well established as they are for ground-penetrating radar, infrared thermography, and spectral analysis of surface waves.

PROJECT-LEVEL PAVEMENT EVALUATION

The purpose of project-level pavement evaluation is to assess the current condition of the pavement, identify the key types of deterioration present, identify deficiencies that must be addressed by rehabilitation, and identify uniform sections for rehabilitation design and construction over the project length.

Distress Evaluation

Rehabilitation of a pavement is most likely to be successful – that is, provide satisfactory performance and cost-effectiveness – if it is selected on the basis of knowledge of the types of distresses occurring in the pavement, and understanding of the causes for those distresses. Many distresses have more than one possible cause. It is important to study the distresses observed in the field survey in order to correctly identify the one or more mechanisms causing the distress observed.

The well-known Pavement Condition Index (PCI) procedure for calculation of a numerical index of pavement condition from distress data, on a scale of 0 to 100, was developed by the U.S. Army Corps of Engineers for application to airfields and to local roads and streets (i.e., military bases).[5] The Corps of Engineers' PCI procedure does not address highway pavements. Some State DOTs, such as Ohio and Washington, have developed highway pavement condition index procedures modeled on the PCI procedure.[51,52] PCI-type procedures, however, are more useful in network-level pavement management – i.e., rehabilitation programming – than for project-level rehabilitation strategy selection and rehabilitation design.

Structural Evaluation

Structural evaluation involves examination of the collected distress, deflection, materials, soils, and drainage information for the following purposes:

- Assessment of the current structural condition of the pavement, that is, how much structural damage has been done to the pavement so far; and
- Assessment of the remaining structural life of the pavement, that is, how many more loadings it can support before failure.

The results of a structural evaluation are also used in dividing a project into structurally uniform sections, identifying areas requiring localized repair, selecting one or more appropriate alternatives for structural improvement, and developing preliminary designs for these alternatives.

Asphalt Pavement Structural Evaluation

Structural evaluation of asphalt pavements may be accomplished using condition data only, deflection measurements only, condition plus deflection data, or traffic data only.

As a general rule, an asphalt pavement is considered to require a structural improvement when 50 percent of the wheelpath area (equivalent to about 10 percent of the total area) of the outer traffic lane has medium- to high-severity alligator cracking.[53] A critical rutting level of one half inch is often cited as indicative of a need for structural improvement. However, rutting may have causes related not only to the load-bearing capacity of the pavement layers, but rather the stability of the mix, so the cause of rutting should be examined before deciding whether or not a structural improvement is the appropriate remedy.

An example of asphalt pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of asphalt pavements, also called the component analysis method.[1] This approach involves calculating an "effective Structural Number" using pavement layer material structural coefficients less than or equal to those that would be assigned to new materials, depending on the types, extents, and severities of load-related distress present.

Examples of deflection-based approaches to structural evaluation of asphalt pavements are the 1993 AASHTO Guide´s deflection method for overlay design,[1] the Asphalt Institute´s overlay design procedure,[4] and Thompson´s ε-AUPP algorithm,[54] in which asphalt pavement strain (ε) is predicted as a function of area under the pavement profile (AUPP), a deflection basin curvature parameter. Deflection analysis may be used in combination with condition-based overlay design, either solely for the purpose of estimating the resilient modulus of the subgrade soil (e.g., the 1993 AASHTO Guide method), or for the purpose of backcalculating the elastic moduli of other pavement layers, and using these elastic moduli in mechanistic-empirical models for fatigue, rutting, and thermal cracking.

A traffic-based approach to structural design of asphalt pavements is the 1993 AASHTO Guide´s remaining life method of overlay design, in which the structural condition of the existing pavement is determined as a function of the ratio of past ESALs to allowable ESALs. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

Concrete Pavement Structural Evaluation

Structural evaluation of concrete pavements may be accomplished using condition data only, condition plus deflection data, or traffic data only. An example of concrete pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of concrete pavements.[1] This approach involves determining an "effective slab thickness" less than or equal to the actual slab thickness, depending on the types, extents, and severities of load-related distress present.

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[53] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, a jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[53] Low-severity transverse cracks are not considered structural distress.

As a general rule, continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[53]

No purely deflection-based approaches to structural evaluation exist for concrete pavements, but some methods exist for using deflection analysis in characterizing the structural condition of a concrete slab. One such example is Rolling´s approach[26] to assigning an effective modulus to the concrete slab as a function of its Structural Condition Index (SCI), the structural component of the Pavement Condition Index (PCI) derived from the rating system developed by Shahin et al.[55]

Structural evaluation of concrete pavements may be accomplished using deflection analysis in conjunction with condition survey results. An example is the method developed by Hall et al.[50] to assign a qualitative rating to the structural integrity of a concrete slab, based on the mean backcalculated concrete elastic modulus, the percentage of backcalculated modulus values less than 2 million psi, the extent and severity of durability problems, the percentage of the outer traffic lane area repaired, and the percentage of the outer traffic lane area needing new repairs.

Deflection analysis may also be used in combination with condition-based structural characterization of concrete pavements for the purpose of estimating inputs to overlay design (e.g., modulus of subgrade reaction, elastic modulus of the concrete, etc., as in the 1993 AASHTO Guide method), or for the purpose of backcalculating the subgrade k value and concrete slab modulus, and using these elastic moduli in mechanistic-empirical distress models.

A traffic-based approach to structural evaluation of concrete pavements is the 1993 AASHTO Guide´s remaining life method of overlay design, in which the structural condition of the existing pavement is determined as a function of the ratio of past ESALs to allowable ESALs. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

Asphalt-Overlaid Concrete Pavement Structural Evaluation

Structural evaluation of asphalt-overlaid concrete pavements may be accomplished using condition data only, or condition plus deflection data. An example of asphalt-overlaid concrete pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of asphalt-overlaid concrete pavements. This approach involves determining an "effective slab thickness," depending on the types, extents, and severities of load-related distress present.

As a general rule, an asphalt-overlaid jointed concrete pavement (plain or reinforced) is considered to require a structural improvement when it has 75 or more medium- or high-severity reflected cracks, joints, and or patches per mile in the outer traffic lane.[19] An asphalt-overlaid continuously reinforced concrete pavement is considered to require a structural improvement when it has 10 medium- or high-severity reflected cracks, punchouts, or failed patches per mile in the outer traffic lane.[19]

No purely deflection-based approaches to structural evaluation exist for asphalt-overlaid concrete pavements. Structural evaluation of asphalt-overlaid concrete pavements may be accomplished using deflection analysis in conjunction with condition survey results. An example is the method developed by Hall et al.[50] No purely traffic-based approach exists for structural evaluation of asphalt-overlaid concrete pavements. The traffic-based remaining life methods provided for asphalt and bare concrete pavements in the 1993 AASHTO Guide are not directly applicable to existing asphalt-overlaid concrete pavements, because the original AASHO Road Test performance models are not directly applicable to this type of pavement.

Functional Evaluation

Functional evaluation involves comparing the pavement´s measured roughness, skid resistance, and rut depth (in the cases of asphalt and asphalt-overlaid concrete pavements) to the agency's standards for these functional parameters.

The 1993 AASHTO Guide recommends that, for the purposes of pavement design, the minimum allowable serviceability be selected as a function of the location (urban, rural) and functional class of the roadway. Hall et al.[56] recommend minimum serviceabilty levels of 3.0, 2.5, and 2.0, for ADT levels greater than 10,000, between 3,000 and 10,000, and less than 3,000 respectively. The American Concrete Pavement Association[57] recommends the same trigger levels and identifies the corresponding California Profilograph profile index levels as 60, 80, and 100 respectively.

As a general rule, faulting is considered to require correction in concrete pavements when it reaches an average level of 0.125 inch in jointed plain concrete pavement or 0.25 inch in jointed reinforced concrete pavement. For the purpose of assessing whether faulting has yet reached an unacceptable level, faulting measured at both joints and transverse cracks should be included in calculating the average.

Drainage Evaluation

The evaluation of drainage adequacy in existing pavements has in the past been largely a cursory and qualitative exercise, limited to observations on whether or not water seemed to be flowing from drainage outlets. Some past and current research advocates a more objective assessment of (a) whether or not the climate is such that the expected inflow into the pavement is greater than the drainage capacity of the pavement base and natural subgrade, and (b) whether or not the subdrainage features, if present, are adequate to accommodate any excess inflow that might occur.[58,59]

Identification of Uniform Sections

Sections within the project that are uniform with respect to design, geometry, materials, structural capacity, soils, distress, traffic, drainage, etc., should be identified on the basis of the collected inventory, materials, distress, deflection, and other data. Appendix J of the 1993 AASHTO Guide presents a method for delineating pavement sections that are statistically homogeneous with respect to one parameter, e.g., maximum deflection.

The simultaneous consideration of several inventory, distress, and deflection parameters could conceivably result in the division of the project into several short sections. The shortest section length for which rehabilitation can realistically be designed and constructed, e.g., one half mile, should be determined. Uniform sections should be combined as necessary to make rehabilitation design sections of at least this minimum length, and the representative conditions over each of these sections should be quantified.

CONCLUSIONS

The *Guide for Selection of Pavement Rehabilitation Strategies* provides a step-by-step process and practical guidance for project-level evaluation and rehabilitation strategy selection for in-service pavements. The *Guide* was developed after a review of the pavement rehabilitation practices of State DOTs, and the literature available on pavement evaluation, rehabilitation techniques, and selection of rehabilitation strategies.

Rehabilitation of a pavement is most likely to be successful if it is selected on the basis of knowledge of the types of distresses occurring in the pavement, and understanding of the causes for those distresses. Although just about any rehabilitation technique can be applied at any time, the goal of rehabilitation strategy selection is to identify the techniques that are best suited to the types of distress present. The individual techniques selected to address the distresses and structural and functional deficiencies observed must be combined into one or more feasible rehabilitation strategy alternatives, developed in sufficient detail that their performance and costs may be confidently estimated.

REFERENCES

1. American Association of State Highway and Transportation Officials*, Guide for Design of Pavement Structures*, Washington, D. C., 1993.

2. American Association of State Highway and Transportation Officials, *Supplement to the Guide for Design of Pavement Structures, Part II – Rigid Pavement Design and Rigid Pavement Joint Design*, Washington, D. C., 1998.

3. The Asphalt Institute, *Thickness Design – Asphalt Pavements for Highways and Streets*, Manual Series No. MS-1, Lexington, KY, 1991.

4. The Asphalt Institute, *Asphalt Overlays for Highway and Street Rehabilitation*, Manual Series No. MS-17, Lexington, KY, 1999.

5. Packard, R. G, *Thickness Design for Concrete Highway and Street Pavements*, Engineering Bulletin EB109-01P, Portland Cement Association, Skokie, IL, 1984.

6. Tayabji, S. D. and Okamoto, P. A., "Thickness Design of Concrete Resurfacing," *Proceedings*, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafeyette, IN, 1985.

7. Federal Highway Administration, *Traffic Monitoring Guide*, 1985.

8. French, A. and Soloman, D., *Traffic Data Collection and Analysis: Methods and Procedures*, NCHRP Synthesis No. 130, Transportation Research Board, Washington, D.C., 1986.

9. Cunagin, W. D., *Use of Weigh-in-Motion Systems for Data Collection and Enforcement*, NCHRP Synthesis No. 124, Transportation Research Board, Washington, D.C., 1986.

10. Highway Research Board, "The AASHO Road Test, Proceedings of a Conference Held May 16-18, 1962, St. Louis, Missouri, Special Report 73, 1962.

11. Miller, J. S., Rogers, R. B., and Rada, G. R*., Distress Identification Manual for the Long-Term Pavement Performance Project*, third edition, 1993.

12. Smith, R. E., M. I. Darter, and S. M. Herrin, *Highway Pavement Distress Identification Manual*, FHWA Contract No. DOT-FH-11-9175 and NCHRP Project No. 1-19, 1979.

13. Shahin, M. Y. and S. D. Kohn*, Pavement Maintenance Management for Roads and Parking Lots,* Construction Engineering Research Laboratory Technical Report M-294, U. S. Army Corps of Engineers, 1981.

14. Shahin, M. Y., *Pavement Management for Airports, Roads, and Parking Lots*, copyright Chapman and Hall, 1994.

15. National Cooperative Highway Research Program, *Current Practices in Determining Pavement Condition*, NCHRP Synthesis No. 203, Transportation Research Board, Washington, D.C., 1994.

16. Federal Highway Administration, *Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements*, Report No. FHWA/RD-83/097, 1984.

17. Hudson, W. R., Elkins, G. E., Uddin, W., and Reilley, K.T., *Evaluation of Pavement Deflection Measuring Equipment*, Report No. FHWA-TS-87-208, Federal Highway Administration, 1987.

18. National Highway Institute, *Techniques for Pavement Rehabilitation*, sixth edition, 1998.

19. Hall, K. T., *Performance, Evaluation, and Rehabilitation of Asphalt-Overlaid Concrete Pavements*, Ph.D. thesis, University of Illinois at Urbana-Champaign, 1991.

20. Southgate, H. F., *An Evaluation of Temperature Distribution within Asphalt Pavement and its Relationship to Pavement Deflection*, Kentucky Department of Highways, Research Report KYHPR-64-20, 1968.

21. Shell International Petroleum Company, *Shell Pavement Design Manual*, London, England, 1978.

22. Hoffman, M. S. and M. R. Thompson*, Mechanistic Interpretation of Nondestructive Pavement Testing Deflections*, Illinois Cooperative Highway Research Series No. 190, University of Illinois at Urbana-Champaign, 1981.

23. Lukanen, E. O., Stubstadt, R., and Briggs, R*., Temperature Predictions and Adjustment Factors for Asphalt Pavements*, Federal Highway Administration Report No. FHWA-RD-98-085, 1999.

24. Crovetti, J. A., *Design and Evaluation of Jointed Concrete Pavement Systems Incorporating Free Draining Base Layers*, Ph.D. thesis, University of Illinois at Urbana-Champaign, 1994.

25. Crovetti, J. A. and Crovetti, M. R. T., "Evaluation of Support Conditions Under Jointed Concrete Pavement Slabs," *Nondestructive Testing of Pavements and Backcalculation of Moduli*, ASTM STP 1198, H. L. Von Quintus, A. J. Bush Ill, and G. Y Biladi, editors, 1994.

26. Rollings, R. S., *Design of Rigid Overlays for Airfield Pavements*, Ph.D. thesis, University of Maryland, 1987.

27. The Asphalt Institute, *Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition*, Research Report 82-2, 1982.

28. Kallas, B. F. and Shook, J. F., "Factors Influencing Dynamic Modulus of Asphalt Concrete," *Proceedings of the Association of Asphalt Paving Technologists*, Volume 38, 1949.

29. Miller, J. S., Uzan, J., and Witczak, M. W., "Modification of the Asphalt Institute Bituminous Mix Modulus Predictive Equation," *Transportation Research Record* No. 911, 1983.

30. Smith, R. E. and R. L. Lytton, *Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements*, Federal Highway Administration Report No. FHWA/RD-83/097, 1984.

31. Hammitt, G. M., II, *Concrete Strength Relationships*, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1971.

32. Darter, M. I., Hall, K., T., and Kuo, C.-M., *Support Under Concrete Pavements*, NCHRP Report No 372, Transportation Research Board, Washington, D.C., 1995.

33. Hall, K. T., Darter, M. I., Hoerner, T. E. and Khazanovich, L., *LTPP Data Analysis Phase I: Validation of Guidelines for K-Value Selection and Concrete Pavement Performance Prediction*, Federal Highway Administration Report No. FHWA-RD-96-168, Washington, D.C., 1997.

34. Middlebrooks, T. A. and Bertram, G. E., "Soils Tests for Design of Runway Pavements," *Proceedings of the Highway Research Board*, Washington, D. C., 1942.

35. U. S. Army, *Rigid Airfield Pavements*, Engineering and Design Manual EM 1110-45-303, Department of the Army, Washington, D. C., 1958.

36. Portland Cement Association, *PCA Soil Primer*, Engineering Bulletin EB007.05S, Skokie, Illinois, 1966.

37. Packard, R. G., *Thickness Design for Concrete Pavement*, Portland Cement Association Engineering Bulletin EB050.03P, Skokie, Illinois, 1973.

38. Darter, M. I., *Design of Zero-Maintenance Plain Jointed Concrete Pavement, Volume 1 – Development of Design Procedures*, Federal Highway Administration Report No. FHWA-RD-77-111, 1977.

39. Illinois Department of Transportation, *Pavement Design Procedures*, Springfield, Illinois, 1995.

40. The Asphalt Institute, *Soils Manual for the Design of Asphalt Pavement Structures*, MS-10, Lexington, Kentucky, 1993.

41. Hall, K. T., Darter, M. I., and Kuo, C.-M., "Improved Methods for Selection of K Value for Concrete Pavement Design," *Transportation Research Record* No. 1505, 1995.

42. Karamihas, S. M., Gillespie, T. D., Perera, R. W., and Kohn, S. D., *Guidelines for Longitudinal Pavement Profile Measurement*, NCHRP Report No. 434, Transportation Research Board, Washington, D.C., 1999.

43. Hall, K. T. and Correa, C. E., "Estimation of Present Serviceability Index from International Roughness Index," *Transportation Research Record* No. 1655, 1999.

44. Highway Research Board, *The AASHO Road Test, Report 5, Pavement Research*, Special Report 61E, 1962.

45. Dujisin, D. and Arroyo, A., *Desarrollo de una Relación Índice de Serviciabilidad (p) – Índice de Rugosidad Internacional (IRI)*, Comisión de Diseño y Evaluación de Pavimentos, Cámera Chilena de la Construcción, Santiago, Chile, 1995.

46. Paterson, W. D. O., *Road Deterioration and Maintenance Effects*, The World Bank, Washington, D.C., 1987.

47. Al-Omari, B. and Darter, M. I., "Relationships Between International Roughness Index and Present Serviceability Rating," *Transportation Research Record* No. 1435, 1994.

48. Gulen, S., Woods., R., Weaver, J., and Anderson, V. L., "Correlation of Present Serviceability Ratings with International Roughness Index," *Transportation Research Record* No. 1435, 1994.

49. National Cooperative Highway Research Program, *Evaluation of Pavement Friction Characteristics*, Synthesis No. 292, 2000.

50. Hall, K. T., Darter, M. I., and Steele, D. A., *Guidelines for Evaluation of Asphalt-Overlaid Concrete Pavements*, Illinois Highway Research Report IHR 532-3, University of Illinois at Urbana-Champaign and Illinois Department of Transportation, Report No. FHWA-IL-UI-246, 1995.

51. Saraf, C. L., *Pavement Condition Rating System – Review of PCR Methodology*, Ohio Department of Transportation, Report No. FHWA/OH-99/004, 1999.

52. Pierce, L. M. and Mahoney, J. P., "Description of Rehabilitation Scoping Process Used in Washington State Pavement Management System," *Transportation Research Record* No. 1524, 1996.

53. Hall, K. T. and Darter, M. I., *Structural Rehabilitation Strategies for Jointed Concrete Pavements, Volume IV – Guidelines for the Selection of Rehabilitation Alternatives*, Federal Highway Administration Report No. FHWA-RD-89-14, 1990.

54. Thompson, M. R., "Hot-Mix Asphalt Overlay Design Concepts for Rubblized Portland Cement Concrete Pavements, *Transportation Research Record* No. 1684, 1999.

55. Shahin, M. Y., M. I. Darter, and S. D. Kohn, *Development of a Pavement Maintenance Management System, Volume I: Airfield Pavement Condition Rating*, U. S. Air Force Civil Engineering Center, Technical Report No. AFCEC-TR-76-27, 1976.

56. Hall, K. T., Connor, J. M., Darter, M. I., and Carpenter, S. H., *Rehabilitation of Concrete Pavements, Volume III – Concrete Pavement Evaluation and Rehabilitation System*, Federal Highway Administration Report No. FHWA-RD-88-073, 1989.

57. American Concrete Pavement Association, *Pavement Rehabilitation Strategy Selection,* Technical Bulletin TB-015.P, Skokie, Illinois, 1993.

58. Wyatt,T. R. and Macari, E. J., "Effectiveness Analsysis of Subsurface Drainage Features Based on Design Adequacy," *Transportation Resaerch Board* No. 1709, 2000.

59. Carpenter, S. H., Darter, M. I., Dempsey, B. J., and Herrin, S., *A Pavement Moisture-Accelerated Distress (MAD) Identification System*, Report No. FHWA/RD-81/079, 1981.

60. Smith, K. D., Peshkin, D. G., Darter, M. I., Mueller, A. L., and Carpenter, S. H., *Performance of Jointed Concrete Pavements, Volume V – Appendix B, Data Collection and Analysis Procedure*s, Federal Highway Administration Report No. FHWA-RD-89-140, 1990.

61. Mahoney, J., Lee, S. W., Jackson, N., and Newcomb, D., Mechanistic-Based Overlay Design Procedures for Washington State Flexible Pavements," Washington StateDOT Report WA-rd-170-1, 1989.

62. Nevada Department of Transportation, *Pavement Structural Design and Policy Manual*, Carson City, Nevada, 1996.

63. Sebaaly, P. E., Hand, A., Epps, J., and Bosch, C., "Nevada's Approach to Pavement Management," *Transportation Research Record* No. 1524, 1996.

64. Mellinger, F. M., "Structural Design of Concrete Overlays," *Journal of the American Concrete Institute*, Volume 60, No. 2, February 1963.

65. Departments of the Army and Air Force, *Rigid Pavements for Airfields Other Than Army*, Army Technical Manual 5-824-3, Air Force Manual 88-6, Chapter 3,1979.

66. Moulton, L. K., "Higway Subdrainage Design," Federal Highway Administration Report No. FHWA-TS-80-224, 1980.