Independent Review of the Mechanistic Empirical Pavement Design Guide and Software

DETAILS

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INDEPENDENT REVIEW OF THE *MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE* AND SOFTWARE

This digest summarizes key findings from NCHRP Project 1-40A, conducted by three consultant teams headed by Professors Marshall Thompson, Ernest Barenberg, and Stephen Brown. Part I of the digest was prepared by Stephen F. Brown, Scott Wilson Pavement Engineering, Ltd.; Part II was prepared by Michael M. Darter and Gregg Larson, Applied Research Associates, Inc., and Matthew Witczak and Mohamed El-Basyouny, Arizona State University.

CONTENTS

NCHRP Project 1-40A was an independent, comprehensive review of the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) and companion software Version 0.7 delivered under NCHRP Project 1-37A in June 2004. The project was carried out by consultant teams in four areas: new hot-mix asphalt (HMA) pavement design; new Portland cement concrete (PCC) pavement design; composite pavement design and design reliability; and low-volume road pavement design. The review began in August 2004; interim reports were made to the Project 1-40 panel several times in 2004 and 2005, and each team's findings and conclusions were reported at a panel meeting in December 2005.

The project was successful and the panel used its interim and final results to direct the development of the new software Versions 0.8 (released November 2005) and 0.900 (released July 2006) by the NCHRP Project 1-40D research team.

Part I of this digest summarizes the findings, conclusions, and recommendations of the independent review of new HMA pavement design, new PCC pavement design, composite pavement design, and design reliability (the results of the review of low-volume road pavement design will be reported in a future RRD). Part II tabulates the responses of the Project 1-40D research team to the essential, high-priority recommendations of the independent reviewers with respect to corrections and improvements to the MEPDG software, which represents the day-to-day implementation of the design guide itself. Most of these recommendations were successfully incorporated in software Versions 0.8 and 0.900, yielding a stable, robust, and accurate tool for pavement design.

PART I

1 BACKGROUND

Scott Wilson Pavement Engineering, Ltd. (SWPE) was engaged by NCHRP under contract 1-40A, along with two other teams (see Table 1), to conduct an independent engineering review of the new mechanistic-empirical (M-E) pavement design guide (the Guide) and associated software (designated Version 0.7, July 2004) developed under NCHRP Project 1-37A

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

TABLE 1 Review team leaders

Team Leader	Key Tasks	NCHRP Contract
Professor S. F. Brown	Pavement rehabilitation, design reliability, executive summary report	1-40A(03)
Professor M. R. Thompson	New flexible pavement design	1-40A(01)
Professor E. J. Barenberg	New rigid pavement design	1-40A(02)

between 1998 and 2004. This was colloquially known as the "AASHTO 2002 Design Guide," but it was not completed by 2002 and has not, to date, been adopted by AASHTO; it is now termed the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) and software. These independent third-party reports were conducted at the request of the, then, AASHTO Joint Task Force on Pavements. The detailed brief for the reviewers is shown in Appendix A and the membership of each of the review teams in Appendix B.

In conducting the review, the engineering basis for each aspect of design and the associated computation procedures were assessed. Comments have also been made on the guidance given to potential users of the Guide. Three separate detailed reports, supported by appendixes, have been presented to the NCHRP Panel (*1–3*) and these form the background to this report, which summarizes the major points and recommendations.

The raw material for this review consisted of the following items:

- The final report of the 1-37A team (*4*) and
- The Design Guide software (Version 0.7).

Additional information was readily made available by the NCHRP Senior Program Officer on request. The approach taken to the various tasks was to study the reports, use the software, conduct discussions among the review teams, and absorb feedback from the NCHRP panel, following presentations at meetings and in response to interim reports. In addition, experienced pavement engineers in the United States were consulted for their views about the Guide and how their DOTs intend to implement it locally.

During the work, the teams became increasingly conscious of the very large parallel research effort—both in progress and being planned—to augment and extend the work done by the 1-37A team. Some of this work was in the form of NCHRP contracts and some was through the FHWA Implementation Group's activities with the state DOTs. Presentations at panel progress meetings provided some insights into this additional work, but, given that the work was incomplete, it was only used as background to the review. However, the initial work reported on contract 1-40B, dealing with local calibration of the Guide, was influential in identifying both problems with the software and the limitations of the original national calibrations carried out by the 1-37A team. The panel decided, on the basis of this work, to curtail the sensitivity analyses for flexible and rehabilitated pavements that were being attempted by the review teams using the software. It also became apparent that the available version of the software did not incorporate the latest information from the 1-37A flexible pavement team. It follows, therefore, that this review was not able to assess the software in as much detail as had originally been planned, but observations on the available version are given in Section 7.

The technical developments reported by the 1-37A team are substantial and are presented in detailed reports supported by appendixes (*4*). The distress prediction procedures bring together for the first time in a single computer program several important aspects of pavement performance that have previously only existed in stand-alone modules. In particular, procedures have combined environmental prediction with structural analysis and distress computation. Hence, a framework has been created for incorporating future research results, some of which will arise from NCHRP projects.

The general practical guidance provided for pavement engineers in the 1-37A reports is, overall, highly detailed. However, the documents in their present form are not user-friendly and will require improvement if they are to help busy engineers. The documentation and the software must be compatible and consistent, and the same unit of measurement should be used throughout.

A further preliminary observation, from the preamble to the 1-37A report (*4*), is that the brief was ". . . development of a design procedure based primarily on existing technology." This is an important point in presenting this evaluation. Since the 1-37A work began in 1998, there had been substantial progress in pavement engineering research, the findings of which were not considered for incorporation in the Guide.

The review team also noted the remarks made by the 1-37A authors in the preamble to their report, in which the authors present their views about the strengths and weaknesses of their work and the further research and development activity that they consider desirable. The 1-37A authors have been engaged by NCHRP to conduct some of this additional work in parallel with this review. The key issues to be addressed are

- Difficulties experienced with calibration of the design models against actual pavement behavior in the field, because of limitations with the Long-Term Pavement Performance (LTPP) database and the related difficulties in dealing with design reliability;
- Problems encountered in developing the software;
- The approximate nature of some of the distress prediction models, which have been included as "place holders" for improved versions to be introduced in future;
- The need for enhanced application of finite element analysis to deal with the non-linear properties of the lower layers in flexible pavements; and
- The need for procedures for designing concrete pavements and overlays of less than 6-in. slab thickness.

2 THE DESIGN PROCESS

Figure 1, taken from the 1-37A report (*4*), summarizes the design process used by the Guide software. The design process has three stages:

1. Evaluation (the data input stage),
2. Analysis (the complex distress prediction stage), and
3. Strategy selection (the design decision stage).

A new philosophy has been adopted: applying mechanistic principles to carry out very detailed distress development computations over the design life of the pavement and incorporation of these procedures within a comprehensive piece of computer

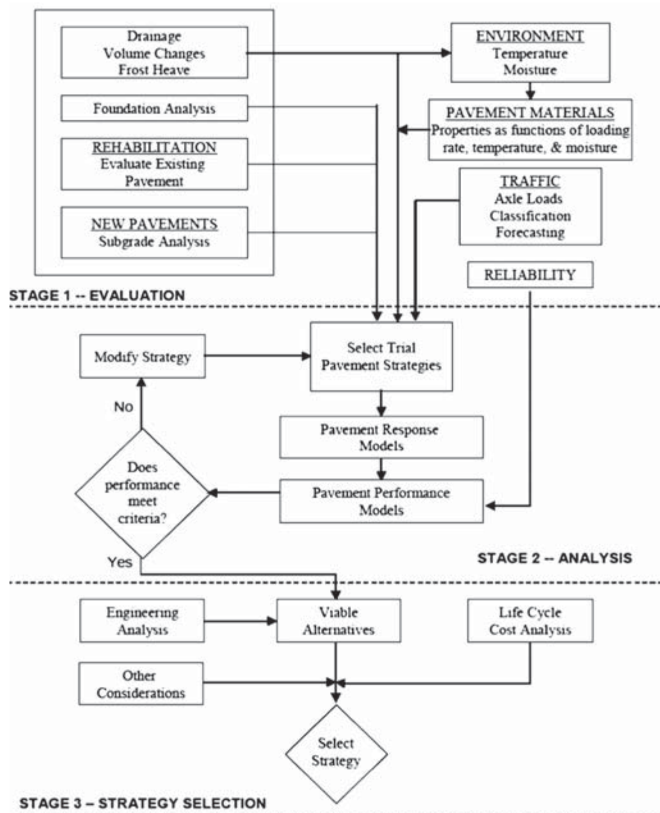

Figure 1 Flow Diagram of Design Procedure (after ARA, *4*)

software. In addition, empirical predictions of riding quality (smoothness) are also carried out using the International Roughness Index (IRI) as the quantitative parameter. The user must run the software repeatedly for a single design. Hence, the previous familiar, well-defined procedures, involving the direct determination of required layer thicknesses for a pavement to accommodate specified traffic, materials, environmental conditions, and performance requirements over a particular design life have been changed. For an engineer to design a pavement, it is necessary to assume an initial structure, compute the distress development over the required design life using the software, adjust the initial design if this exceeds acceptable levels, re-compute the distresses, and continue in this way until a satisfactory design is achieved.

Three levels of design are available; the user can select the one that best suits the level of input detail available. Level 1 requires extensive, detailed input, including data from the laboratory testing of materials, while Level 3 relies on empirical relationships between easily obtainable parameters and those required for the design computations. Level 2 requirements fall between these two extremes. For new pavement design, the details required for Levels 1 and 2 are not generally available at the design stage of a project, given that the actual materials may not be identified until construction is about to commence. These comments apply to the imported materials, but not the subgrade soils. Even so, the properties of soils and granular materials are difficult to determine in the laboratory in a way that reproduces the field situation. By contrast, for an existing pavement, field testing, particularly using the Falling Weight Deflectometer (FWD), can be carried out to determine the in situ properties of materials. In addition, for bound layers, cores can be cut to provide appropriate laboratory test specimens.

The first stage of the design process involves entering all the input data, which, for pavement rehabilitation design, includes all the field and laboratory testing associated with structural evaluation of the existing pavement. The first stage also embraces input for traffic loading and for the Extended Integrated Climatic Model (EICM), both of which represent major advances relative to previous design guides. A key change in the Guide relative to earlier versions is the move from the concept of Equivalent Single Axle Loads (ESALs). This was based on very old data from the AASHO Road Test (*5*), the conditions of which differed significantly from those on modern highways, and application of the "4th Power Law" to determine the relative damaging potential of different wheel loads. The availability of detailed traffic load spectra and the ability to perform numerous pavement structural analyses has led to this change. Consequently, more accurate distress predictions can be obtained using the actual traffic load spectra.

The EICM, developed from earlier work (*6–8*), has been fully incorporated in the Guide software but its detailed application, which is central to the distress prediction computations, requires many previously unconsidered material properties, many of which would be difficult to measure in a practical situation, so extensive reliance has to be placed on empirical methods of estimation. Use of the EICM allows the prediction of temperature and moisture conditions at all depths in the pavement throughout its design life. Given that the mechanical properties of materials and the development of distress are strongly influenced by these two variables, the software offers clear benefits.

The definition of design reliability has changed since the previous AASHTO Guide (*9*) to reflect the use of distress prediction techniques. Reliability used to be defined as the probability that the actual number of ESALs to a terminal serviceability would be less than the predicted number. In the new Guide, reliability is defined as the probability that a particular distress, such as fatigue cracking, will be less than a selected critical value.

The Guide software incorporates structural analysis subroutines as an essential part of the design process in Stage 2. Finite element analysis has been used for rigid pavements and the process has been effectively sped up through the application of neural networks. Linear elastic layered system analysis is used and the program (JULEA) has been incorporated for flexible pavements. Finite element analysis is available for dealing with the non-linear resilient properties of soils and granular materials at Level 1. Given the vital role of back-analysis in pavement evaluation for rehabilitation design, which requires that non-linearity be considered, it is difficult to see why the program does include a suitable subroutine. Indeed, very little is said about this key aspect of rehabilitation design.

A particular level of reliability can be specified for each distress mode and for IRI, which has replaced the Present Serviceability Index (PSI) as the measure of pavement smoothness. The inclusion of IRI, as a measure of riding quality in the Guide, does not jibe with the analytical (mechanistic) approach to design. It is not possible to compute IRI using theoretical analysis and, indeed, the Guide deals with its prediction empirically, albeit by incorporating various predicted levels of distress. It can be argued that IRI has no place in a pavement design guide because it is, essentially, a tool for pavement management. Furthermore, if correct design is applied in order to limit cracking (and rutting, in the case of flexible pavements), global moisture-related movements are restricted, and good construction practice is followed, then good riding quality will result. Experience among the users of the Guide suggests that the predicted values of IRI are rarely critical to assess a design. Agencies interested in assessing the value of IRI for a design that satisfactorily addresses rutting and cracking could still use empirical relationships to determine a value, but these relationships should not be regarded as a structural design parameter.

The M-E design approach taken in developing the Guide involved incorporating theoretical concepts for various aspects of the design process and modifying them through field calibration based, almost entirely, on data from the LTPP GP sections. This approach to deriving material "models" generally involves the use of multiple regression analyses to derive material constitutive models from laboratory or field data—an application of a philosophy that has been used extensively in the United States for many years. This approach is a poor substitute for applying sound theoretical concepts. Several of the design models, such as that for reflection cracking in asphalt overlays, are entirely empirical and lack the advantage of being derived from a theoretical base.

A major shortcoming of the Guide, in its present form, is a lack of balance in the level of detail and accuracy that is combined in the distress computations. For instance, it is possible to base a design on a combination of detailed, measured, complex modulus master curves for HMA, but with simple estimates of subgrade and granular layer resilient modulus derived from soil classification or values of the California Bearing Ratio (CBR).

3 FLEXIBLE PAVEMENT DESIGN

The M-E pavement design approach is based on an assumption that load-induced pavement structural responses (e.g., stresses, strains, and deflections) can be used to predict the development of pavement distress, in the form of rutting or cracking, through the use of Transfer Functions. Thus, the key factors of interest considered in the flexible pavement design review were the procedures used to characterize the elastic moduli of the various paving materials and the subgrade soils, because these form the most important input to the structural response calculations. The veracity of the transfer functions was also closely studied.

A detailed review of flexible pavement design is given by Thompson et al. (*1*) and further observations are included in Brown et al. (*3*).

3.1 Resilient Modulus for Subgrade

The Guide uses CBR for soils and granular materials as a basis to estimate resilient modulus (M_r). However, research has shown that the relationship between these parameters is not reliable for either material type (*10, 11*). This unreliability results from the non-linear stress-strain relationships involved and the fact that CBR is, at best, a measure of undrained shear strength, which does not relate closely to resilient properties at relatively low stresses, but may relate to permanent deformation resistance. This relationship was derived from research at TRL in the United Kingdom. Croney (*12*) has noted that the relationship is based on wave propagation measurements for in situ M_r, which were at very low stress levels, and field CBR tests, the results from which were not very reliable.

Given that the typical default values of M_r at OMC values provided in Table 2.2.51 of the Guide are based on the CBR relationship, these values should be reviewed. In addition, some guidance enabling the user to select M_r values that consider nonlinearity for granular materials and soils would be helpful, particularly for Level 2. Guidance for estimating values for fine-grained soils from parameters such as the clay content, Plasticity Index, and water content, could also be provided. For candidate granular materials, a database of laboratory properties could be developed for local areas with an emphasis on resilient modulus and resistance to permanent deformation.

The Guide does not generally reflect developments in soil mechanics of relevance to pavement design, with the exception of the way in which soil suction-water content characteristics have been used to adjust resilient modulus values. However, there are errors in the equation used by the Guide software (2.3.5) to compute the change in M_r as a consequence of changes in water content. In addition, no allowance is made for surface infiltration in flexible pavement design.

3.2 HMA Complex Modulus

A prediction model for the complex modulus of HMA is used in the Guide for situations when experimental results are not available and for the lower levels of design. The so-called Witczak model is used, but others are available such as the Hirsch model (*13*) and consideration should be given to offering the option of using either. The latest version of the Witczak model (*14*) should be incorporated. It would be helpful for the complex modulus computations to be available as stand-alone options in the software. Whichever method is used, the design computations require that a complex modulus master curve be available that relates this parameter to reduced loading frequency. The frequency for a particular situation is derived from an estimation of load pulse duration in the HMA sublayer under consideration. Given that this assumption is inherent in the Witczak model, the conversion of loading time (t) to frequency (f) is based on the relationship:

$$f = 1/t \tag{1}$$

When dealing with experimental data from sinusoidal loading of materials, an alternative conversion has been used in other circumstances, notably in the classic Van de Poel approach to bitumen stiffness estimation (*15*):

$$f = 1/2\pi t \tag{2}$$

However, in the context of the Guide, Equation 1 is considered appropriate.

In its present form, the Guide procedure for estimating complex modulus in the HMA layers results in unrealistic values, particularly for thick layers, showing a decrease in predicted modulus with depth in hot weather that is counterintuitive. This quandary results from the loading time/frequency effect overriding the temperature effect. Selecting a design frequency related to vehicle operating speed is recommended as an alternative to the present method.

Effective in situ loading time needs to be investigated further, not least because its definition for use in design is complex. Brown (*16*) pointed out that, even for a particular vehicle speed, there is no unique load pulse length for the layer because the vertical stress pulse at a particular depth differs from the horizontal pulses and both also vary with depth. The procedure in the Guide uses the vertical stress pulse, which is a simplification that needs to be justified. Brown suggested that the appropriate value should be determined by averaging the vertical and the two horizontal pulse lengths and also averaging them through the HMA layer or sublayer being considered.

3.3 Fatigue Cracking

Fatigue cracking of HMA is dealt with by procedures for the traditional, bottom-up, cracking mode and by introducing, for the first time, a similar approach for top-down cracking. The procedures follow well-established lines, except that the laboratory models have been heavily modified through a field calibration process, despite very large variability. No mention is made in the Guide of the possibility of an endurance limit in fatigue for thick asphalt construction, which has been widely discussed in recent years (*17*).

The following generic HMA fatigue algorithm should be introduced to give users an alternative option:

$$N_f = K_1 \left(1/\epsilon_{AC}\right)^{K2} \left(1/E_{AC}\right)^{K3} \tag{3}$$

in which

ϵ_{AC} = Maximum tensile strain in the HMA
E_{AC} = Stiffness of HMA layer
K_1, K_2, K_3 = Material constants

Top-down cracking is assumed to be longitudinal without any reason being given. Top-down cracking has been a subject of much research in recent years and is a more complex problem than assumed in the procedure proposed in the Guide. The standard error of estimate is very large, making use of the model unreliable and impractical. It would be better at this stage to omit the prediction of this distress type, pending further research to give a sounder basis for design.

3.4 Rutting

The Guide has abandoned the traditional vertical resilient subgrade strain criterion for rutting. This is welcomed, but the alternative method proposed for prediction of rut depth analytically, involving permanent strain prediction models for each layer, is not adequate for practical use. The development of permanent strain in pavement materials is very difficult to predict from basic material characteristics because of the many variables involved. For instance, in HMA, this particular mechanical property, even more than dynamic stiffness or fatigue cracking resistance, requires a laboratory test to be conducted on the material under consideration. This is because the aggregate structure has a fundamental influence on the result. Consequently, details such as particle surface characteristics, shape, and grading, together with packing and orientation after compaction, are all influential and cannot reliably be predicted using an empirical model.

The philosophy used in the Guide assumes that most rutting is caused by volume change in the HMA following compaction under traffic, whereas much field evidence shows that rutting is principally a shearing phenomenon, at least for well-constructed HMA layers, and a major output from the SHRP research supported this (*18*).

The permanent deformation model used in the Guide takes the same general form for HMA, unbound materials, and soils, and expresses permanent strain as directly proportional to resilient strain. There is an empirical relationship between these two parameters for HMA, but no fundamental reason why this should be so. Permanent strain is essentially controlled by the level of repeated applied stress expressed as a ratio between shear and normal components. Ongoing work in this field under NCHRP Project 9-30A will provide an opportunity to identify improved techniques for modeling permanent deformation in HMA.

Major assumptions have been made in the Guide procedure based, in some cases, on little evidence. One example of this is the relative contribution to rutting from the various layers, which was derived from minimal field data.

For the present, it would be better to rely on good mixture design and testing to limit rutting from the HMA layers and to use allowable stress criteria to deal with the lower layers. Stress criteria could be based on accumulated research knowledge from repeated load triaxial testing, which has identified "Threshold Stress" limits for many materials (*17*). The similar "Shakedown Limit" concept (*19*) also holds promise for future application.

3.5 Thermal Cracking

The Guide uses a complex procedure to predict thermal cracking in HMA. However, the properties of the HMA layer are assumed to be constant with depth, which could present a problem for pavements with thick HMA construction. Difficulties could arise in the event of an unusually cold winter unless the database in the EICM is extended to accommodate a longer history of records.

3.6 Other Materials

The Guide procedures for design of pavements incorporating chemically stabilized materials should be brought up to date to reflect recent advances, notably in connection with the use of lime stabilization.

The Guide could be improved by including more advice on sustainable construction (e.g., issues such as recycling and the use of materials that would not comply with current specifications, such as marginal aggregates and industrial byproducts). Cold-mixed asphalt-treated granular materials and techniques, such as foamed bitumen, fall into this general category, but are not considered in the Guide. These various materials will become increasingly important as demands for more sustainable construction are made by highway authorities and end-product, performance-based specifications are introduced.

4 RIGID PAVEMENT DESIGN

4.1 JPCP Design

The design process for JPCP represents a significant step forward. It is well formulated and presents the state of knowledge available to the 1-37A team at the time that their work commenced. The mechanistic concepts, incorporation of many parameters not included in previous design procedures (e.g., climatic effects, transfer functions converting pavement responses to pavement distresses, and incorporation of traffic stream and axle load distribution concepts) are all to be commended.

Typical designs for JPCP carried out by the review team (*2*) appear reasonable and agree with experience. It is, therefore, considered that the Guide procedures for JPCP can be implemented when the detailed issues set out below have been resolved.

4.1.1 Issues to be Resolved

The exact role that curling and warping plays during hardening of the concrete and during daily/monthly temperature and moisture cycles is not clear from the documentation and models presented in the Guide. At issue is the arbitrary approach to establishing equivalent temperature gradients to account for the built-in curling/warping gradient and the moisture gradient within the slab. Further experimental and modeling work is required to accurately determine the effect on incremental damage.

Curling and warping produce static, long-term stresses of a significant magnitude. It is not clear that these stresses can be simply added to those caused by transient traffic loading when using a linear fatigue damage accumulation model such as Miner's Hypothesis.

There is little discussion of how negative temperature gradients cause curling stresses to produce top-down cracking. It is not clear how these effects are accounted for in the neural network computation environment. More documentation on top-down versus bottom-up cracking and further validation of both phenomena are needed.

There must be a separation between permanent warping that occurs during concrete hardening, and the warping that results from climatic changes during the pavement service life. How these effects combine to produce a critical tensile stress and the influence of creep during the initial hardening stage should be considered.

The Guide recommends −10°F as the effective temperature to determine permanent curl/warp. However, this value will be affected by time of placement, joint spacing, load transfer at joints and base/slab interface conditions, some of which cannot be predicted at the design stage. There is insufficient justification for use of this single value. The model assumes shrinkage warping can be accounted for by use of an equivalent negative temperature profile that produces a concave upward curling of the slab. However, shrinkage warping is not always an additive effect and might counteract downward curling.

There is hardly any discussion of modeling for expansion and contraction of pavement slabs resulting from the change of average slab temperature on a daily or seasonal basis. The effects on load transfer efficiency (LTE) and of compressive force in the pavement during summer on fatigue calculations for transverse cracking need to be considered.

Local calibration/validation of the design procedure is required using local experience. State DOTs are encouraged to develop catalogs of designs for their conditions to ensure that the designs are rationally developed and incorporate agency experience.

The omission of longitudinal cracking as a distress mechanism is questionable. It is a particularly damaging form of distress because such cracks can propagate indefinitely and are known to be a serious form of distress in the Western United States. In addition, corner cracking is not included as a quantified distress mechanism, although it is discussed in the Guide.

4.1.2 Input Data

A major issue is the amount and complexity of input data called for in the design procedure. Some data are highly critical; others have relatively little effect on design. A procedure to classify these data into three categories is recommended. Type 1 data could incorporate information that is absolutely vital such as

- Traffic data;
- Slab thickness;
- Concrete strength, shrinkage and thermal properties;
- Subgrade properties;
- Base type and properties; and
- Climatic conditions.

Type 2 data could embrace information not used in the various design models or for which the design procedure is not sensitive. Type 3 data could incorporate parameters to which the design procedure is sensitive but the appropriate data would not normally be available at the time of design. Inclusion of values that are tentative at best could lead to misleading conclusions with regard to the sensitivity of the design to these parameters.

The Guide recommends that the design value for Modulus of Subgrade Reaction (k) be obtained from an assumed value of resilient modulus that is, in turn, obtained from a correlation with CBR. This approach is regarded as very unreliable. A dynamic k value is assumed, whereas CBR is obtained from a static test. This issue is discussed in more detail in Section 2.2 in connection with flexible pavements. Different soil types will have significantly different relationships between dynamic and static test results. Estimated resilient modulus values given in the Guide for the base and subbase layers seem too high, particularly when they are estimated from soil classification.

4.1.3 Design Sensitivity

The software for rigid pavement design was shown to work well and run much faster than that for flexible pavements. Consequently, the Review Team carried out some work on sensitivity issues that revealed some anomalies. In one particular study, it was found that subgrade type and strength had very little effect on slab cracking, even though the subgrade moduli values ranged from less than 1,000 psi to nearly 30,000 psi. In many cases, pavements on stiffer soil performed worse than those on softer subgrades. In an equal number of cases, the opposite was true.

4.2 CRCP Design

The design procedure for CRCP is a major step forward in understanding factors that might affect the behavior and performance of pavement. The analysis procedures provide valuable guidance on the importance of key parameters affecting the performance of CRCP. The algorithms incorporated to compute structural response and estimate damage are logically sound and instructive. However, the models used to predict punchouts, the principal failure mechanism, are extremely complicated and only one possible mechanism has been considered. The review team considers that the models do not correctly analyze the response and failure mechanisms of CRCP due to loads and climatic conditions, because the equations appear to be more research-oriented than useful as design tools. The procedure requires numerous inputs to solve more than 70 equations. Even though many of the input variables are computed or assumed by the software, many of them are not familiar to most design engineers.

For the above reasons set out in detail in Barenberg et al. (*2*), the review team believes that major issues must be resolved before the CRCP design procedure can be implemented.

4.2.1 Issues to be Resolved

One of the primary issues to be resolved relates to determining the thicknesses of PCC required to provide a desired level of reliability. Using the Guide, the thickness of CRCP is equal to or somewhat greater than that for JPCP, which is contrary to the design procedures used in many states (for which similar thicknesses are apparent). The excessive CRCP thickness is probably caused by the Guide's procedures for estimating factors that affect punchouts.

A basic assumption made in the design procedure is that there is a good correlation between crack spacing and crack width. Field observations reveal that this is not the case—crack width depends on the time when the crack occurs because time affects the shrinkage drying that initiates cracking. Another reason is that crack spacing varies and crack width depends on the spacing of the two cracks it dissects. Principally for these reasons, the review team questions the accuracy of the entire algorithm for punchout prediction.

A major concern is the apparent confusion between bond and friction between the base and slab. Bond prevents one layer from moving relative to the other, whereas friction indicates a force needed to cause relative horizontal movement. Bond will prevent one layer from lifting from the other whereas friction has no resistance to vertical separation. For CRCP design, no account is normally taken of bond but slab/base friction is considered in order to provide for changes in slab length.

Another concern is the effect of crack spacing on punchouts. The descriptions in the 1-37A report (*4*) imply that the probability of punchouts increases as the transverse crack spacing becomes smaller. At the same time, there are several statements to the effect that smaller crack spacing will result in smaller crack width with better load transfer efficiency. In some cases the number of punchouts was found to increase with the base stiffness, which is counter-intuitive; the exact reason for this phenomenon is not known and must be resolved.

4.2.2 Design Sensitivity

Punchout distress was found to be sensitive to coefficient of thermal expansion, curling and warping, slab thickness, percent steel, and concrete strength, but not to aggregate type or to the erodability of the base, which is contrary to the experience of most DOTs and to the impression left by the 1-37A report.

Base/slab friction has no effect on performance. Although curing effectiveness influences crack development and spacing and, hence, punchouts, the computations are insensitive to curing method. Designs are also insensitive to coarse aggregate type, which is contrary to the experience of many state DOT engineers.

5 PAVEMENT REHABILITATION

Detailed observations on the Guide's coverage of rehabilitation design are given by Brown et al. (*3*). Given that the same philosophy and distress models are used for the design of pavement rehabilitation as for new pavements, the observations in Sections 2 and 3 of this report also largely apply to rehabilitation design. The principal difference is at Stage 1 (Figure 1), in which the input data accommodate the results of material and structural evaluation investigations carried out on the existing pavements. The Guide should emphasize that rehabilitation design will become increasingly important because the highway system in the United States has largely been built. Also, rehabilitation design can be conducted to a greater level of reliability than new pavement design, because the highway already exists and measurements can be made of key parameters (e.g., traffic, material properties, and structural integrity) through field and laboratory testing. Equilibrium water conditions will have been established in the subgrade and the level of distress that exists at the time of evaluation can be quantified.

5.1 Pavement Evaluation

An advantage of rehabilitation design is that field testing, particularly using the Falling Weight Deflectometer (FWD), can be carried out to determine the in situ properties of materials. The pavement survey procedure in the Guide specifies very wide spacing (typically 500 ft for FWD) between test points, even for Level 1 analysis. Such spacing may be appropriate for network analysis, but for detailed evaluation of a site, from which rehabilitation measures are to be developed, a spacing of around 60 ft is desirable. Other field testing techniques, including the Dynamic Cone Penetrometer and Ground-Penetrating Radar can also be effectively deployed for existing pavements. In addition, for bound layers, cores can be cut to provide appropriate laboratory test specimens.

The application of good structural back-analysis of FWD deflection data provides vital information on the effective stiffness of each of the pavement layers. This information can be used to assess both the level of distress (particularly through cracking) and to provide input for overlay or other rehabilitation design measures. One of the most important aspects of back-analysis is the ability it gives the designer to fit the theory to the actual pavement by ensuring that the measured resilient response of the pavement matches the value determined from structural analysis. This is a situation that cannot be achieved when analysing a pavement that has yet to be constructed.

In the Guide, the characterization of existing HMA stiffness when dealing with HMA overlays and with PCC overlays is inconsistent. For the HMA overlays, FWD back-analyzed values can be used, but not for the PCC overlays, which are considered illogical. Use of the FWD for evaluation of concrete pavements is less well covered in the Guide than that for flexible construction. In particular, the procedure for determining a Modulus of Subgrade Reaction (*k*) from back-analysis needs to be clarified.

In addition to its use for determination of in situ effective layer stiffnesses, the FWD can be applied to the measurement of other important pavement characteristics such as interlayer bond, concrete joint properties, and the detection of incipient cracks in cement treated bases. Use of the FWD can either be part of a rather simple evaluation process based on measured deflection parameters, an intermediate process using simple two-layer back-analysis, or a more complex procedure for experienced pavement engineers (*3, 19*).

5.2 Rehabilitation with HMA

Relative to the advice given in the Guide, more reliance should be placed on data from field testing and less on laboratory testing of cores. The stiffness of cores tested in the laboratory is likely to differ from FWD-derived values, even after allowance for possible loading time and temperature differences. This difference occurs because cores essentially give point values for coherent material, whereas FWD-derived values represent the effective stiffness in situ, accounting for cracking, de-bonding, or voided asphalt materials. Improved advice on the effect of age hardening and traffic damage on HMA should be given in the Guide, in order to better understand the in situ material characteristics for rehabilitation design.

The Guide requires that the "as new" material properties for an existing pavement be determined. Given that the pavement may have been in service for many years, making this determination is difficult. It is also illogical because the pavement parameters that influence rehabilitation design should be the existing ones, not those that may have occurred at the time of construction. This feature of the Guide results from the same approach being adopted for rehabilitation as for new construction.

The Guide uses equivalent laboratory values of resilient modulus for unbound material derived from the FWD values using simple ratios based on earlier work. This approach does not seem well-founded.

Design stiffness for chemically stabilized materials is mainly related to laboratory values in the Guide. However, weaker stabilized materials may well have deteriorated to a granular condition after many years of trafficking in an existing pavement. Because this condition can be quantified using back-analysis of FWD data, such analysis is recommended for all levels of design.

Interface conditions between various pavement layers are considered in the Guide. A value between 0 (no bond) and 1 (full bond), apparently on a non-linear scale, can be incorporated in the analysis for calculating load-induced stresses and strains. The concept is believed to be based on assumptions in the JULEA program. However, advice on the sensitivity of the design thickness to this parameter and the recommended values for various cases are not described. The designer commonly assumes that full bond exists between all pavement layers. However, field studies have shown that this is often not the case and the power of FWD back-analysis could be harnessed to quantify an interlayer stiffness (*20*).

Reflection cracking is noted in many sections of the Guide as a distress mechanism commonly associated with HMA overlays to both rigid and flexible pavements. However, reflection cracking is treated simplistically compared with other aspects of performance, and prediction is based on a simple empirical formula. The empirical reflection crack model produces reasonable results, but needs to be expanded to cover a wider range of pavement conditions in terms of thickness, stiffness, and joint characteristics. Some suggestions are given by Brown et al. (*3*).

The use of a fabric placed below the HMA overlay to prevent or delay reflection cracking is one of the techniques included in the Guide. For design purposes, it is assumed, without justification, that a properly installed fabric can reduce the HMA overlay by 2 in., but this value is an entirely empirical allowance. The Guide makes no specific reference to the use of grid reinforcement in asphalt. Materials of this type provide genuine reinforcing elements, the best ones having the ability to interlock with the HMA aggregate. They can be made of high-tensile polymeric material, glass fiber, or steel, and some are supplied with a geotextile backing to aid installation. When correctly installed, some of these materials have been shown to be effective at reducing reflection cracking, particularly from thermal movements. A recent report provides useful information on this subject (*21*).

Advice on optimizing strengthening recommendations and the available options, other than HMA overlay, should be given in the Guide. These can include inlays, combinations of inlay and overlay, partial or complete reconstruction, and recycling options.

5.3 Rehabilitation with Concrete

For the determination of flexural strength in existing concrete, consideration should be given to specifying an indirect tensile test and deriving the flexural strength from this test. Indirect tensile tests are much easier to perform than flexural tests because they can be carried out on cores, rather than on beams. In addition, the relationship between indirect tensile and flexural strengths is believed to be much more certain than that between compressive and flexural strengths.

The effect of load transfer efficiency (LTE) on computation of stresses in PCC and in the prediction of joint faulting is highly significant and strongly affects the occurrence of traffic-related reflection cracks. Therefore, it is appropriate that considerable effort has been made in the Guide to express LTE as correctly as possible. However, the details relating to effects of aggregate interlock and dowel LTE appear to require some corrections (*3*).

Faulting is most sensitive to a quantity described as "differential energy of subgrade deformation," calculated in the Guide from the deflection under load and various joint parameters. Although this calculation is entirely logical (because it is the factor that takes account of the effects of traffic loading), the calculation is surprisingly insensitive to subgrade fines content and the number of wet days per year.

Calibration work for concrete pavement restoration should be carried out using data from sites covering a wider range of conditions than previously, and more data are needed on the performance of CRCP overlays to existing PCC pavements for heavily trafficked sites.

The prediction of transverse cracking and joint faulting in JPCP overlays does not appear to match the available evidence particularly well. The issue should be investigated further.

6 DESIGN RELIABILITY

Pavement design and rehabilitation are complex and involve many uncertainties, variability, and approximations. Reliability is critical for estimating the effect of construction and material variability on pavement design and performance. The same variance and reliability models have been used for design of both new and rehabilitated pavements. Design reliability is defined as the probability that each of the distress types will be less than a selected critical level over the design period. Average inputs are used in the design guide and average responses and distress are predicted. The various distress mechanisms are taken to have normal (Gaussian) probability distributions.

Variability and, hence, reliability has been estimated by comparing predictions with measured values from LTPP sites. Depending on the level of the input data, variability will include factors such as measurement errors and errors associated with material characterization parameters, traffic and environmental conditions, and errors associated with the model prediction algorithms. For the deterioration modes, there was more calibration data available at low levels of pavement distress than for more seriously damaged pavements. This means that the variability determined for the higher levels of distress is likely to be less reliable because it is based on fewer data points. Apart from thermal cracking, the same standard deviations have been used for all design Levels. This implies that there is no reduction in variability, and hence improvement in reliability, when more accurate input parameters are used.

The use of techniques such as a Monte Carlo simulation should be investigated so that the variability of input parameters can be used directly to estimate the variability in predicted performance and, hence, the reliability of design.

Reliance on LTPP data as a basis for model calibration and establishing the reliability of designs is questionable. A recent report by Hajek et al. (*22*) concerning the traffic data for LTPP sections raises serious questions about the reliability of the data.

A major concern with numerous calibration constants used in the Guide is their application at so many levels. It would be preferable if the procedures could progress as far as possible using only analytically based concepts (without calibration constants) and apply a single adjustment or correction factor at the end, if it is required. To form judgments about this, it would be necessary to have high-quality field performance data available. Such data are generated from Accelerated Full-Scale Pavement (APT) testing such as that conducted at WesTrack (*23*), MnRoad (*24*) and, more recently, at the National Center for Asphalt Technology (*25*).

7 DESIGN SOFTWARE

The version of software made available for this review (dated June 2004) did not incorporate the latest revisions for flexible pavement and rehabilitation design from the 1-37A contractors. Nonetheless, it is clear that the software has many useful characteristics that can be of great help to pavement engineers. However, this version had many errors, which resulted in failed runs, and numerous problems were encountered by the many users who worked with it. When used for rigid pavement design, the software performed much more efficiently and quickly than for flexible pavements. However, nearly all reviewers reported some problems and, in a number of instances, calculations and results did not appear to agree with what was written in the Guide. Specific issues that arose from the review team's use of the software are set out in the detailed reports (*1–3*).

The Guide's very detailed analytical computations pertaining to distress prediction procedures, which are also incorporated in the software, result in excessive computing times, particularly for flexible pavements. Given that the user has to make manual adjustments to an initially unsatisfactory design before re-running the software and has to continue to do this until the design is acceptable, the overall design process requires considerable time. The software provides a Microsoft Windows environment. Users might expect the iterative computations to be incorporated in the software so that various "what if" scenarios (such as alternative materials) could be explored with the outputs given in terms of required layer thicknesses.

The input data required for design are extremely comprehensive. A tabular summary with critical observations has been provided by Brown et al. (*3*). Much of the data input seems too detailed and unrealistic or unnecessary, such as the requirement to specify cement content for PCC when mechanical properties are also required. Very extensive data are required on climatic conditions, requested on an hourly basis. This relates to the very detailed analyses of distress, the results of which are presented on a monthly basis. For rehabilitation, the assumption that construction records will be available and provide a reliable source of input for evaluation and design is naïve. It is far better to measure the characteristics required by the software, preferably through field testing or, alternatively, by laboratory testing of field samples.

Given the desirability to input detailed traffic loading data, the software developed under NCHRP Project 1-39 must be made compatible with the Guide software. Compatibility would allow the detailed data collected by DOTs to be directly used in design.

In rehabilitation design, the input data required for existing HMA and PCC are similar to those for new materials. Parameters such as cement type and the curing technique for concrete or rheological characteristics of HMA binders in their "as-new" state after short-term aging, are impossible to determine for an in-service pavement.

The various changes proposed for the design process, which have been outlined in this report, should be incorporated into the software. In addition, it is recommended that stand-alone versions of the structural analysis and EICM subroutines should be accessible to the user so that particular outputs, such as values of stress or strain, could be accessed.

Appendix D of the 1-37A report is the software User Guide that, helpfully, takes the reader through some typical designs with views of the relevant screens provided to illustrate what can be expected as a design case is being worked through. However, inconsistencies in terminology between this document and the main report will need to be corrected in the context of publishing a user-friendly manual for the whole design process.

Serious work remains to be done on the software. A professionally conducted beta testing exercise is recommended before a final version is made freely available for the use of design engineers. It would be very unfortunate if new versions contained the level of significant errors apparent in the review version.

8 RECOMMENDATIONS

The three detailed reports, which form the backdrop to this digest (*1–3*), have set out numerous essential and desirable actions that the review team considers necessary before the Guide is acceptable for use in engineering practice. The sections below set out the most important of these changes and attempt to capture the essential features of the review's overall recommendations.

8.1 Flexible Pavement Design

8.1.1 Essential

1. Incorporate the new prediction model (Bari and Witczak, *14*) for HMA complex modulus and allow users the option of using the Hirsch model (*15*).
2. Improve the application of soil mechanics principles to the characterization of soils and unbound materials and, in particular, omit the use of CBR in design. Provide improved guidance on the estimation of M_r from basic soil characteristics. Correct the adjustment procedure for M_r resulting from changes in water content.
3. Resolve queries about the correct determination of in situ loading time for an HMA layer.
4. Omit the top-down cracking distress prediction computations and await the results of ongoing research on this subject.
5. Replace the rut depth prediction procedures with good HMA mixture design combined with a permanent deformation test and the introduction of allowable stress levels for the granular layers and subgrade.
6. Provide a more flexible approach to the modelling of asphalt fatigue and introduce the concept of an endurance limit, particularly for heavy-duty pavements.

8.1.2 Desirable

1. Omit the use of IRI as part of the structural design procedure, because it is essentially a pavement management tool.
2. Emphasize local calibration of the models and predictions rather than adjusting national calibrations.
3. Improve the guidance given to use of chemically stabilized materials and to other cold-mix and recycled materials.
4. Include surface water infiltration as a factor to be accounted for in design.

8.2 Rigid Pavement Design

8.2.1 Essential

1. Simplify the design process, particularly with regard to the required inputs.
2. Conduct state-level calibration/validation for JPCP.
3. Improve the treatment of curling and warping for JPCP.
4. Address the incompatibility between CRCP design thicknesses generated by the Guide and current practice.
5. Improve the way in which the "zero stress" condition is defined with respect to the time of construction for CRCP.
6. Consider other mechanisms for CRCP punchout failure, in addition to the one used in the Guide.

8.2.2 Desirable

1. Include longitudinal and corner cracking as distress mechanisms and provide improved advice

with regard to raveling. Review the permanent warp assumptions when a granular base is used.

2. Improve the way in which the Guide deals with climatic effect for JPCP design.

3. Address the issues of crack width and crack spacing as part of the validation process for CRCP.

4. Reconsider the significance of curing condition and base erodability for CRCP.

5. Resolve the effect of stabilized bases for CRCP and the relationship between friction and bond.

8.3 Pavement Rehabilitation Design

8.3.1 Essential

1. Use in situ material properties obtained from pavement evaluation as input parameters for rehabilitation design.

2. Give better advice on HMA stiffness prediction for existing pavements.

3. Give advice on other uses of the FWD, in addition to the determination of pavement layer stiffnesses.

4. Specify closer spacing for FWD testing, coring, and DCP testing for the various design levels.

5. Investigate and carry out more research of laboratory-resilient modulus predictions of unbound materials from field values determined from FWD data using various conversion factors.

6. Improve the procedures for structural evaluation of concrete pavements.

7. Improve the determination of LTE between slabs and across cracks.

8. Check and correct, as appropriate, the detail concerning base erodability, upward curl, and overburden on subgrade in relation to the computations for faulting in concrete slabs.

8.3.2 Desirable

1. Give recommendations on the effect of interlayer bond condition on pavement evaluation, life prediction, and recommended treatment.

8.4 Software

8.4.1 Essential

1. Correct all the errors in the software and update it in the light of developments.

2. Conduct a professional beta testing exercise before issuing the software for general use.

8.4.2 Desirable

1. Allow the HMA complex modulus prediction subroutine(s), together with JULEA, EICM, and the finite element analysis programs to be used independently of the design process.

2. Provide for critical response parameters in terms of load-induced stresses and strains to be available as part of the software output.

8.5 Other Matters

8.5.1 Essential

1. Address the shortcomings in the calibration procedures for rehabilitation models used in rigid pavement rehabilitation.

2. Re-address the whole issue of reliability in light of the work dealing with local calibration under way in Project 9-30.

3. Investigate the application of Monte Carlo simulation or alternative methods in order to improve the understanding of variability in design and the effects of variability in input data.

4. Improve the guidance to users of the Guide within a more succinct, user-friendly version of the document. Ensure that the terminology in the software and in the Guide is consistent.

5. Revise the Guide for use in the short term by offering a more balanced approach in which the various elements of design at each level are approximately of the same complexity.

6. Expand the advice on recycling options.

7. Introduce the concept of long-life pavements.

8.5.2 Desirable

1. Omit IRI from the design process because it is a pavement management tool and cannot be predicted analytically.

2. Given the problems encountered with the use of LTPP data for calibration of many elements in the Guide, reconsider adoption of the national calibration, at least for flexible pavements.

3. Over the long term, reduce the dependence on empiricism in the Guide and increase the basis of sound theory, making use of the wide range of research conducted since work on the Guide began in 1998.

4. Introduce simple design charts based on local calibration for various states.

5. Propose a better model for reflection cracking.

6. Provide better guidance on the use of grids, fabrics, and stress-absorbing membrane interlayers (SAMIs) for mitigating reflection cracking.

7. Improve the quality of pavement evaluation information for rigid pavements and the advice on design of rehabilitation using PCC.

9 CONCLUDING DISCUSSION

The Guide in its present form is not suitable for immediate use by design engineers. It is a substantial piece of research that, with the further work being undertaken, could be developed into a powerful tool for design. It embraces innovations and combines for the first time in a single computer program several important aspects of design previously used independently. Inevitably, because of the variable levels of knowledge and development in the enormous number of aspects to pavement design, the soundness of the underlying engineering principles varies considerably.

The field calibration took a considerable effort by the 1-37A contractors, but it is inevitable that huge variability should have been experienced with use of the kind of data generated by LTPP. Therefore, the reliability of the designs is questionable. In addition to carrying out further calibration/validation work using more reliable performance data, the vast experience of designs used in the past should be accommodated. Any new method must provide solutions that agree with good performance in the past in comparable circumstances. The advantage of an analytically based method is that it can meet new conditions (in terms of materials, climate, and types of construction) in addition to standard situations.

The importance of rehabilitation for the future is not adequately recognized in the Guide. Many improvements could be made based on experience elsewhere; these have been set out in Brown et al. (*3*).

The various recommendations set out in this report, and supported by more detailed discussion in the companion reports (*1–3*), have been put forward by the review team with a view to improving the quality and reliability of the Guide as successive versions are developed and it moves toward acceptance by the pavement engineering community. The recommendations are based on extensive experience in all aspects of pavement design, including an international perspective.

The software can be a user-friendly package for engineers, but all the reported difficulties with its use should be corrected and the recent improvements should be incorporated. Before a usable Guide can be introduced to design practice, a number of strands of current and recent research need to be accommodated to provide improvements. In addition, a professionally planned beta testing program is recommended for the software. The final product must work well and reliably; otherwise, it will be very difficult for confidence to be gained in its use by design engineers. The software will need to be accompanied by a well-written, professionally presented user guide that provides sufficient background and guidance, but is also far shorter than the comprehensive 1-37A research reports.

PART II

Detailed responses of the Project 1-40D team to the essential recommendations made by the Project 1-40D technical panel and the three independent review teams follow. (Note: N/P = not planned for Versions 0.900 or 1.0).

Panel Recommendations

Area-#	Recommendation	1-40D Team Response	Version
P-1	Eliminate calculation of IRI.	**Disagree.** AASHTO pavement design has considered the highway user through serviceability (or smoothness) for 40+ years, and this vital criterion should continue to be included. The IRI models include indirectly the long-term profile effects not included in the flexible or rigid M-E models, such as profile changes as function of age, climate, and subgrade properties.	N/P

(*continued*)

Panel Recommendations (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		On the rigid side, IRI works well. The standard error is good and IRI is a direct function of the M-E main distress types predicted. On the flexible side, the existing models have relatively high errors and are missing or insensitive to key distresses. New, improved IRI predictions models for new HMA, HMA over HMA, and HMA over PCC are under development. These new models are being developed using a much larger and more recent LTPP data set that includes the following: • New HMA—GPS 1 & 2, SPS 1, 5, & 8 (>1200 time series data points). • HMA/HMA—GPS 6, SPS 5 (>700 time series data points). • HMA/PCC—GPS 7, SPS 6 (>300 time series data points). The new IRI models include key M-E-predicted distresses, including rut depth, top-down and bottom-up fatigue cracking, and transverse cracking. Their effect on IRI is more significant than the existing models. Also included are pavement foundation (subgrade) properties known to affect the potential for foundation pavement movement (swelling and heaving), along with climatic factors that drive foundation movement. The improved models have considerably reduced model prediction errors. When considering the standard error of the IRI models, it must be realized that the overall error (scatter of data points around the predicted versus measured IRI one-to-one plot) includes IRI measurement error, section replication (or pure) error, and section input error as well as the true model error (lack of fit). The true model error is believed to be significantly less than the total error. Also, a much more robust data set is now available : >2000 observations, including older pavements and better representation of climate, subgrade, and traffic conditions experienced in the United States. The Project 1-37A panel directed inclusion of IRI in the MEPDG. IRI should be retained; the Version 0.900 recalibration will substantially improve the error in the IRI calculations.	
P-2	Include endurance limit in flexible and rigid cracking models.	**Agree.** No applicable results were available to the 1-37A team during the development of the MEPDG. However, endurance limit can be easily implemented in both flexible and rigid designs. The MEPDG software will be modified to include the endurance limit by keeping a tally of the loads that exceeded the limit and outputting that quantity or	0.900

Panel Recommendations (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		percentage. Inclusion of an endurance limit has no practical effect on the flexible design results because the analysis is still the same; Projects 1-40D and 9-38 will provide additional information on this topic. However, this capability will be included with an upper limit of 100 microstrain. For rigid design, a stress-to-strength ratio will be included to implement an endurance limit; the percentage of damage caused by ratios below 0.5 will be reported.	
P-3	Re HMA PD, turn off permanent deformation model?	**Disagree.** What is the downside of considering only fatigue cracking? We believe that it would be a major step backward for several reasons. First, the recalibrated model will be improved over the existing model and future models will be improved over this one; over time and use, a very robust permanent deformation model will evolve. Next, such an approach will tend to overestimate the thickness and can't be used for warranty or PRS applications. Third, the effect of poor mixtures on premature failures cannot be known. Eliminating permanent deformation would be tantamount to deleting an aspect of pavement design that has the most effect on highway safety. Last, there would be no way to derive equivalent flexible and rigid pavement designs using a criterion such as limiting strain only.	N/P
	Incorporate repeated load testing parameters?	**Agree in principle.** The use of repeated load test parameters to supplement the E* modulus will be evaluated in Version 1.0.	1.0
P-4	Don't use LTPP data. Set up so that all calibration is done on a local rather than national basis.	**Disagree.** LTPP is the largest single source of high-quality, reliable data collected anywhere in the world. Contrary to popular belief, a well-constructed, statistically balanced national calibration exercise should be able to account for all local issues, provided the data are well represented in the database. Robust national calibration is needed as a foundation for local calibration. However, agencies can use local data to further *refine* nationally calibrated models. Not every agency will or needs to conduct a local calibration. Therefore, local calibration is an <u>option</u>, not a <u>requirement</u>. Further, it is unclear how the validity of local calibration can be judged without comparison with national calibration statistics. The calibration data set being used in NCHRP Project 1-40D includes other data than LTPP, including MnRoad, WesTrack, the AASHO Road Test, NCAT test track, and other field pavement experiments. Local calibration data provides a means to reduce bias (over or under prediction) and the standard error of estimate. At least one-half of the errors in the current models are input, distress measurement, and pure	0.900 and 1.0

(*continued*)

Panel Recommendations (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		(replication) errors. Further, LTPP has formal and standardized methods of collecting distress data, which may or may not be the case for locally collected data. So, it is unclear how the validity of local calibration can be judged without comparison with national calibration statistics. However, we will test this hypothesis in Version 0.900 by taking out state data sets for comparison. Robust national calibration as a foundation for local calibration is needed. For Arizona, use of the national calibration as a starting point gave excellent results. Not every state will conduct a local calibration. In Version 1.0, statistical jackknifing techniques to test national calibration could be applied. LTPP data continue to be improved, cleaned up, and updated (Version 0.7 was calibrated with data collected through 2000, whereas Version 0.900 will use performance data collected through 2004). The Project 1-40D contractors have also reviewed and improved the data. Inclusion of modified calibration factors in Version 1.0 will permit the effective use of local calibration data.	
P-5	Incorporate new E* model.	**Agree.** This will be done in Version 1.0.	1.0
P-6	Include FWD back-calculation capability in the rehab design.	**Agree.** However, the Project 1-37A panel directed that no specific back-calculation computational method be used. If resources are available, future work can incorporate a general ARA, Inc., interface program that will be able to run several popularly used back-calculation programs. This program will be made available to AASHTOWare, pending further discussions. This program includes several available back-calculation methods and data-processing capabilities.	1.0 or later
P-7	Re change in design philosophy, calculation of performance prediction versus use of limiting strains?	**Disagree.** The incremental damage and performance prediction approach was directed by the Project 1-37A panel over a 6-year period. As stated above, going back to a limiting strains concept would make it all but impossible to develop equitable flexible and rigid designs and would greatly limit the capability and usefulness of the design method. In practice, any performance type can be "turned off" by the user, by changing the output instructions to allow display of only selected performance data. The MEPDG provides a framework for going forward in "bite size" portions. Version 0.900 will be much more robust in areas where there are interactions of different levels of input data. What are the benefits of having capabilities that are not available in the present AASHTO guide? The MEPDG allows comparison of equivalent HMA and PCC designs and provides a tool for LCCA. The	

Panel Recommendations (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		AASHTO guide overestimates the thickness needed for both HMA and PCC pavements. The capability for Monte Carlo simulations is needed; this was directed by the Project 1-37A panel, but could not be accomplished within the time and resources available to that project. It is believed that, overall, it will be far better to begin to implement the full MEPDG now, even with some known model deficiencies, than to revert to limiting strain or other criteria only. Clearly, large errors are inherent in this approach. Future research will enhance all of the existing models in the MEPDG, but this can only happen if they can be evaluated across the country at the same time.	
P-8	Turn off HMA top-down cracking model.	**Disagree.** This model will be recalibrated for Version 0.900 using a larger dataset. At present, no other viable model is available. We suggest this model remain in the MEPDG with the option to turn *on* a display of its results.	N/P
P-9	Unbound layer rutting.	**Agree.** Will further consider in Version 1.0. However, unbound layer modulus is a function of the material type. There are already two models for unbound materials in the software (same form, different shift factors). Lytton also used a single model for a variety of materials. Dr. Sherif El-Badwy's Ph.D. thesis at Arizona State University found that one model form was appropriate for a variety of subgrade types in Arizona. The new ICM will improve estimates of rutting in unbound layers. The Version 0.900 recalibration with cleaned-up LTPP data will reduce prediction errors significantly.	1.0
P-10	Re viability of the CRCP cracking model, incorporate Version 10 of the Zollinger model.	**Disagree.** Version 10 is not recommended for inclusion at this time by either Dr. Won or Dr. Zollinger for CRCP. It is recommended that future inclusion of various aspects of Version 10 technology be fully considered.	N/P
P-11	Improve the HMA thermal cracking model.	**Agree.** The use of additional sections and data does improve the Level 3 predictions, but with some inconsistency in the results. However, the model may require revision; this work is in progress, including necessary laboratory testing. Dr. Bill Buttlar is under contract to work on this issue for NCHRP. ASU has prepared specimens and is testing at different binder contents to check the thermal cracking model with Level 1 inputs.	0.900
P-12	Incorporate placeholders for future capabilities.	**Agree.** The software can include an initial screen with a summary of impending or potential changes due to research projects in progress on such critical topics as endurance limit, top-down cracking, and local calibration.	1.0

Recommendations of 1-40A(01) Flexible Design Team

Area-#	Recommendation	1-40D Team Response	Version
F-1	Check consistency of guide and software.	**Agree.** Version 0.900 will be accompanied by a supplement to the design guide that will describe changes in the software and how they affect the guide itself.	0.900 and 1.0
F-2	Fix all software bugs identified during the public evaluation and independent review.	**Agree.** Has been or will be done in Versions 0.8 and 0.900.	0.900 and 1.0
F-3	Moisture-modulus reduction equation.	**Agree.** Accomplished in Version 0.8.	0.900
F-4	Incorporate new G* and E* prediction models developed at ASU.	**Agree.** Will be done in Version 1.0.	1.0
F-5	Include an option to use the Hirsch model for E*.	**Disagree.** Comparisons show poorer results than with Witczak E* and G* equations (see the results based on 5000 data points in Javed Bari's Ph.D. dissertation). The present form of the Hirsch model requires different inputs and is based on a smaller dataset.	N/P
F-6	Resolve time of loading and frequency issues.	**Agree.** Will study this issue and incorporate changes in Version 1.0 if possible. This will include the addition of an optional capability for inputting layer moduli. Frequency is defined as 1/t, not as 1/ω. This is the definition used in engineering, physics, and chemistry, and is the appropriate form for this application.	1.0
F-7	Evaluate default modulus at OMC values in Table 2.2.51.	**Agree.** This will be done and the software updated as necessary. Two sets of default values are provided at present, one at OMC and the other at equilibrium.	0.900 and 1.0
F-8	Remove top-down cracking model.	**Disagree.** This model will be recalibrated for Version 0.900 using a larger dataset. At present, no other viable model is available. We suggest this model remain in the MEPDG with the option to turn *on* a display of its results. We believe it is far better to implement a model with some deficiencies than to ignore the distress altogether.	N/P
F-9	De-emphasize IRI predictions in evaluation of flexible designs.	**Disagree.** A new, up-to-date, and greatly expanded data set from LTPP has been obtained and new IRI models developed that show lower standard error and account for all the MEPDG-predicted distress types for flexible pavements.	1.0
F-10	Incorporate improved "robust" models for rutting and fatigue cracking.	**Disagree.** See responses to items P-2, P-3, and F-11. Calculated strain provides a connection with repeated load deformation. The new version of the ICM in Version 0.900, which will incorporate the Thorntwaite moisture index and provide a better indication of base and subgrade moisture, will improve estimates of rutting in unbound layers. Recalibration with cleaned-up LTPP data will reduce prediction errors significantly. Absence of	N/P

Recommendations of 1-40A(01) Flexible Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		measured trench data puts a lower limit on the error, but trenching will be done in 9-30A. This is a theoretically sound model. Lytton's model, looked at originally, gave much poorer results.	
F-11	In fatigue, provide a generic form of the equation. Include an endurance limit.	**Agree.** Will consider for Version 1.0. N cannot be accurately calculated without considering E*. K_3 can already be set to zero if desired, but this would necessitate a recalibration. This form may not give the expected results in the program. A relation between K_1 and K_2 will add complexity to the calculation. A strain limit will be added.	1.0
F-12	Provide PD models for each distinct type of material in a flexible pavement structure.	**Agree.** Will be considered in Version 0.900 recalibration. However, unbound layer modulus is a function of the material type. There are already two models for unbound materials in the software (same form, different shift factors). Lytton also used a single model after investigating 40 different soils. Sherif El-Badwy's Ph.D. dissertation research found that one model form was appropriate for a variety of subgrade types in Arizona. Although the basic model is the same, it is currently calibrated for fine- or coarse-grained materials. Will further consider in Version 1.0.	0.900 and 1.0
F-13	In Level 2, include monthly inputs for all layers of the pavement section.	**Disagree,** but can test in Version 1.0 if so directed. This is already done for unbound materials, but if it is implemented for HMA, the effects of aging will be eliminated. HMA modulus depends on frequency and temperature. Model already predicts modulus on a monthly basis. Will not permit interaction between mix and structural designs, nor will it account for aging over time. Would also affect calibration, probably to a high degree of error.	1.0?
F-14	Focus on local rather than national calibration.	**Disagree,** but the results of Project 1-40B will be adopted in Version 1.0. Will also test in Version 0.900 by taking out state data sets for comparison. Need robust national calibration as a foundation for local calibration. For Arizona, use of the national calibration as a starting point gave excellent results. Not every state will conduct a local calibration. In Version 1.0 could apply statistical jackknifing techniques to test national calibration. LTPP data continues to be improved and cleaned up; 1-40 contractors have also reviewed and improved on the data. Inclusion of modified calibration factors in Version 1.0 will permit the effective use of local calibration data.	0.900/1.0
F-15	50% minus 200 material criterion too high?	**Disagree** with using 35%. Will change the write-up in the design guide supplement to provide justification for value of 50% and discuss how it will be applied.	N/P

(*continued*)

Recommendations of 1-40A(01) Flexible Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
F-16	Present defaults for AASHTO classes only.	**Disagree.** Better to have both because the two classifications do not match. However, the team will look again at how closely the Unified and AASHTO classes match.	N/P
F-17	CSM design is not state of the art. Use NLA information.	**Agree.** This major effort will be considered in Version 1.0. There was a paucity of data for cement-treated bases in the initial calibration dataset.	1.0
F-18	No consideration of cold-mixed asphalt-treated granular materials.	**Agree.** Will consider in Version 1.0. There is a lack of a specific model form for cold-mixed materials, so would use same model as for asphalt concrete. It will be included in the pull-down menu as another layer type.	1.0
F-19	Uncouple the component programs: EICM, JULEA, and FEM.	**Disagree.** Stand-alone versions of all these programs exist now that are readily available to the technical community.	N/P
F-20	Provide selected response outputs such as stress, strain, and displacement.	**Agree in principle.** Can be done in Version 1.0, but at a reduced number of calculation points. Otherwise, the running time of the program will be unacceptably increased. An alternate but still undesirable solution would be to run JULEA as a stand-alone program.	1.0

Recommendations of 1-40A(02) Rigid Design Team

Area-#	Recommendation	1-40D Team Response	Version
R-1	Overall design process, input requirements: Inputs that have little effect on designs or those that cannot be determined when designs are being made should be shown and "grayed out" so they cannot be changed without justification and significant effort by the design engineer.	**Agree.** 1-40D has already grayed out or made some inputs more difficult to change. These include permanent curl/warp and others related to shrinkage. Others that meet the suggested criteria will also be identified and either grayed out or placed elsewhere. Alternatively, default values may be tailored to the requirements of individual states in future versions.	0.900 and 1.0
R-2	Overall design process, traffic inputs: Simplify the traffic inputs. They are too complex and incorporating design inputs into the design procedure is time consuming. Consider incorporation of traffic input program TRAFLOAD for inputting traffic data.	**Agree.** Some changes have already been made. The software has been modified to accept outside files containing all required input traffic data. Version 1.0 work will provide the capability to use the TRAFLOAD product for this purpose. However, this will not substantially simplify the design process because all of the same data are required by TRAFLOAD. Most traffic inputs are also required for the current 1993 AASHTO Guide in order to compute ESALs properly.	0.900 and 1.0
R-3	Software validation and verification: All software should be beta tested before releasing it to design agencies.	**Agree in principle.** The 1-40D team will alpha test new versions of the software, but there are no funds available to conduct beta testing. This could be done as part of Version 1.0 if funds are provided, or it may be a task best left to AASHTOWare.	1.0?

Recommendations of 1-40A(02) Rigid Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
R-4	Design manual editing: Manuals accompanying the MEPDG software must be carefully edited so that statements in the manuals are consistent with the software.	**Agree.** Text changes necessitated by corrections or enhancements of the software in Version 0.900 will be included in the supplement to the design guide planned at the end of Version 0.900. A full review of the consistency of text and software is desirable, but will require funds above what is now planned for Stages 0.900 and 1.0.	1.0?
R-5	Develop regional/state calibrations for JPCP: Prominently display ranges of values and regions of the U.S. used when developing the calibration coefficients.	**Agree.** The 1-40D team strongly supports the concept of regional or state calibration of the MEPDG. Such recommendations were made in the 1-37A documentation. Such an effort is under way in several states at this time, and Project 1-40B is developing a manual to guide such activities. The display of regional values used to develop regional calibration coefficients is a good idea and will be done in Version 1.0 (in coordination with Project 1-40B).	1.0
R-6	JPCP curling/warping and permanent warping: Give special attention to curling and warping during and after hardening. Improve the estimation of the Effective Permanent Curl/Warp value to account for additional factors known to affect this parameter.	**Agree.** In Project 1-37A, significant effort was spent in trying to accomplish this recommendation. The 1-40D team will make a major effort to develop a better procedure to estimate this input during the design stage. Insufficient data as to construction conditions for the LTPP and other calibration sections limit the ability to completely fulfill this recommendation. However, this recommendation can be partially addressed in Version 1.0 or later by renewing efforts to relate the permanent curl/warp parameter to prevalent construction, materials, and climatic conditions as available data permit.	1.0 or later
R-7	JPCP, Evaluation of climatic effects: The methods by which climatic conditions are used in the design procedure and the relationship between curling and warping must be carefully documented. Among the factors to consider:	**Agree.**	0.900 and/or 1.0
	1. More precisely explain the stresses and conditions that cause top-down and bottom-up cracking and the means to separate these distresses in the design process.	1. A more detailed account will be included in the supplement provided with Version 0.900.	
	2. Discuss how creep was taken into account in early-stage warping and curling process.	2. Creep is indirectly considered in the permanent curl/warp value because it is based on long-term performance of JPCP and CRCP which gives the slabs the opportunity to settle into the base course.	

(*continued*)

Recommendations of 1-40A(02) Rigid Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
	3. More thoroughly explain curl stresses in slabs.	3. A more thorough explanation will be included in the supplement.	
	4. Separate and display the warping, curling, and load-associated stresses as an optional output for the designer.	4. Complete separation is not possible because, to make the process computationally efficient, load, thermal, and moisture stress computations are intertwined in the MEPDG software.	
R-8	CRCP, Validation and Streamlining of Input Data: Justify and validate the inputs required in the design procedure. Procedure requires substantially more inputs than current design procedures. Not all inputs are fully justified or validated. Make efforts to classify inputs assigning levels of varying importance or significance to each, so that the design process can be simplified.	**Agree.** This recommendation can be carried out in full. A fuller explanation of inputs and their effect and significance will be prepared as part of the supplement to the design procedure. Regarding the number of inputs for CRCP as compared with the current AASHTO procedure, the following results were found: AASHTO: 94 inputs MEPDG: 130 inputs This comparison does not include axle load distribution for either the AASHTO guide or the MEPDG and monthly vehicle volume adjustments for the MEPDG. The increase in the number of inputs for the MEPDG is not dramatic considering the analytical power it affords the designer. Some inputs have already been “grayed out” or made more difficult to change. These include permanent curl/warp and shrinkage inputs. Others that meet the suggested criteria will also be identified and either grayed out or made difficult to change.	0.900 and 1.0
R-9	CRCP, Thickness Design Consistency: In some instances, thickness requirements obtained using the MEPDG are not consistent with current practice. This issue needs to be resolved or reasons for the differences adequately expressed.	**Disagree.** The conclusion that CRCP thickness is too great is based on one reference prepared by Roesler and Kohler at the U. of Illinois. The reviewers state that this reference concludes that “the thickness of CRCP is as much as 30 percent greater than slabs for JPCP.” Communication with Drs. Roesler and Kohler determined that neither of them had made such a comparison and that the statement attributed to them is incorrect. They prepared an illustrative design of a CRCP for very heavy traffic in the Chicago area and obtained a thickness of 14 in., similar to what IDOT is building for very heavy traffic loads. We verified their CRCP results and then used the same inputs for a JPCP pavement with the result that a 15-in. JPCP slab would be needed. Many other direct comparisons of CRCP and JPCP have been made and CRCP is generally thinner or occasionally equal to JPCP. Other comparisons of CRCP thickness were made between the AASHTO guide and the MEPDG. The AASHTO method always gives a thicker CRCP	0.900 and 1.0

Recommendations of 1-40A(02) Rigid Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		design (as it does with JPCP). Texas DOT sponsored a major sensitivity analysis of CRCP and found no issues with thickness. Missouri DOT compared JPCP and CRCP for a given project and determined that a 10-in. CRCP was approximately equivalent to a 12-in. JPCP. Thus, there appears to be no evidence that the MEPDG produces unreasonable thickness results. However, this issue does indicate a need to reduce the standard error of model prediction as much as possible (the punchout model has a relatively high standard error) and major efforts will be made in the 1-40D recalibration effort to improve the CRCP design method so that better predictions will be made and the standard error reduced.	
R-10	CRCP, Zero-Stress Verification: The zero-stress temperature (permanent curl/warp) input has a significant effect. Guidelines must be provided to assist the designer in selecting the appropriate input for this variable for different conditions and locations. Factors that affect this input include concrete temperature during hydration, climatic conditions at time of placement, and quantity and type of cement.	**Agree.** There are two inputs mentioned here: zero stress (temperature at set) and permanent curl/warp. The zero-stress temperature is critical to CRCP and both are critical to JPCP. Indeed, this recommendation is similar to R-6 above for JPCP. During Project 1-37A, major resources were spent in trying to accomplish this recommendation for JPCP, but similar resources were not available for CRCP. The 1-40D team will make a major effort to estimate this input specifically for CRCP. Our ability to do this is limited, however, by insufficient data on construction conditions for LTPP section and other calibration sections.	1.0?
R-11	CRCP, Alternative Failure Mechanisms: Punchout mechanisms other than the one in the design procedure need to be considered, evaluated, and validated.	*Note: the MEPDG specifies that crack widths should be < 0.02 in, crack LTE > 95%, and crack spacing < 6 ft over the design life. If these criteria are met, punchouts will rarely be a major problem.* The MEPDG includes the classic punchout mechanism identified many years ago as the major distress type in CRCP by several researchers based on field and theoretical observations. The mechanism calculates the transverse tensile bending stress at the top of the CRC slab between the truck wheels (of an axle) placed between two narrowly spaced transverse cracks. The transverse cracks may (<95% LTE) or may not (>95% LTE) be deteriorated. The rectangular piece of PCC may or may not punch down into the base. More than 80% of the so-called "punchouts" for sections used to calibrate the punchout model did *not* punch down, but simply showed the longitudinal fatigue crack (top down).	1.0 or later

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Recommendations of 1-40A(02) Rigid Design Team (Continued)

Area-#	Recommendation	1-40D Team Response	Version
		The 1-37A team took great pains in putting together the performance data for the original calibration; this included obtaining and reviewing year-by-year distress maps from each LTPP section considered and manually verifying the database. In this review the team observed that the predominant form of distress is the classical longitudinal crack between two transverse cracks, usually 3 to 5 ft from the edge of the slab. Occasionally this rectangular piece of PCC punched down into the base forming the classic punchout. However, Y-cracks and "ladder" cracks/punchout clusters were also observed, albeit not as commonly. The punchout count from the LTPP and other databases used in the calibration also included Y-cracks that had punched down (medium to high severity). Other mechanisms such as single deteriorated transverse cracks were not included in the count. The 1-40A reviewers identified one punchout mechanism they thought was not included in the MEPDG: ". . . some recent research results that indicate loss of load transfer is not the only condition for the development of punchouts. Under controlled test conditions it has been shown that punchouts can develop even though there is no measurable loss in load transfer across the cracks." **Agree.** Transverse cracks do not need to deteriorate; permanent deformation or a major settlement of the foundation can lead to a longitudinal crack in the CRCP. However, the MEPDG does *not* assume that transverse cracks must deteriorate. Fatigue damage is accumulated at the top of the slab with every load application even if no crack deteriorates. This mechanism is already included in the MEPDG. In summary, this recommendation for consideration of other punchout mechanisms is reasonable and can be done in Version 1.0 or later if the specific mechanisms can be identified. The supplement will include a clearer discussion of the conditions where transverse cracks do not deteriorate and of other potential punchout mechanisms. A change of terminology from punchout to "short longitudinal fatigue crack" may also be helpful since the former term has created confusion.	

Recommendations of 1-40A(03) Team on Flexible Design

Area-#	Recommendation	1-40D Team Response	Version
F-SW-1	Omit the top-down cracking distress prediction computations and await the results of ongoing research on this subject.	**Disagree.** Will recalibrate for Version 0.900. Leave it in, but disabled with the option to turn it on in the analysis. No other viable model available. Much more data is now available.	N/P
F-SW-2	Improve the application of soil mechanics principles to the characterization of soils and unbound materials, and, in particular, omit the use of CBR in design.	**Agree in principle.** Continued improvement in the treatment of soils and unbound materials is both necessary and desirable. However, the MEPDG uses modulus as the input, not CBR.	N/P
F-SW-3	Put the rut depth prediction procedures in abeyance and replace with good HMA mixture design combined with a permanent deformation test and the introduction of allowable stress levels for the granular layers and subgrade.	**Disagree.** While a sound HMA mix design is always required, there is a serious downside to putting the rut depth prediction in abeyance. Such an approach will tend to overestimate the required layer thickness, and it is not compatible with warranty or PRS applications. Further, the Version 0.900 recalibration with cleaned-up and larger LTPP and other datasets will substantially improve the model correlation and further reduce its standard error.	N/P
		Disagree. The use of allowable stress levels for granular layers and subgrade is similar to the CBR method in which a great enough layer thickness is used to reduce the stress level below the allowable limit. While this methodology can be added to the MEPDG as a separate, optional design criteria if directed by the panel, it is not recommended.	N/P
F-SW-4	Provide in the software for input of traffic data from the program developed under NCHRP Project 1-39.	**Agree.** This capability is planned for incorporation in Version 1.0.	1.0
F-SW-5	Introduce the FWD back-analysis subroutine into the software to deal with non-linearity of the lower layers of the pavement.	**Agree.** However, the Project 1-37A panel directed that no specific back-calculation computational method be used. If resources are available, future work can incorporate a general ARA interface program that is licensed for use to AASHTOWare; this program includes several available back-calculation methods.	1.0 or later
F-SW-6	Improve the guidance to users of the Guide within a more succinct, user-friendly version of the document.	**Agree in principle.** However, no resources are currently available to carry out this effort. [*Note: A project to accomplish this recommendation is tentatively planned for FY 2007.*]	N/P
F-SW-7	In revising the Guide for use in the short term, a more balanced approach is needed in which the various elements of design are approximately of the same level of complexity.	**Agree in principle.** The Project 1-37A panel directed a flexible "mix and match" approach in which the designer can choose the same or different levels of complexity for the various design elements. At the most basic level (Level 3), default values can be used exclusively, but combining Level 2 and 3 values or Level 1 and 2 values will reduce the error and increase the reliability of designs.	N/P

Recommendations of 1-40A(03) Team on Rigid Design

Area-#	Recommendation	1-40D Team Response	Version
R-SW-1	JPCP and CRCP New Designs: Include longitudinal and corner cracking as distress mechanisms and provide improved advice with regard to raveling.	**Agree.** The 1-37A research team was directed to not include corner cracking. The current MEPDG framework can easily incorporate a corner cracking model; this distress is significant in non-doweled JPCP but does not occur in properly doweled JPCP. Longitudinal cracking is a potentially significant type of distress that could be added to the JPCP analysis. This distress appears to occur in dryer climates in the western United States. Longitudinal cracking initiating at the top of the slab is directly calculated in the CRCP design procedure; the critical transverse stress location is identified and fatigue damage is accumulated at this point. It is certainly possible to apply a similar algorithm to JPCP. Additional resources would be required to accomplish this.	1.0 or later
R-SW-2	JPCP and CRCP New Designs: Review the permanent warp assumptions when a granular base is used.	**Agree.** This excellent suggestion will be carried out during the Version 0.900 recalibration. The effect of base course type (including granular base) on the permanent curl/warp parameter for JPCP and CRCP will be directly considered during the recalibration effort. It is possible that the parameter will be reduced when a softer base course is used.	0.900
R-SW-3	JPCP and CRCP Rehabilitated Designs: Improve procedures for structural evaluation of concrete pavements. Include longitudinal cracking as one of the distresses considered in pavement evaluation.	**Agree.** The relevant design guide text will be modified in the supplement to include longitudinal cracking of JPCP in the pavement evaluation process.	0.900
R-SW-4	JPCP and CRCP Rehabilitated Designs:	**Agree to all.**	0.900
	Correct error noted in Equation 3.2.28, which forms part of the computation of load transfer efficiency (LTE) between slabs and across cracks.	Equation is being reviewed and will be corrected as needed.	
	Review determination of dowel LTE contribution, where there may be an error in the equations leading to very high LTE values under all conditions.	Equations are being reviewed and will be corrected as needed.	
	Check the advice on computation of base LTE contributions (Tables 3.4.8 and 3.7.20).	A check of base LTE contribution will be made to ensure reasonableness.	

Recommendations of 1-40A(03) Team on Rigid Design (Continued)

Area-#	Recommendation	1-40D Team Response	Version
R-SW-5	JPCP and CRCP, Check and correct the detail concerning base erodibility, upward curl, and overburden on subgrade in relation to the computations for faulting:	**Agree to all.** These items are being checked and corrected as needed. A new base erosion model has been developed that considers the loss of support under CRCP more realistically. The definition of base erosion will be made clearer for JPCP.	0.900
	In Part 2, base erodibility is described in terms of Class rather than Factor.		
	A unit is required for upward curl.		
	The unit for overburden on subgrade should be appropriate for pressure (psi) rather than force (lb).		
R-SW-6	JPCP and CRCP, Shortcomings in the calibration procedures for rehabilitation models:	**Agree in principle,** but the available data are what they are.	0.900 and 1.0
	Calibration work for JPCP restoration should be carried out using data from sites covering a wider range of conditions.	We will search for additional data for restoration projects. However, all relevant data from LTPP and other sources was used for this purpose in Project 1-37A.	
	Further data are needed on the performance of CRCP overlays to existing PCC pavements for heavily trafficked sites.	Project 1-40B identified some additional CRCP overlay performance data from Illinois from both experimental sites and one in-service project.	
	Prediction of transverse cracking and joint faulting in JPCP overlays does not appear to match the available evidence particularly well, and the issue should be investigated further, both by seeking additional evidence and by carrying out a wider range of comparative designs. This may eventually identify areas where the model could be improved.	The recalibration of new JPCP will hopefully provide improved modeling that may show better prediction for JPCP overlays.	

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APPENDIX A: BRIEF FOR REVIEW TEAMS

Note: The detail given below was the brief for the SWPE team. The other teams had the same general terms of reference with differences of detail relating to their specific tasks.

BACKGROUND

At the request of the AASHTO Joint Task Force on Pavements (JTFP), NCHRP initiated Project 1-37A in 1996 to develop a guide for the design of new and rehabilitated pavement structures. In contrast to the current AASHTO Guide for Design of Pavement Structures, the guide recommended in 2004 by the Project 1-37A research team is based on mechanistic-empirical (M-E) principles, provides a uniform basis for the design of flexible, rigid and composite (rehabilitated) pavements and employs common design parameters for traffic, subgrade, environment and reliability.

A key component of the JTFP's plan for implementation and adoption of the M-E pavement design guide and software is an independent, third-party review to test the design guide's underlying assumptions, evaluate its engineering reasonableness and design reliability and identify opportunities for its implementation in day-to-day design production work.

OBJECTIVE

Conduct an independent engineering review of the mechanistic-empirical pavement design guide and software developed in NCHRP Project 1-37A.

WORK PROGRAM

Accomplishment of the project objective will require the following tasks:

Task 1. Plan and conduct an independent engineering review of the mechanistic-empirical pavement design guide developed in NCHRP Project 1-37A, with specific emphasis on (1) the methodology employed in the guide to estimate the reliability of pavement designs and (2) its use for the design of composite (rehabilitated) pavements. In the plan make use, insofar as possible, of existing data and results from completed or ongoing studies.

This will be accomplished through the following sub-tasks:

Task 1.1. Assess the reasonability, soundness and completeness of the guide's supporting engineering concepts and its process and procedures.

Task 1.2. Conduct an appraisal of the consistency and sensitivity of results for composite (rehabilitated) pavement designs.

Task 1.3. Evaluate the design reliability methodology used in the guide.

Task 1.4. Conduct an analysis of whether the distress models in the guide can be calibrated to match the predicted performance of select pavement structures with their known historical performance.

Task 1.5. Review the completeness with which the final design guide and software address the review comments of the NCHRP 1-37A panel and the JTFP.

Task 1.6. Assess the availability, clarity, and completeness of the necessary test protocols for materials.

Task 1.7. Assess the opportunities for agency implementation of the design guide and software in routine, day-to-day design production work.

Task 2. Assess the clarity, ease of use, capabilities, speed, and stability of the design software and identify opportunities in the conceptual operation and structure of the design process, including the use of the environmental model outputs, to decrease the overall runtime of the software.

Task 3. Meet with the NCHRP project panel at the beginning of the review and at its midpoint to present an update on progress, key issues, principal findings, and recommendations.

Task 4. Prepare a final report that presents key findings and recommendations, identifies specific deficiencies and errors in the design guide and software and suggests corrective actions, including short-term research activities.

Task 5. Prepare a concise summary report of the key findings and recommendations of Projects 1-40A (01), 1-40A (02), and 1-40A (03). [Contract 1-40(04) was introduced after these terms of reference had been issued].

APPENDIX B: MEMBERSHIP OF REVIEW TEAMS

Contract 1-40A(03): **Scott Wilson Pavement Engineering, Ltd.**

Team Leader: **Professor Stephen Brown**

Team Members:
Mr. Robert Armitage
Dr. Bachar Hakim
Prof. Andrew Collop
Dr. Nicholas Thom
Dr. Costanzo Graffi
Dr. Gordon Airey

Brief: Pavement Rehabilitation. Design Reliability. Executive Summary report for whole project.

Contract 1-40A(01)

Team Leader: **Prof. Marshall Thompson**

Team Members:
Prof. Sam Carpenter
Prof. Barry Dempsey
Prof. Bob Elliott

Brief: Flexible pavement design.

Contract 1-40A(02)

Team Leader: **Prof. Ernest Barenberg**

Team Members:
Dr. Jamshid Armaghani
Dr. Halil Ceylan
Dr. Shiraz Tayabji
Dr. Moon Won

Brief: Rigid pavement design.

These digests are issued in order to increase awareness of research results emanating from projects in the Cooperative Research Programs (CRP). Persons wanting to pursue the project subject matter in greater depth should contact the CRP Staff, Transportation Research Board of the National Academies, 500 Fifth Street, NW, Washington, DC 20001.

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