Lessons Learned from MEPDG Development: A Confession of the JPCP MEPDG Developers

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Abstract
After 15 years from the initiation of the National Cooperative Highway Research Program (NCHRP) 1-37A project, the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under that study has been adopted by the American Association State Highway and Transportation Officials (AASHTO) and is in the process of adoption by many transportation agencies in the United States and abroad. Although it may take some time until the MEPDG completely replaces current empirical procedures, it is important to reflect on the lessons learned from the MEPDG development. The MEPDG was never envisioned as the "final answer" for all the pavement design problems, but rather an important milestone in the continuous improvement of the pavement design methodology. The MEPDG was supposed to reflect the state of the practice of the late 1990s. The MEPDG documentation includes volumes of appendixes that provide information for researchers, but some information that is important for future improvements of the MEPDG has never been recorded. This paper describes some of the experiences and lessons learned from the development of the MEPDG. This experience may be useful in the development of future M-E designs for jointed plain concrete pavements (JPCP).

Introduction to Mechanistic-Empirical Rigid Design

The history of improvements to concrete pavement design has followed two major tracks: the development of mechanistic models and the development of empirical design equations. The early mechanistic-based design procedures for rigid pavements utilized Westergaard's model (Westergaard 1926, Westergaard 1927) to determine structural responses such as stresses and deflections. Westergaard's model treats a pavement as a medium-thick Kirchhoff plate resting on a Winkler (spring) foundation. Westergaard derived solutions for the various loading conditions such as a wheel load applied at the slab interior, edge, or corner. He also obtained solutions for curling of infinite and semi-infinite slabs subjected to a linear temperature gradient throughout the slab's thickness. To develop these solutions Westergaard introduced significant simplifications, such as infinite slab size, single slab (no effect of the adjacent slabs), single placed layer (no base or subbase), single wheel load (no multiple wheel axles), and uniform support (no separation of the concrete slab from the subgrade) (Ioannides et al. 1999).

Figure 1 illustrates the nondimensionalized stresses and deflections are plotted against the ratio of wheel radius to relative stiffness, *a/l*. The definition of these terms can be found elsewhere, including Ioannides et al. (1999). An analysis of Figure 1 (using Westergaard solutions) shows that the critical concrete tensile stresses occur at the bottom of the slab under edge loading, and the largest deflections occur under corner loading. As a result, early mechanistic-based design procedures aimed to

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reduce cracking by limiting the ratio of edge loading stresses to concrete strength and reduce faulting by limiting corner deflections. This in turn created a heavy emphasis on PCC thickness and strength in those early rigid pavement designs.

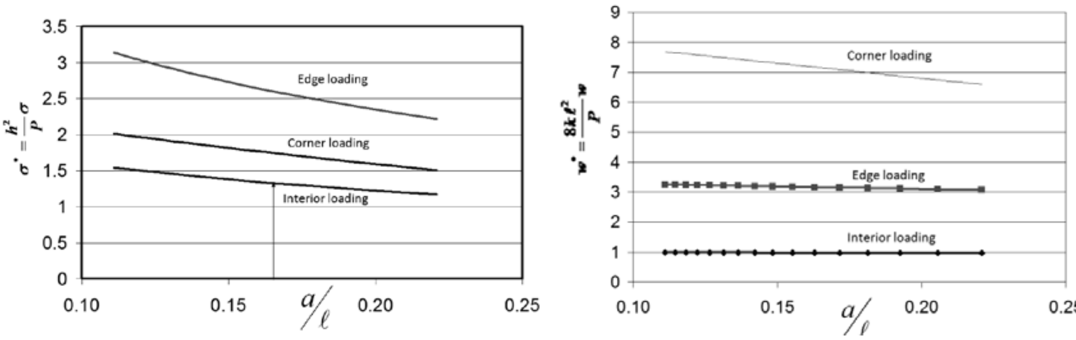

Figure 1. Non-dimensional Westergaard solutions in graphical form for critical stresses (at left) and critical deflections (at right)

It was later discovered that the presence of stabilized bases improved pavement performance, which was accounted for through "bumping" k-value, a concept which mathematically leads to decrease of predicted critical stresses and deflections in the pavements. However, this did not properly describe the structural behavior of the pavement system and reduces the true significance of the base course as well as ignoring slab/base friction.

The introduction of finite element programs for rigid pavement analysis such as KENSLAB (Huang 1993) and ILLI-SLAB (Tabatabae and Barenberg 1980) and subsequent improvements of ILLI-SLAB over almost two decades (Ioannides 1984, Korovesis 1990, Khazanovich 1994) allowed researchers to address important limitations of the Westergaard model and the effect of base layer contributions to performance. This involved the analysis of multi-slab, multi-layered rigid pavements subjected to a combined action of a nonlinear gradient and multi-wheel loading. Nevertheless, in spite of improvements to the structural model, the concept of limiting bottom-surface edge stresses due to heavy-axle edge loading and stresses and deflections due to corner loading remained unchanged until recently in the MEPDG.

Another important development in rigid pavement design was the creation of the American Association of State Highway Officials (AASHO) Road Test at a test site in Ottawa, Illinois. Test traffic was inaugurated in 1958 on 7 miles of two-lane pavements in the form of six loops and a tangent, half concrete, half asphalt. The 836 test sections employed a wide range of surface, base, and subbase thicknesses. The AASHO Road Test ended November 30, 1960. The test data established the relationships for pavement structural designs based on expected loadings over the life of a pavement. The AASHO Road Test provided the foundation for analytical evaluation of stresses and deflections from moving vehicles.

Based on the results of the AASHO Road Test, an interim AASHTO pavement design guide was developed in 1962 that went through major revisions in 1972, 1986, and 1993 (AASHTO 1972, 1986, 1993). Unlike the mechanistic concept of limiting structural responses (from Westergaard), the serviceability concept was introduced, which developed an empirical relationship between design parameters (e.g. PCC thickness, flexural strength, site conditions (coefficient of subgrade reactions), and traffic) and the loss of serviceability over time.

Until recently, this procedure was used by the majority of state and local transportation agencies in the United States. The results of the AASHO Road Test were used for the development of other design procedures. For example, AASHO Road Test data were used by the PCA in the development of the erosion concept and calibration of a design procedure that incorporated the erosion concept. This concept generalized Westergaard's concern for corner deflections. The PCA design procedure was used by U.S. and especially foreign transportation agencies (Packard 1977).

The various editions of the AASHTO Guide for Design of Pavement Structures have served for several decades; however, many serious limitations exist including the following.

Traffic loading and reliability. The original Interstate pavements were designed for 5 to 10 million equivalent single axle loads, whereas today these same pavements must be redesigned for 50 to 200 million axle loads, and sometimes more. The existing AASHTO Guide cannot be used reliably to design for this level of traffic. Road Test pavements sustained slightly over 1 million axle load applications—less than the traffic carried by many modern pavements within the first year of their use—and the equations forming the basis of the earlier procedures were based on regression analyses of the AASHO Road Test data.

Climatic effects. Because the AASHO Road Test was conducted at one specific geographic location, it is impossible to address the effects of different climatic conditions on pavement performance.

Subgrade variability. One type of subgrade was used for all test sections at the Road Test. For example, at the Road Test nearly all of the JPCP failed from severe pumping and erosion of the unbound aggregate base course. Many types exist nationally that result in different performance of highway pavements.

Base courses. Only one unbound dense granular base/subbase material was included in the main rigid pavement sections of the AASHO Road Test. These exhibited significant loss of modulus due to frost and erosion. Today, various stabilized base types are used routinely, especially for heavier traffic loadings. Unbound aggregate bases are also used routinely but are provided with improved drainage and joint load transfer to limit deflections.

Construction practices and drainage. Pavement designs, materials, and construction were representative of those used at the time of the Road Test. No subdrainage was included in the Road Test sections, but positive subdrainage has become common in today's highways.

Joint load transfer and spacing. All AASHTO JPCP pavements included dowel bars that had diameters 1/8 the thickness of the PCC slab. None of these joints faulted significantly. The subsequent revisions to the AASHTO Design Guide made it possible to design a JPCP without dowels by making the slab thicker. Nothing could have been worse! This single "improvement" is responsible for the joint faulting failure of thousands of miles of JPCP. Similarly, all joints were spaced at 15-ft at AASHTO but many agencies used longer spacings that resulted in much greater transverse cracking of the slabs.

Design life. Because of the short duration of the Road Test, the long-term effects of climate and aging of materials were not addressed. The AASHO Road Test was conducted over 2 years, while the design lives for many of today's pavements are 20 to 60 years.

These deficiencies are especially important given that the AASHTO procedures were empirical, and extrapolations of AASHTO design equations beyond the AASHO Road Test data may lead to unreliable results. For example, for very heavy traffic, the required PCC thickness prescribed by AASHTO93 design is unrealistically high. The design of thicker JPCP to allow for the removal of dowels ultimately resulted in joint faulting, erosion, and greater slab cracking.

During the development of the 1986 AASHTO Guide, it was recognized that future design procedures would be based on mechanistic-empirical principles (see Part IV). At that time, the available PCA rigid design procedure could not meet that need as it did not predict pavement distresses such as cracking and faulting. Also, it did not account for climatic effects and the structural model was overly simplistic.

In 1997, the NCHRP initiated Project 01-37A to develop a modern M-E design procedure. ERES Consultants – later ARA, Inc. – was selected as the prime contractor to conduct the research of the project, with Mr. John Hallin as the project's principal investigator (NCHRP 2004). ERES Consultants were responsible for the rigid portion of the M-E design and the overall software development. Dr. Matt Witczak of the University of Maryland (later Arizona State University) led the flexible M-E design effort. The remainder of the paper will describe the experiences of the JPCP design efforts under 01-37A.

NCHRP 01-37A: Development of a Mechanistic-Empirical Procedure for New and Reconstructed Pavements

Team Vision The 1-37A team vision was guided by a number of basic principles. First, the 1-37A team sought to apply validated, state-of-the -practice technologies. 1-37A also aimed to provide an equitable design basis from the standpoint of pavement type selection. This means that inputs for both rigid and flexible designs should be as similar as possible. For example, both designs should be characterized similarly in terms of traffic, subgrade, reliability, and climate. A traffic spectrum approach to traffic loading was adopted in favor of the flawed equivalent single-axle concept. It was recognized that the 21st century design procedure would use powerful design software rather than paper charts. At that time, the idea was that investing more efforts in material, traffic, and climate characterizations would result in better performance predictions and therefore more economical design.

Technology preceding 1-37A

For more than a decade prior to the start of 1-37A, members of the 1-37A team were actively involved in the improvement of performance models for key distresses for faulting, cracking, and IRI. While the initial models were purely empirical, over time they became more and more mechanistic. For example, the JPCP transverse cracking model evolved from the semi-empirical RIPPER models to the more rational PAVESPEC models (Smith et al. 1990, Yu et al. 1998a, Hoerner et al 2000). NCHRP 1-30 also provided improved structural FE modeling and the AASHTO 1998 rigid pavement design procedure (Darter et al. 1995). A similar progression took place in the faulting model. With co-sponsorship from Minnesota and Michigan DOTs, ERES developed ISLAB2000, which enabled more efficient and sophisticated structural modeling of rigid pavements (Khazanovich et al. 2000). Finally, as the primary contractor on the LTPP project, ERES had intimate knowledge of the LTPP database, which helped significantly in the validation and calibration of 1-37A M-E models. In fact, without the massive amount of LTPP field behavior and performance data the MEPDG could not have been calibrated or validated.

Immediate challenges in NCHRP 01-37A

ESAL to Load Spectra It should be noted, however, that the majority of the old performance models used ESALs to characterize traffic loading. The project anticipated difficulties in recalibrating models from ESALs to axle spectra, however, the required efforts and complexity of this issue was dramatically underestimated. The main drawback of the ESAL concept is that it assumes that damage from various axle types and axle weights is the same for all types of distresses (e.g. JPCP cracking and faulting, CRCP punchouts). The ESALs were also based on only one site location and pavement design. Other models that were important and also needed to be incorporated (such as cracking deterioration models) used ESALs, which assumed the same relative damage.

The use of axle spectra instead of ESALs freed 1-37A of this limitation, but it required the introduction, development, and calibration of cumulative damage models individually for each type of distress on a month-by-month basis. This was a major undertaking that significantly increased the complexity of the 1-37A effort. Use of the spectral approach also eliminated the different equivalency factors between flexible and rigid pavements that caused much confusion over 40 years. Now the use of "Number of Trucks" in the design lane over the design life as a "ball park" index of traffic level is far easier to understand and is the same between pavement types.

Subgrade Characterization (E-to-k Conversion)

As stated above, one of the requirements for 1-37A was the use of the same inputs for both flexible and rigid projects for subgrade, traffic, and climate. This required that a subgrade lab measured or estimated resilient modulus be input for both types of pavements which was a major challenge (Darter, et al, 2006). The main structural model for flexible pavements is the assumption of an elastic solid half-space. Although such a model is perfectly appropriate for flexible pavement models, it does not offer any advantages for rigid design. At the same time, the assumption of an elastic half-space significantly increases the computational demands for the solution of a rigid system.

The research team realized that neither Winkler nor the elastic half-space idealization were entirely adequate when applied to real soils, and that the predictions from both exhibit discrepancies with observed in-situ behavior. The Winkler model assumes no shear interaction at all between adjacent spring elements and results in a foundation parameter, k, which is sensitive to the size of the plate used in its determination. On the other hand, the elastic half-space model ascribes to the foundation a higher degree of shear interaction than is occurred in real soils, resulting in infinite stress predictions under the edges and corners of a plate resting on it. Furthermore, the Winkler foundation assumes zero deflections beyond the edge of the load plate, whereas the elastic half-space assumes a gradual decrease in deflection beyond the edges of the load plate. The deflection responses of real soils beyond the edge of the load plate are between the Winkler and elastic half-space model predictions. Figure 2 illustrates the various models for the slab and subgrade interaction.

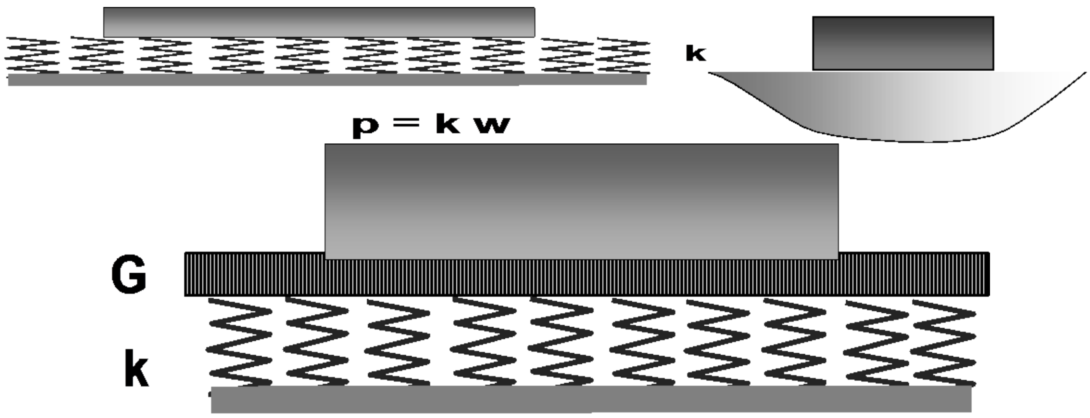

Figure 2. Winkler model (top-left) versus Elastic solid half-space (top-right) versus Pasternak model (bottom) under plate load

The initial approach of the research team was the use of the two-parameter (TP) model, often referred to as the Pasternak or Vlasov subgrade, to characterize the subgrade (Pasternak 1954, Vlasov and Leontiev 1966). The Pasternak model offers an attractive alternative to the ES continuum in providing a degree of shear interaction between adjacent soil elements. The Pasternak model consists of a shear layer placed on top of a Winkler spring foundation.

Several finite element programs developed for analysis of rigid pavement, such as ILLISLAB (Ioannides et al. 1985; Khazanovich, 1994), permit analysis using TP models. In the recommendations of the 4th International Symposium on Theoretical Modeling of Concrete Pavements to the 8th International Symposium on Concrete Roads, the Pasternak model was named the preferable option for subgrade modeling.

Nevertheless, the implementation of the Pasternak model for the subgrade resulted in significant difficulties:

1. All previous studies did not consider the effect of separation of the slab from the Pasternak subgrade. In some cases, this can result in numerical instabilities.
2. Modeling of the subgrade behavior under joints was found to be more complicated than anticipated.
3. It was recognized that k-values from the Pasternak models and Winkler models had to be different to provide the same stresses and deflections in the concrete slab. As many engineers in the United States had developed an intuition for k-values based on the Winkler representation of the subgrade, switching to the Pasternak model would cause addition confusion and possible design errors.

After weighing the advantages and disadvantages, the 1-37A project leadership team determined that while the Pasternak model could be considered for use in future M-E models, the technology was not appropriate for the incorporation of the Pasternak model into 1-37A at that time. The decision was made to use the Winkler model and develop a procedure to convert subgrade resilient modulus to subgrade k-value.

The equivalency “E-to-k” approach was a comparison of deflection basins with the elastic half-space and Winkler representations of the subgrade. However, direct incorporation of this concept was found to be inappropriate, as it resulted in unrealistically low k-values for a given subgrade. A close examination of the problem identified that this complication was caused by an inadequacy in the elastic half-space model: a real subgrade is a non-linear, stress sensitive material. As the subgrade stresses under the rigid pavement are lower than those under a thin flexible pavement, it is not appropriate to use the same resilient modulus for structural modeling. Moreover, the thicker the concrete pavement, the higher the Young's modulus that should be used for the same subgrade.

To address this challenge, the following multi-step procedure to convert the subgrade resilient modulus to a coefficient of subgrade reaction (k-value) was adopted.

1. Select a trial design consisting of all layers: PCC slab, base, embankment and/or subgrade (including bed rock) and appropriate moduli for each layer. Using the elastic layer program JULEA, simulate a 9,000-lb Falling Weight Deflectometer (FWD) load with the plate radius 5.9 in and compute PCC surface deflections at 0, 8, 12, 18, 24, 36, and 60 in from the center of the load plate.
2. Adjust the subgrade resilient modulus to account for the lowered deviator stress level beneath a PCC slab and base.
3. Using the elastic layer program JULEA and layer moduli and thicknesses again simulate a 9,000-lb FWD load with the plate radius equal to 5.9 in, and with the recalculated subgrade resilient modulus.
4. Calculate PCC surface deflections at 0, 8, 12, 18, 24, 36, and 60 in from the center of the load plate.
5. Use the Best Fit method to compute the dynamic modulus of subgrade reaction using the PCC surface (FWD) deflections (Khazanovich et al 2001).

The dynamic k-values that resulted from this process were found to agree with those obtained from FWD backcalculations of in-service pavements. Note that the "dynamic" k-value is approximately twice the traditional "static" k-value. The use of the Winkler foundation instead of the elastic half-space permitted computationally efficient procedures for the prediction of stresses and deflections in the rigid pavement. However, improvements of the subgrade characterization adopted by 1-37A are still warranted and could be the subject of future pavement research. Additional discussion on this issue can be found in elsewhere (Khazanovich et al 2006). In addition, the process of obtaining the proper subgrade resilient modulus design input using backcalculation or materials testing needs much further clarification.

Enhanced Integrated Climate Model (EICM)

One of the main challenges faced by the 1-37A project for both the flexible and rigid designs was the integration of the EICM into the cumulative damage calculations. In the past, some performance models such as the RIPPR models used the results of EICM simulations for climate characterizations, but this was conducted for a limited number of runs for fewer climatic regions and the most typical pavement design features. 1-37A decided to incorporate EICM directly and conduct month-by-month (day and night) simulations for each location and design feature as a part of the design process. This decision required significant modifications of EICM itself and the performance models, both in terms of model development and model calibration.

To characterize the climate for a given location, EICM relied upon regular climatic weather station data for a given location in five aspects: air temperature; wind speed; percent sunshine; precipitation; and relative humidity. Earlier versions of EICM used data for daily intervals, however at the time of the 1-37A, the EICM database was updated to hourly inputs. This resulted in higher than expected temperature differences between the top and the bottom of the PCC slab. The initial reaction was that the problems laid with EICM predictions; however, comparisons with MnROAD measured thermal gradients indicated that the EICM predictions were within reason. This forced the 1-37A leadership team to reconsider Westergaard's assumption of a linear temperature distribution through the PCC slab. 1-37A then adopted an approach developed by Ioannides and Khazanovich (1998) for the nonlinear distribution of temperature through the PCC slab.

Faulting Modeling

It was recognized by the 1-37A research team that significant corner deflections do not necessarily lead to faulting if the joints have high load transfer efficiency. This is why the differential energy (DE) concept, which accounts for deflections on both sides of the joint, was a natural mechanistic parameter for faulting damage accumulation (Khazanovich 2004). However, several important elements of the JPCP faulting model had to be modified during the 1-37A project work. The previous faulting models used simplified equations for the deflections of slab corners and did not consider the effect of slab curling. Also, the joint load transfer efficiency was characterized by a representative aggregate interlock, which did not vary over time. During the course of 1-37A, the LTE limitation was addressed through the modification of the crack deterioration model, developed by Zollinger et al (1999).

Computational Efficiency

1-37A adopted an incremental damage approach, which required the simulation of how pavement damage occurs in nature, incrementally, load by load, over continuous time periods. By accumulating damage semi-monthly or monthly, both daytime and nighttime, the design procedure becomes very versatile and comprehensive. The major advantages of the incremental damage accumulation approach were as follows:

- The design procedure accumulates damage similar to how it occurs in the field, incrementally.
- The increments (monthly for rigid pavement) are selected to match climatic (temperature and moisture) changes that cause changes in layer materials, changes in joint openings, changes in traffic loadings, and material aging and property changes over time.
- The effect of traffic loadings during daytime and nighttime (due to the reversal in temperature gradients) thus affecting the top and bottom of the slab can be considered.

This presented the 1-37A rigid and flexible design teams with the challenge of computing responses (stresses, strains, and deflections) at locations through the pavement system for various loads at each time increment. For this problem, the rigid and flexible design teams selected different approaches.

To realistically simulate rigid pavement responses, the 1-37A structural models considered over thirty input parameters (including PCC thickness, modulus of elasticity, Poisson's ratio, unit weight, and coefficient of thermal expansion; base thickness, modulus of elasticity, unit weight, and Poisson's ratio; coefficient of subgrade reaction; PCC temperature at 11 evenly spaced nodes in the PCC layer; joint spacing and load transfer efficiency of a PCC/shoulder joint; axle type, weight, tire pressure, lateral truck distribution, and wheel spacing). Although direct incorporation of a finite element code into the 1-37A rigid design procedure was possible (with some simplifications), this would have greatly increased the computational run-time required for a single design project since millions and millions of computation were now required over a long design life.

However, this incremental approach made the MEPDG a much more powerful design procedure since it could directly consider all of the variations mentioned above. Thus, the rigid team invested significant project resources into the development of accurate, computationally efficient stress and deflection predictions. This effort required the use of dimensional analysis and similarity concepts, which identified several key nondimensional parameters governing structural responses (such as ratio of joint spacing to radius of relative stiffness and the nondimensional Korenev's temperature gradient). Based on this rigorous mathematical formulation, it was shown that a solution of any given pavement system can be obtained from the solution of similar pavement systems with up to seven independent parameters. Neural networks were developed to predict stresses and deflections for those similar systems and closed-form coefficients relating stresses and deflections from the neural networks in the original systems were also obtained. To train the neural networks, several hundred thousand ISLAB2000 runs were performed. While this approach was challenging and tedious, it resulted in the accurate and nearly instantaneous calculation of slab responses in the analysis process.

A similar challenge was faced by the flexible design team, which required the computation of stresses and strains at each time increment for multiple locations through the system using layered elastic analysis. Rather than adopt the rigid approach, the flexible team directly incorporated a layered elastic analysis program, JULEA, into their approach for the solution of flexible pavement responses at each time interval. Although initially it required more than one hour a single project simulation, the flexible team viewed this run-time issue as one that would be resolved by advances in computing. This decision allowed the flexible team to avoid computational efficiency issues and focus on other challenges.

During the development of 1-37A, it was not clear to members of the project team which approach to the calculation of pavement responses was more appropriate in the long-term. However, after fifteen years it is clear that that the rigid team approach was preferable. The difference in computational run-time in the flexible and rigid design simulations was a matter of minutes versus seconds, and while it was an inconvenience but tolerable in a routine design process, the longer run time for flexible projects caused significant concerns for industry and in the design community. The need for computational efficiency in the structural response modeling – rather than hoped for improvements in computing – was acknowledged in AASHTO DARWin-ME efforts to optimize software to reduce run time for both flexible projects and rigid projects.

The difference in run-time for 1-37A was a far more serious issue in terms of the calibration process, when hundreds of thousands of simulations must be conducted in order to effectively calibrate the design models using available pavement data. This example provided a very important lesson for the future developers of M-E design procedures: the computational efficiency of the models is an important consideration. This point should not be overlooked if a comprehensive calibration of the models and design process is a major goal.

Unexpected (Development) Challenges

Although at the beginning of the project, the 1-37A team had no illusions about the difficulty of the project work, several unexpected challenges were encountered in the course of the project work. The following subsections describe a few of the more notable challenges in closer detail.

Top-down cracking When the 1-37A project was initiated, it was generally accepted that fatigue cracking in JPCP initiated at the bottom of the slab and propagated to the top (bottom-up cracking). This theory was based on the Westergaard model. A forensic study conducted by ERES Consultants under a separate project resulted in major revisions to the 1-37A procedure.

In the early 1990s, the Pennsylvania DOT reconstructed I-80 with 13.0-in. JPCP using 20-ft joint spacing. Although the expected design life was 40 years, several years after reconstruction the observed slab cracking generally began as a hairline transverse crack located at or near midslab, and the cracks generally became of high severity after only a few months, with the greatest amount of deterioration occurring during the fall months (when joints and cracks tend to open more). Pavement coring indicated that the cracks appeared to start at the top of the slab and progress downward (top-down cracking). Forensic studies ruled out obvious causes of the failure, such as inadequate PCC strength or thickness.

The main challenge of the distresses observed at I-80 was the fact that the cracks were top-down. This contradicted Westergaard's theory that bottom surfaces stresses are always the most critical. Indeed Westergaard's edge loading stresses, calculated for the slab bottom, are always greater than Westergaard's corner loading stresses, calculated at the slab top surface. Measured day-time temperature gradients are greater than night-time thermal gradients, which results in greater Westergaard curling tensile stresses at the bottom of the slab than at the top. This made a mechanistic explanation of this distress a very challenging task.

Fortunately, a recently developed program for the finite element analysis of rigid pavement responses, ISLAB2000, permitted a large number of simulations to determine conditions under which the observed phenomenon might exist (Beckemeyer et al. 2002). As a result of this investigation, the research team identified that two Westergaard assumptions must be reconsidered:

1. Westergaard's assumption that a slab is flat in the absence of a temperature gradient or wheel load should be reconsidered in light of built-in curling. Built-in slab curling can be caused by both differential shrinkage or a thermal gradient during final set of the concrete (Yu et al. 1998b). This may significantly increase tensile stresses at the top surface and decrease tensile stresses at the bottom. Extensive research conducted by Armaghani and Tia at the University of Florida , the Technical University of Munich, and the University of Chile beginning in the 1980s defined built-in curling and the significant upward curling of nearly all PCC slabs.
2. Westergaard originally considered only a single wheel load in his solutions. Later, it was generalized by the analysis of an axle load. Although this analysis is valid for bottom surface stresses, it was found

that an analysis of only an axle load is insufficient in the calculation of critical top surface stresses for upward curled slabs.

After these assumptions were reconsidered, it was found that top surface stresses can be comparable to those at the bottom of the slab; moreover, if the critical bottom stresses are caused only when the axle load is applied immediately at the slab edge, high top surface stresses can be caused even from truck application some distance from the slab edge. The fact that many if not nearly all slabs are curled upward most of the time also contributes to increased fatigue damage on the top. Calibration of the fatigue transverse cracking for JPCP revealed that on average, a -10F linear gradient through the slab resulted in minimizing the standard error of estimation across the calibration database. This value was termed Permanent Curl/Warp Effective Temperature Difference. However, it is obvious that every paving section has its own value based on thermal gradient at final set (depends mainly on solar radiation and curing) plus differential (irreversible) shrinkage at early ages of the surface of the slab. This topic needs far more study and improvement.

Although analysis of the top-down cracking was not in the initial scope of the 1-37A project, it was clear to the 1-37A team that the I-80 experience along with the above mentioned findings from other studies could not be overlooked. Therefore the 1-37A team decided to address this issue. The 1-37A team was able to incorporate this work into the rigid design process, and as a result for sufficiently thick rigid designs, top-down cracking controls the design and not bottom-up.

Calibration of Distress Models

The mechanistic-empirical models for fatigue cracking and joint faulting required calibration against field data. This had been done before by the rigid pavement team for the cracking and faulting models developed in the 1970s, 80s, and 90s. However, the extent of the LTPP and other data now available was an order of magnitude greater than what was previously available. The collection, extraction, cleaning and placing of this data into a database for use in the calibration effort was far more extensive than expected. However, having such a widely diverse database of sections for JPCP from a large majority of States and a few Canadian Provinces proved to be one of the most important aspects of validating the MEPDG design procedure. This database also made it possible to provide extensive defaults for all MEPDG inputs that were realistic and had been measured across North America. Without this extensive data, the models would not have been calibrated and validated very well and may have not been accepted by the reviewers of the procedure and the designers.

Reliability

From the beginning of the 1-37A project, reliability was recognized as an important part of the future design process. Even though mechanistic concepts provide a more accurate and realistic methodology for pavement design, a practical method to consider the uncertainties and variations in design and construction was needed so that a new or rehabilitated pavement could be designed for a desired level of reliability. The approach used in the AASHTO 1993 was seriously flawed and could not be used in the MEPDG (Darter et alo. 2005).

A Monte Carlo-based analysis was envisioned as the most promising tool to allow reliability analysis for rigid pavement distress predictions. However, it was found to be too computationally demanding to be combined with the incremental damage design. The rigid team developed a procedure combining Monte Carlo simulation with a damage accumulation procedure which would achieve acceptable execution time without jeopardizing the accuracy of the analysis (Darter et al. 2005). However, due to the longer computational time for flexible the approach could not be used because both rigid and flexible approaches had to be similar.

A less computationally demanding and practical approach to reliability was adopted for both rigid and flexible pavements. The reliability of each distress and IRI was considered separately and the standard error of prediction was based directly on the prediction model calibration error. The higher the error of prediction based on calibration data, the greater the structural capacity of the pavement to achieve design reliability. The approach did not account for the effect of design input variability on the design reliability which could be accomplished by the Monte Carlo simulation. At this time, the current version of the MEPDG does not permit the quantification of benefits of more accurate inputs in improving the reliability of the design. Therefore, improvement of the reliability analysis must be a major focus for future research in M-E design improvements.

Software development

As was stated above, from the beginning of the 1-37A project, it was recognized that the design process would require the use of software. However, the complexity of software development for 1-37A purposes was significantly underestimated. In the early stages of the project work, the rigid team mainly concentrated on the technical aspects of the design process. Toward the latter stages of 1-37A, it became evident that an inadequate user interface for the software would seriously jeopardize the wide adopted of the 1-37A M-E procedure.

To better judge the user interface, the 1-37A software interface was compared with that of the ISLAB2000 (Khazanovich et al. 2000). This comparison clearly indicated deficiencies in the usability of the 1-37A software. As a result, the 1-37A rigid team recommended the complete redesign of the user interface for the design program. The 1-37A leadership approached a private consultant, Efim Shats, who was involved in the development of the ISLAB2000 pre-processor program. Mr. Shats developed a prototype of the user interface of the new design program which incorporated many ideas from the ISLAB2000 interface. The main concept of this user interface remained in all subsequent version of the MEPDG software, and this improved interface contributed to the wide adoption of the MEPDG upon its public release.

A large complex software program like the MEPDG would be a good candidate for open sourcing. This would have allowed a more rapid identification of program bugs as well as allowing other researchers easier access to the source code to develop new and improved models.

Instead of Conclusions… ("What's Next in M-E Design?")

Many States and Canadian Provinces are now implementing and using the DARWin-ME design procedure, which manifests the success of the MEPDG as developed from the work of NCHRP 01-37A. However, the current models and algorithms were never considered as a final solution, just a platform for future developments. The developers hope that this will be kept in mind and that AASHTO will not hesitate to continue to refine and improve the MEPDG/DARWin-ME procedures. For example, in the last several years, significant improvements have been made to accommodate the effect of dowel misalignment on pavement performance and the design and performance of composite PCC-PCC pavements under the SHRP2 R21 project. These changes can be incorporated into DARWin-ME in the near future.

As noted, the MEPDG platform is very flexible and specific deficiencies can be improved and incorporated into the DARWin-ME software through continuing research. Some possible future research efforts encountered during the work of 1-37A that can still be addressed are as follows.

Longitudinal cracking model. Currently, the MEPDG considers only transverse cracking and distress models. However, it is quite likely that longitudinal cracking may become the primary mode of failure in the case of projects that use of widened lanes and thinner slabs. Therefore, the development of a longitudinal cracking model should be a high priority for the M-E modeling of rigid pavements. Significant progress has been made in this direction at the University of Illinois (Hiller and Roesler 2008), but more remains to be done.

Permanent curl/warp equivalent temperature difference. As stated above, the MEPDG was the first M-E procedure to incorporate the concept of built-in curl as well as drying shrinkage differential. Sensitivity analyses identified this as an important parameter. However, the recommended magnitude was obtained through field calibration rather than mechanistic analysis. Several approaches have been explored for the estimation of a built-in curling parameter, but currently there is no universally accepted method available. The development of such a procedure to be incorporated into design practice will significantly improve the accuracy of rigid design.

Slab-base interaction. The use of JULEA by 1-37A to backcalculate subgrade k-value brought an unexpected consideration into the MEPDG design procedure. This was the ability to consider friction between the slab and base course. This ability was limited to full friction (no slippage) and zero friction, however, it showed a very large effect on pavement damage calculations. The slab/base friction had not been considered prior to this except for it incorporation into the NCHRP 1-30 AASHTO 1998 Supplemental Design procedure. Subsequent calibration for slab fatigue cracking has shown that most base courses exhibit high friction but that when attempts are made to “break” the bond for CTB or LCC base courses, the loss of friction has dire consequences on future slab fatigue cracking.

Rehabilitation designs. During the course of the 1-37A research, the majority of the efforts were dedicated to new design. Although these models form a valid basis for a rehabilitation design procedure, the rehabilitation designs can be improved. Considering that most of the future work in the United States and Canada will be rehabilitation and not new construction, this direction is important for future research in rigid pavements and M-E design.

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