THE MONISMITH LECTURE
ESTABLISHED
BY
ASCE GEO-INSTITUTE
PAVEMENTS COMMITTEE
Carl L. Monismith, Dr. Eng., P.E.
The Robert Horonjeff Professor of Civil Engineering - Emeritus

Director Emeritus
Pavement Research Center
Institute of Transportation

University of California
Berkeley
INTERNATIONAL CONFERENCE ON THE STRUCTURAL DESIGN OF ASPHALT PAVEMENTS

AUGUST 20-24, 1962

UNIVERSITY OF MICHIGAN
ANN ARBOR, MI
M-E FLEXIBLE PAVEMENT DESIGN: CHALLENGES AND ISSUES

Marshall R. Thompson, Ph.D., P.E.
Professor Emeritus

Department of Civil Engineering
University of Illinois @ U-C
AASHO ROAD TEST
DECISION TIME
DEFLECTION OR SN ???
• “The performance of the flexible pavements was predicted with essentially the same precision from load-deflection data as from load-design information.” (SN)

• “Deflections taken during the spring when the subsurface conditions were adverse gave a better prediction of pavement life than those taken in the fall.”

• “There was high degree of correlation between deflection and rutting.”
1959 SPRING NORMAL DEFLECTIONS

\[
\log W_{2.5} = 9.4 + 1.32 \log L - 3.25 \log d
\]

\[
\log W_{1.5} = 10.18 + 1.36 \log L - 3.64 \log d
\]

L – Axle Load (kips)

d – deflection (mils)
Log $W_{2.5}$

AASHO ROAD TEST
AASHO ROAD TEST
SOME EARLY M-E DESIGN EFFORTS

ANN ARBOR CONFERENCE – 1962

ASPHALT INSTITUTE – AIRPORT PAVEMENTS
ANN ARBOR CONFERENCE - 1972

SHELL PAVEMENT DESIGN MANUAL
(ANN ARBOR CONFERENCE – 1977)
(PUBLISHED – 1979)

SOUTH AFRICAN PROCEDURE
(ANN ARBOR CONFERENCE – 1977)

ASPHALT INSTITUTE : MS-11 1981

1986 AASHTO GUIDE - PURSUE M-E

AASHTO M-E (2007)

AASHTO Ware Pavement M-E (2013)

OTHER USA / INTERNATIONAL EFFORTS
(IL DOT – 1989 – FULL-DEPTH HMA)
M.W. Witczak
Staff Engineer @ Asphalt Institute

“Design of Full-Depth Asphalt Airport Pavements”

3rd International Conference on the Structural Design of Asphalt Pavements
London – 1972

ASPHALT INSTITUTE
Thickness Design – Asphalt Pavements for Air Carrier Airports
ASPHALT INSTITUTE M-E

* 1977: INITIATED M-E EFFORTS

* 1981: MS-1
THICKNESS DESIGN- ASPHALT PAVEMENTS
FOR HIGHWAYS AND STREETS
DAMA - ELP COMPUTER PROGRAM
Components of a Mechanistic Design Procedure

Inputs
- Material Characterization
  - Paving Materials
  - Subgrade Soils
- Traffic
- Climate

Structural Model

Pavement Responses: $\sigma, \varepsilon, \Delta$

Transfer Functions

Pavement Distress / Performance

Final Design

Design Reliability
RESILIENT MODULI INPUTS
FOR PRACTICAL
MECHANISTIC PAVEMENT DESIGN

1999 TRB
Resilient Modulus and
Mechanistic Pavement Design:
Are We There yet??
$E_R = \frac{\sigma_D}{\varepsilon_R}$
FINE-GRAINED SOILS

“STRESS SOFTENING”
\[ E_R = \sigma_D / \varepsilon_R \]

FINE-GRAINED

\[ \approx 6 \text{ psi} \]

\( \sigma_D \) - DEVIATOR STRESS
Modulus @ Deviator Stress of 6 psi
ARITHMETIC MODEL
SEMI-LOG MODEL

\[ \text{LOG } E_{R_i} = a - (b \times \delta_D) \]
SEMI-LOG MODEL

Optimum $W = 0.4\%$
$\log E_R = 1.182 - 0.021 \sigma_D$

Optimum $W = 1.8\%$
$\log E_R = 1.029 - 0.037 \sigma_D$
The generalized model used in MEPDG design procedure is as follows:

\[ M_r = k_1 P_a \left( \frac{\theta}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \]

where

- \( M_r \) = resilient modulus, psi
- \( \theta \) = bulk stress
- \( \sigma_1 + \sigma_2 + \sigma_3 \)
- \( \sigma_1 \) = major principal stress.
- \( \sigma_2 \) = intermediate principal stress
- \( \sigma_3 \) = minor principal stress
- confining pressure
- \( \tau_{oct} \) = octahedral shear stress
- \( \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2} \)
- \( P_a \) = normalizing stress
- \( k_1, k_2, k_3 \) = regression constants
DEGREE OF SATURATION EFFECTS
FREEZE-THAW EFFECTS
$E_{Ri}$ INPUTS

* $E_R$ TESTING

* FWD & B/C
  (PEDOLOGY - SOIL SERIES)

* ESTIMATES
  - STRENGTH
    ($Q_u$, CBR, DCP)
  - $\sigma/\varepsilon$ (PURDUE)
  - % CLAY/PI
  (PEDOLOGY - SOIL SERIES)

* TYPICAL VALUES
SUBGRADE VARIABILITY

* COVs >> 50% COMMON FOR FWD BACKCALCULATED MODULI !!!

* LAB TESTING VARIABILITY ???
ILLI-PAVE DEFAULT SUBGRADE S
Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design

A Synthesis of Highway Practice
SUBGRADE MODULUS = ???
GRANULAR MATERIALS

“STRESS-HARDENING”
GRANULAR MATERIAL MODULI ARE STRESS DEPENDENT

\[ M_R = K_1 \theta^{K_2} \]
## Typical Theta Model Parameters

Table 2.3  Typical values for $k_1$ and $k_2$ for unbound base and subbase materials ($M_R = k_1 \theta k_2$).

<table>
<thead>
<tr>
<th>Moisture Condition</th>
<th>$k_1^*$</th>
<th>$k_2^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>6,000 - 10,000</td>
<td>0.5 - 0.7</td>
</tr>
<tr>
<td>Damp</td>
<td>4,000 - 6,000</td>
<td>0.5 - 0.7</td>
</tr>
<tr>
<td>Wet</td>
<td>2,000 - 4,000</td>
<td>0.5 - 0.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Moisture Condition</th>
<th>$k_1^*$</th>
<th>$k_2^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>6,000 - 8,000</td>
<td>0.4 - 0.6</td>
</tr>
<tr>
<td>Damp</td>
<td>4,000 - 6,000</td>
<td>0.4 - 0.6</td>
</tr>
<tr>
<td>Wet</td>
<td>1,500 - 4,000</td>
<td>0.4 - 0.6</td>
</tr>
</tbody>
</table>

* Range in $k_1$ and $k_2$ is a function of the material quality.
<table>
<thead>
<tr>
<th>Moisture State</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>$8000 \theta^{0.6}$</td>
</tr>
<tr>
<td>Damp</td>
<td>$4000 \theta^{0.6}$</td>
</tr>
<tr>
<td>Wet</td>
<td>$3200 \theta^{0.6}$</td>
</tr>
</tbody>
</table>

**AASHTO BASE**

<table>
<thead>
<tr>
<th>Moisture State</th>
<th>Developed Relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damp</td>
<td>$M_R = 5400 \theta^{0.6}$</td>
</tr>
<tr>
<td>Wet</td>
<td>$M_R = 4600 \theta^{0.6}$</td>
</tr>
</tbody>
</table>

**AASHTO SUBBASE**
ALL DATA
(271 Data Points)
LOG K = 4.657 –1.807*n

$E_R = K \theta^n$

$\log K = A + b^n$ (271 Data Points)

A’s and b’s for the various materials

Legend:
A  Silty Sands
B  Sand Gravels
C  Sand Aggregate Blends
D  Crushed Stone
E  Limerock
F  Slag

# Recommended Theta Values (psi) Base Course

<table>
<thead>
<tr>
<th>Asphalt Concrete Thickness (inches)</th>
<th>Roadbed Soil Modulus (psi)</th>
<th>Resilient Modulus (psi)</th>
<th>Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3,000</td>
<td>7,500</td>
<td>15,000</td>
</tr>
<tr>
<td>Less than 2</td>
<td>20</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>2 - 4</td>
<td>10</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>4 - 6</td>
<td>5</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>Greater than 6</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>
Stress states (θ) which can be used as a guide to select the modulus value for subbase thicknesses between 6 and 12 inches are as follows:

<table>
<thead>
<tr>
<th>Asphalt Concrete Thickness (inches)</th>
<th>Stress State (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 2</td>
<td>10.0</td>
</tr>
<tr>
<td>2 - 4</td>
<td>7.5</td>
</tr>
<tr>
<td>greater than 4</td>
<td>5.0</td>
</tr>
</tbody>
</table>
The generalized model used in MEPDG design procedure is as follows:

\[ M_r = k_1 P_a \left( \frac{\theta}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \]

where

- \( M_r \) = resilient modulus, psi
- \( \theta \) = bulk stress
- \( \sigma_1, \sigma_2, \sigma_3 \) = major principal stress, intermediate principal stress, minor principal stress
- \( \tau_{oct} \) = octahedral shear stress
- \( P_a \) = normalizing stress
- \( k_1, k_2, k_3 \) = regression constants
Study of LTPP Laboratory Resilient Modulus Test Data and Response Characteristics: Final Report

FHWA-RD-02-051
A. Yau & H. Von Quintus
B. Fugro-BRE, Inc.
n+1: Lower Layer

BARKER & BRABSTON
FAA-RD-74-199 (1975)
Field measurements and theoretical considerations have indicated that the dynamic modulus of an unbound vase layer ($E_2$) must be related to the modulus of the subgrade ($E_3$)."

The following relationship is utilized:

$$E_2 = k \times E_3$$

$$k = 0.2 \times h_2^{0.45}$$

$2 < k < 4$

$h_2$ – thickness of the granular layer (mm)
<table>
<thead>
<tr>
<th>$T_{AC}$, in.</th>
<th>$E_{AC}$ (ksi)</th>
<th>100</th>
<th>500</th>
<th>1,400</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td></td>
<td>33.5</td>
<td>31.4</td>
<td>29.5</td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td>30.4</td>
<td>26.4</td>
<td>23.3</td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td>25.0</td>
<td>21.5</td>
<td>18.9</td>
</tr>
<tr>
<td>8.0</td>
<td></td>
<td>21.7</td>
<td>18.2</td>
<td>16.5</td>
</tr>
</tbody>
</table>

$$E_R = 9000 \theta^{0.33}$$

(9000 lbs. = 80 psi)
E (ksi) = 37.8 – (5.7 * [LOG ET³ / 100])
R² = 0.98     SEE = 0.9 ksi
ILLI-PAVE ANALYSES

* HMA SURFACE :
  + 4-6-8 INCHES
  + MODULUS = 500 ksi

* 10-INCH GRANULAR BASE :
  + Mr (psi) = 5000 * θ^0.5
  + Φ = 45°

* SUBGRADE SOIL
  + SOFT: ERi = 3 ksi / Qu = 13 psi
  + MEDIUM: ERi = 7.7 ksi / Qu = 23 psi
  + STIFF: ERi = 12.3 ksi / Qu = 33 psi

* LOADING: 9 kips @ 80 psi (Typical FWD)
### SUBGRADE & HMA THICKNESS EFFECTS

<table>
<thead>
<tr>
<th>HMA (ins)</th>
<th>SUBGRADE ERi (ksi)</th>
<th>SURFACE DEFLECTION (mils)</th>
<th>BASE MOD MID-PT/AVERAGE (ksi)</th>
<th>THETA* (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>SOFT/3</td>
<td>30.3</td>
<td>18.4/19.3</td>
<td>14.9</td>
</tr>
<tr>
<td>4</td>
<td>MEDIUM/7.7</td>
<td>23.7</td>
<td>22/22.5</td>
<td>20.2</td>
</tr>
<tr>
<td>4</td>
<td>STIFF/12.3</td>
<td>20.1</td>
<td>23.2/23.6</td>
<td>22.2</td>
</tr>
<tr>
<td>6</td>
<td>MEDIUM/7.7</td>
<td>17.2</td>
<td>17.6/17.4</td>
<td>12.4</td>
</tr>
<tr>
<td>8</td>
<td>MEDIUM/7.7</td>
<td>13.4</td>
<td>14.6/14.8</td>
<td>8.5</td>
</tr>
</tbody>
</table>

* THETA IS FOR THE AVERAGE BASE MODULUS
## OFFSET EFFECTS / SOFT SUBGRADE
(4-inch HMA)

<table>
<thead>
<tr>
<th>OFFSET (inches)</th>
<th>MID-PT MODULUS (ksi)</th>
<th>THETA - θ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>18.4</td>
<td>13.5</td>
</tr>
<tr>
<td>9</td>
<td>16.8</td>
<td>11.3</td>
</tr>
<tr>
<td>12</td>
<td>14.3</td>
<td>8.2</td>
</tr>
<tr>
<td>15</td>
<td>12.2</td>
<td>6</td>
</tr>
<tr>
<td>18</td>
<td>12</td>
<td>5.8</td>
</tr>
<tr>
<td>22</td>
<td>11.1</td>
<td>4.9</td>
</tr>
<tr>
<td>26</td>
<td>9.5</td>
<td>3.6</td>
</tr>
<tr>
<td>31</td>
<td>8.2</td>
<td>2.7</td>
</tr>
<tr>
<td>36</td>
<td>7.1</td>
<td>2</td>
</tr>
<tr>
<td>42</td>
<td>6.3</td>
<td>1.6</td>
</tr>
<tr>
<td>MATERIAL NC DOT DATA</td>
<td>R² THETA MODEL</td>
<td>R² UZAN MODEL</td>
</tr>
<tr>
<td>---------------------</td>
<td>---------------</td>
<td>---------------</td>
</tr>
<tr>
<td>1</td>
<td>0.992</td>
<td>0.998</td>
</tr>
<tr>
<td>2</td>
<td>0.998</td>
<td>0.999</td>
</tr>
<tr>
<td>3</td>
<td>0.993</td>
<td>0.999</td>
</tr>
<tr>
<td>4</td>
<td>0.992</td>
<td>0.998</td>
</tr>
<tr>
<td>5</td>
<td>0.994</td>
<td>0.994</td>
</tr>
<tr>
<td>6</td>
<td>0.996</td>
<td>0.999</td>
</tr>
<tr>
<td>7</td>
<td>0.996</td>
<td>0.998</td>
</tr>
<tr>
<td>8</td>
<td>0.994</td>
<td>0.994</td>
</tr>
<tr>
<td>9</td>
<td>0.992</td>
<td>0.994</td>
</tr>
<tr>
<td>10</td>
<td>0.989</td>
<td>0.989</td>
</tr>
</tbody>
</table>

**THETA:**  \( M_R = K_1 \Theta^{K_2} \)  

**UZAN:**  \( M_R = K_1 \Theta^{K_2} \times (\sigma_D)^{K_3} \)

**BOTH MODELS CAPTURE THE STRESS HARDENING EFFECT**
HMA MODULUS

* $E_{\text{HMA}}$ IS INFLUENCED BY TIME OF LOADING AND TEMPERATURE

* MUST BE CONSIDERED IN M-E PAVEMENT DESIGN!!

* EXTENSIVE PAST R&D ON THE ISSUE

* RECENT FHWA PUBLICATION IS AN EXCELLENT Reference
STRUCTURAL MODELS

* ELASTIC LAYER PROGRAMS
  (MANY OPTIONS / MEPDG – JULEA)

* AXYSYMERIC FINITE ELEMENT
  (STRESS DEPENDENT Es)
  (FAILURE CRITERIA)
  (USE SUPERPOSITION)
  (AVAILABLE IN EARLY VERSION MEPDG)

* 3-D FINITE ELEMENT
  (COMPUTATIONALLY INTENSIVE)

* NEURAL NETWORKS
ADDITIONAL DESIRABLE STRUCTURAL MODEL FEATURES

• ANISOTROPY

• RESIDUAL STRESSES
TRANSFER FUNCTIONS

CRITICAL FACTORS!!!
SUBGRADE RUTTING
SUBGRADE TRANSFER FUNCTIONS

• SUBGRADE VERTICAL STRAIN

• SUBGRADE STRESS RATIO (SSR)
  SSR = DEV STRESS / Q_u
VERTICAL STRAIN CRITERIA

\( \varepsilon = L \ (1/N)^m \)

<table>
<thead>
<tr>
<th>AGENCY</th>
<th>L</th>
<th>m</th>
<th>RD (INS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AI</td>
<td>1.05*10^{-2}</td>
<td>0.223</td>
<td>0.5</td>
</tr>
<tr>
<td>SHELL</td>
<td>2.8*10^{-2}</td>
<td>0.25</td>
<td>PSI / 2.5</td>
</tr>
<tr>
<td>50%</td>
<td>2.8*10^{-2}</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>85%</td>
<td>2.1*10^{-2}</td>
<td>0.25</td>
<td>CROW</td>
</tr>
<tr>
<td>95%</td>
<td>1.8*10^{-2}</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>TRL/1132 (85%)</td>
<td>1.5*10^{-2}</td>
<td>0.253</td>
<td>0.4</td>
</tr>
</tbody>
</table>
“Therefore it is proposed to drop the subgrade strain criterion and rely on a single criterion that limits the flexural stress or strain at the underside of the base layer to a permissible level to achieve the required pavement life.”

NOTE: The procedure utilizes “foundation classes” and “equivalent elastic half spaces” to characterize the composite foundation support.

CL 1 => 50 MPa (7.3 ksi)  
CL 2 => 100 MPa (14.6 ksi)  
CL 3 => 200 MPa (29 ksi)  
CL 4 => 400 MPa (58 ksi)
Transfer Functions:
Subgrade Rutting-
Vertical Strain Design Criteria

\[ N = 10000 \times \left( \frac{0.000247 + 0.000245 \times \log_{10}(E_R)}{\varepsilon_v} \right)^{0.0658 \times E_R^{0.559}} \]

(Adapted from Barker and Brabston, 1975)

**VERTICAL COMPRESSION STRAIN AT TOP OF SUBGRADE, \( \varepsilon_v \times 10^{-3} \)**

**ANNUAL STRAIN REPETITIONS (20 YEAR LIFE)**

**E_s = 30,000 PSI**

- 15,000
- 9,000
- 3,000
CURRENT FAA SUBGRADE STRAIN CRITERIA

\[ C < 12,100: \quad C = \left(\frac{0.004}{\varepsilon_v}\right)^{8.1} \]

\[ C > 12,100: \quad C = \left(\frac{0.002428}{\varepsilon_v}\right)^{14.21} \]
SUBGRADE STRESS RATIO (SSR)

SSR = SUBGRADE DEVIATOR STRESS / $Q_u$
Transfer Functions: Subgrade Rutting-SSR

Influence of SSR on Permanent Deformation

Bejarano & Thompson (2001)

DuPont Clay

\[ q_u = 28 \text{ psi} \]
\[ \gamma_d = 98 \text{pcf} \]
\[ w = 26\% \]

UNSTABLE!!!

STABLE Behavior

Load Applications

Permanent Strain
Transfer Functions:
Subgrade Rutting-SSR

Permanent Deformation vs. SSR

Bejarano & Thompson (2001)
## Transfer Functions: Subgrade Rutting-SSR

### SSR General Guidelines

<table>
<thead>
<tr>
<th>Damage Potential...</th>
<th>Low/Acceptable</th>
<th>Limited</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SSR...</strong></td>
<td>0.5 / 0.6</td>
<td>0.6 to 0.75</td>
<td>&gt; 0.75</td>
</tr>
</tbody>
</table>

UNIVERSITY OF IL R&D
WES: BETA – COVERAGE – SSR RELATIONS

C: COVERAGE
SSR: SUBGRADE STRESS RATIO

\[ \log(\text{BETA}) = \frac{(1.7782 + (0.2397 \times \log(C)))}{(1 + (0.5031 \times \log(C)))} \]

\[ \text{BETA} = \frac{(3.14 \times \text{SUBGRADE VERTICAL STRESS})}{\text{CBR}} \]

\[ \text{CBR} \approx \frac{Q_U \text{ (psi)}}{4.5} \]

\[ \text{SSR} = \frac{\text{SUBGRADE VERTICAL STRESS}}{Q_U} \]

\[ \text{THUS: SSR} = \frac{\text{BETA}}{14.1} \]
<table>
<thead>
<tr>
<th>COVERAGES</th>
<th>BETA</th>
<th>SSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>13.34</td>
<td>0.946</td>
</tr>
<tr>
<td>1,000</td>
<td>9.89</td>
<td>0.701</td>
</tr>
<tr>
<td>10,000</td>
<td>8.10</td>
<td>0.574</td>
</tr>
<tr>
<td>20,000</td>
<td>7.72</td>
<td>0.548</td>
</tr>
<tr>
<td>50,000</td>
<td>7.30</td>
<td>0.518</td>
</tr>
<tr>
<td>100,000</td>
<td>7.02</td>
<td>0.498</td>
</tr>
</tbody>
</table>
HIGH ESAL PAVEMENTS

• SUBGRADE RUTTING – NORMALLY NOT A PROBLEM

• “WORKING PLATFORM” – ESSENTIAL FOR PAVING!!!!!
GRANULAR MATERIAL RUTTING
MINIMUM HMA "COVER THICKNESS"
MEPDG RUTTING MODEL
GRANULAR MATERIALS

Permanent Deformation Models
APPENDIX E
Review of Current Permanent Deformation Models

Typical Model Forms

\[ \varepsilon_p = a + b(\text{LOG } N) \]

\[ \varepsilon_p = A N^b \]

Ullidtz Model

\[ \varepsilon_p = a(\sigma_d/p_0)N^c \]
Tseng and Lytton (1989) presented a three-parameter permanent deformation model to predict the accumulation of permanent deformation through material testing. The parameters were developed from the laboratory established relationship between permanent strains and the number of load applications. The curve relationship is expressed as follows:

\[ \varepsilon_a = \varepsilon_0 e^{-\left(\frac{\rho}{N}\right)^\beta} \]

Where \(\varepsilon_a\) is the axial permanent strain; \(N\) is the number of load applications, \(\varepsilon_0\), \(\beta\), and \(\rho\) are material parameters that are different for each sample, and are determined based on the water content, resilient modulus, and stress states for base aggregate and subgrade soils through multiple regression analyses.
Pavement ME Rutting Damage Model

\[ \delta_a(N) = \beta_1 \left( \frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left( \frac{\rho}{N} \right)^\beta} \varepsilon_v h \]

- \( \delta_a(N) \) = Permanent deformation corresponding to \( N \) load applications
- \( \beta_1 \) = Field calibration factor
- \( \varepsilon_0, \rho \) = Material properties
- \( \varepsilon_r \) = Resilient strain from lab tests to determine material properties
- \( \varepsilon_v \) = Vertical resilient strain computed for sublayer
- \( h \) = Sublayer thickness
Framework for Improved Unbound Aggregate Base Rutting Model Development for M-E Pavement Design

93rd Annual Meeting of the Transportation Research Board

Liang Chern Chow
Debakanta (Deb) Mishra
Erol Tutumluer

University of Illinois at Urbana-Champaign
Pavement ME Rutting Damage Model

\[ \delta_a (N) = \beta_1 \left( \frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^\beta} \varepsilon_v h \]

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- \( \beta_1 \) = Field calibration factor
- \( \varepsilon_0, \rho \) = Material properties
- \( \varepsilon_r \) = Resilient strain from lab tests to determine material properties
- \( \varepsilon_v \) = Vertical resilient strain computed for sublayer
- \( h \) = Sublayer thickness

\[ f (N, \text{thickness}, M_R, W_c, \varepsilon_r) \]

No stress state!
# Aggregate Shear Strength Properties

<table>
<thead>
<tr>
<th>Label</th>
<th>Cohesion</th>
<th>Friction Angle</th>
<th>Compaction Water Content</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$c$</td>
<td>degree</td>
<td>%</td>
</tr>
<tr>
<td>psi</td>
<td>kPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material G1</td>
<td>12.4</td>
<td>85.1</td>
<td>50 $\omega_{opt} \pm 0.1$</td>
</tr>
<tr>
<td>Material G2</td>
<td>8.6</td>
<td>59.4</td>
<td>45 $\omega_{opt} \pm 0.8$</td>
</tr>
<tr>
<td>Material B</td>
<td>0.2</td>
<td>1.1</td>
<td>51 $\omega_{opt} \pm 0.2$</td>
</tr>
<tr>
<td>Material L</td>
<td>0.3</td>
<td>2.4</td>
<td>45 $\omega_{opt} \pm 0.1$</td>
</tr>
</tbody>
</table>

$\omega_{opt} = \text{Optimum moisture content}$

**N.C. DOT data**

![Graph showing shear stress and normal stress](image-url)
Shear Stress Ratio (SSR) Concept

Shear Stress Ratio = \frac{\text{Applied Shear Stress}}{\text{Shear Strength}} = \frac{\tau_f}{\tau_{max}}

\tau_{max} = c + \sigma_f \tan \phi

\sigma_f = \frac{2\sigma_3 + 2\tan^2 \phi \sigma_3 + \sigma_d + \tan^2 \phi \sigma_d - \sqrt{\tan^2 \phi \sigma_d^2 + \tan^4 \phi \sigma_d^2}}{2(1 + \tan^2 \phi)}

\tau_f = \sqrt{(\sigma_d / 2)^2 - (\sigma_f - (\sigma_3 + \sigma_d / 2))^2}
Repeated Load Triaxial Testing for Permanent Deformation Characterization
Specimen Preparation and Setup
Test Protocol

- Single-stage loading permanent deformation tests
  - 10,000 cycles at SSR = 25%
  - 10,000 cycles at SSR = 50%
  - 10,000 cycles at SSR = 75%

- Confining pressure = 34.5 kPa (5 psi)

- 150 mm × 150 mm specimen at OMC and MDD conditions
Permanent Deformation Test Results

\[ \varepsilon_p = A (N)^B \]
Development of Improved Rutting Model

\[ \varepsilon_p = A \ (N)^B \ (\sigma_d)^C \ (\tau_f / \tau_{\text{max}})^D \]

A, B, C, D = Regression parameters
\( \varepsilon_p \) = Permanent strain
\( N \) = Load cycle
\( \sigma_d \) = Applied stress
\( \tau_f \) = Applied shear stress
\( \tau_{\text{max}} \) = Shear strength at failure
\[ \varepsilon_p = A (N)^B (\sigma_d)^C \left( \frac{\tau_f}{\tau_{\text{max}}} \right)^D \]
STRENGTH PARAMETERS ARE IMPORTANT FACTORS IN PREDICTING PERMANENT DEFORMATION OF GRANULAR MATERIALS!!!
COMPLICATING FACTORS

• STRENGTH INCREASE WITH LOADING

• STRESS HISTORY EFFECTS
SHEAR STRENGTH INCREASE WITH REPEATED LOADING DENSE GRADED CRUSHED GRAVEL BASE
Thompson & Smith (TRR 1278)
STRESS HISTORY: HIGH TO LOW
CUMULATIVE DAMAGE

???
“AASHTO has recently determined that the current model for unbound pavement materials underestimates the structural impact of high quality aggregate base.”

“AASHTO encourages each licensing agency to calibrate and validate using local materials”


* NC DOT Project @ University of Illinois
\[ \Delta_{p(\text{HMA})} = \varepsilon_{p(\text{HMA})} h_{\text{HMA}} = \beta_{1r} k_z \varepsilon_{r(\text{HMA})} 10^{k_1 r} n^{k_2 r} T^{k_3 r} \]

where:

- \( \Delta_{p(\text{HMA})} \) = Accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.,
- \( \varepsilon_{p(\text{HMA})} \) = Accumulated permanent or plastic axial strain in the HMA layer/sublayer, in./in.,
- \( \varepsilon_{r(\text{HMA})} \) = Resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA sublayer, in./in.,
- \( h_{\text{HMA}} \) = Thickness of the HMA layer/sublayer, in.,
- \( n \) = Number of axle-load repetitions,
- \( T \) = Mix or pavement temperature, °F,
- \( k_z \) = Depth confinement factor,
- \( k_{1r,2r,3r} \) = Global field calibration parameters (from the NCHRP 1-40D recalibration; \( k_{1r} = -3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606 \)), and
- \( \beta_{1r,2r,3r} \) = Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

\[ k_z = (C_1 + C_2 D) 0.328196^D \]

\[ C_1 = -0.1039 \left( H_{\text{HMA}} \right)^2 + 2.4868 H_{\text{HMA}} - 17.342 \]

\[ C_2 = 0.0172 \left( H_{\text{HMA}} \right)^2 - 1.7331 H_{\text{HMA}} + 27.428 \]

where:

- \( D \) = Depth below the surface, in., and
- \( H_{\text{HMA}} \) = Total HMA thickness, in.
Figure 10. Comparison of Measured and Predicted Total Rutting Resulting from Global Calibration Process
“The objective of this research was to propose revisions to the HMA rut-depth transfer function in the MEPDG for consideration by NCHRP and the AASHTO Joint Task Force on Pavements.”

Carl Monismith was the Panel Chairman.
TRANSFER FUNCTIONS CONSIDERED

* Original MEPDG

* Verstraten \((\sigma_{\text{DEV}})\)

* Asphalt Institute - Modified Leahy
  \((\sigma_{\text{DEV}} \text{ and } \varepsilon_V)\)

  • WesTrack
    (shear strain and stress)
<table>
<thead>
<tr>
<th>TRANSFER FUNCTION</th>
<th>$R^2$</th>
<th>$S_e$ – in.</th>
<th>$S_e / S_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MEPDG</td>
<td>0.583</td>
<td>0.1085</td>
<td>0.665</td>
</tr>
<tr>
<td>Modified Leahy</td>
<td>0.699</td>
<td>0.1045</td>
<td>0.611</td>
</tr>
<tr>
<td>WesTrack</td>
<td>0.712</td>
<td>0.091</td>
<td>0.585</td>
</tr>
</tbody>
</table>
“With proper calibration, all four transfer functions accurately simulated the evolution of AC pavement rutting, and there were no statistically or practically significant differences among results obtained with the four functions. All of the transfer functions were calibrated to provide reasonable predictions of rut depth.”

REASONABLE PREDICTIONS ???

MEPDG DESIGN CRITERIA
Interstate: 0.40 in.
Primary: 0.50 in.
Others(< 45 mph): 0.65 in.
HMA RUTTING

* MATERIALS SELECTION (AGGREGATES – ASPHALT)

* MIXTURE DESIGN (SUPERPAVE)

* CONSTRUCTION QC/QA

RUT RESISTANT !!!
HMA  FATIGUE
NATIONAL HMA FATIGUE MODEL

Figure 11  Comparison of Cumulative Fatigue Damage and Measured Alligator Cracking Resulting from Global Calibration Process
AASHTO TP 8-94

Standard Test Method for Determination of the Fatigue Life of Compacted HMA Subjected to Repeated Flexural Bending
FATIGUE DESIGN

- Tensile Strain at Bottom of Asphalt
- Tensile Strain in Flexural Beam Test

Other Configurations
FATIGUE TESTING

• Tensile Strain in Flexural Beam Test
  – Other Configurations

– 10 Hz Haversine Load, 20° C, Controlled Strain
K1 = Intercept
K2 = Slope
AC FATIGUE

\[ N = K_1 \left( \frac{1}{\varepsilon_{\text{AC}}} \right)^{K_2} \]

K2' > K2
FATIGUE ALGORITHMS

\[ N_f = K1(1/\varepsilon)^{K2} \]

AASHTO MEPDG FORMAT

\[ N_f = 0.00432 * k_1 * C(1/\varepsilon)^{k2} \left(1/E_{HMA}\right)^{k3} \]

\( K_1 \) - HMA Thickness Factor

\( C \) – Mix Factor \((V_b \& V_a)\)

Beta Factors - Calibration

\((k2 = 3.9492 / k3 = 1.281)\)
IDOT HMA FATIGUE DATA SUMMARY
84 MIXES

\[ N = K_1 \left(1/\varepsilon\right)^{K_2} \]

Minimum \( K_2 \): 3.5
90% \( K_2 \): 4.0
Average \( K_2 \): 4.5
OTHER STUDIES

\[
\log K1 = \frac{1.1784 - K2}{0.329}
\]
THERE IS

NO "UNIQUE"

HMA FATIGUE ALGORITHM !!!!
IMPORTANT ISSUE FOR HMA OVERLAY DESIGN !!!!

REMAINING LIFE !!!!
FATIGUE ENDURANCE LIMIT
FEL
PERPETUAL PAVEMENT DESIGN

CRITERIA:

• HMA CUMULATIVE FATIGUE DAMAGE WILL NOT OCCUR

• PERIODIC MILL-FILL
Monismith & McLean

“Technology of Thick Lift Construction: Structural Design Considerations”

1972 AAPT Proceedings

70 Micro-Strain Endurance Limit!!
70 Micro Strain Test

University of Illinois

Failure @ Stiffness < 50%
FATIGUE ENDURANCE LIMIT

* Damage and Healing Concepts and Test Data Support a Strain Limit (the FEL) Below Which Fatigue Damage Does Not Accumulate

• FEL **Is Not The Same** for All HMAs.

• Carpenter – UofI
  21 HMAs / Range: 90 – 300 με/ AVG: 125
Michael Nunn
“Long-Life Flexible Pavements”
8th ISAP Conference
Seattle, WA - 1997
ASPHALT PAVEMENT ALLIANCE (2000)

“PERPETUAL PAVEMENTS”

Huddleston – Buncher – Newcomb
Design of Long-Life Flexible Pavements for Heavy Traffic

http:\\www.trl.co.uk
“Design Principles for Long Lasting HMA Pavements”

Thompson & Carpenter

ISAP Symposium
Design & Construction of Long Lasting Asphalt Pavements

Auburn, AL
June -2004
HMA FATIGUE

\[ N = K_1 \left( \frac{1}{\varepsilon_{AC}} \right)^{K_2} \]

ENDURANCE LIMIT

70 \( \mu \varepsilon^* \)

PERPETUAL PAVEMENT

*Monismith and McLean ('72 AAPT)
FEL = ???????

K1 & K2 = ????????
CURRENT NCHRP RESEARCH
NCHRP 9-38
Endurance Limit of HMA for Preventing Fatigue Cracking in Flexible Pavements (2010 – NCAT/AUBURN - RAY BROWN)

NCHRP 9-44
Developing a Plan for Validating an Endurance Limit for HMA Pavements (AAT- BONAQUIST - Completed)
* FEL IS NOT CONSTANT FOR A GIVEN HMA!!
* FEL VARIES WITH HMA MODULUS! (FEL SMALLER FOR HIGHER MODULUS)
* REST PERIODS ARE HELPFUL (RP > 2.5 SECONDS)
<table>
<thead>
<tr>
<th>HMA MODULUS (ksi)</th>
<th>FEL: $\mu$-STRAIN (RP – 1 SEC.)</th>
<th>FEL: $\mu$-STRAIN (RP – 5 SEC.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>46</td>
<td>122</td>
</tr>
<tr>
<td>600</td>
<td>37</td>
<td>102</td>
</tr>
<tr>
<td>1000</td>
<td>31</td>
<td>89</td>
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<tr>
<td>1500</td>
<td>27</td>
<td>80</td>
</tr>
<tr>
<td>2000</td>
<td>25</td>
<td>75</td>
</tr>
<tr>
<td>3000</td>
<td>21</td>
<td>66</td>
</tr>
</tbody>
</table>

HMA MODULUS RANGE – CHAMPAIGN, IL (10-INCH FULL-DEPTH)
PER: NCHRP 9-44A (BEAM TESTING)
NEW NCHRP PROJECT: 09-59

Binder Fatigue, Fracture, and Healing and Their Contribution to Hot-Mix Asphalt Fatigue Performance
IL PERSPECTIVE

* “HOTTEST MONTH” HMA MODULUS IS PROBABLY ADEQUATE FOR “PRACTICAL” PP DESIGN

* CRITICAL INPUT IS FEL
  FEL = ???
DESIGN RELIABILITY
RELIABILITY

STRUCTURAL RESPONSES
\((\sigma - \varepsilon - \Delta)\)

PAVEMENT DISTRESS(ES)

UTILIZE VARIABILITY IN MEASURED RESPONSES TO CONSIDER RELIABILITY
THE ONLY STRUCTURAL RESPONSE THAT CAN BE CONVENIENTLY MEASURED ON A "LARGE SCALE" IS SURFACE DEFLECTION!!!

FWD – RWD – TSD

VARIABILITY IN Δ AND "BASIN SHAPE PARAMETERS"
SOME SIGNIFICANT PARAMETERS

$\Delta_0$

$\text{SCI} = \Delta_0 - \Delta_{12}$

AUPP (AREA UNDER PAVEMENT PROFILE)
AUPP = \frac{(5*D_0 - 2*D_1 - 2*D_2 - D_3)}{2}

All Ds in mils
IDOT-FULL-DEPTH HMA

\[
\log \varepsilon_{\text{HMA}} = 1.53 \log \Delta_0 + 0.319
\]

\[
\log \text{SSR} = 1.28 \log \Delta_0 - 2.21
\]

(SSR = SUB DEV 6 / \(Q_U\))

\[
\log \varepsilon_{\text{HMA}} = 1.001 + 1.024 \log (\text{AUPP})
\]

\(\Delta_0 : \text{mils}\)

\(\varepsilon_{\text{HMA}} : \text{micro-strain}\)
IDOT-CONVENTIONAL FLEXIBLE PAVTS

\[
\text{LOG } \varepsilon_{\text{HMA}} = 1.113 \text{ LOG } \Delta_0 + 0.91
\]

\[
\text{LOG } \text{SSR} = 1.67 \text{ LOG } \Delta_0 - 2.88
\]
\[
(\text{SSR} = \text{SUB DEV } 6 / Q_U)
\]

\[
\text{LOG } \varepsilon_{\text{HMA}} = 0.999 + 1.014 \text{ LOG } (\text{AUPP})
\]

\[\Delta_0 : \text{mils}\]
\[\varepsilon_{\text{HMA}} : \text{micro-strain}\]
SUMMARY & OBSERVATIONS

* M-E DESIGN HAS SIGNIFICANTLY PROGRESSED SINCE THE 60`S AND CONTINUES TO EVOLVE/IMPROVE

* PERFORMANCE PREDICTIONS ARE NOT “CONSISTENTLY SATISFACTORY”

* CALIBRATION IMPROVES PERFORMANCE PREDICTIONS

* NEED TO CAPITALIZE ON THE ATTRIBUTES OF FINITE ELEMENT MODELS
  + ACCOMMODATE STRESS DEPENDENT MODULI
  + UTILIZE FAILURE CRITERIA
  + ACCOMMODATE ANISOTROPY
  + CONSIDER RESIDUAL STRESSES
  + RECONCILE LAB-FIELD DISCREPANCIES
* CONTINUE TO DEVELOP/REFINE MATERIAL CHARACTERIZATION PROCEDURES & MODELS (MODULUS – STRENGTH- FAILURE CRITERIA- FATIGUE)

* DEVELOP IMPROVED TRANSFER FUNCTIONS (RUTTING – FATIGUE – FATIGUE ENDURANCE LIMIT)

* DEVELOP IMPROVED CUMULATIVE DAMAGE MODELS
  
  • EVALUATE IMPACT OF STRESS HISTORY EFFECTS
  
  • REASONABLE EXPECTATIONS

WE ARE PROGRESSING!!!
KEEP UP THE GOOD WORK!!!
THOMPSON’S PRINCIPLES

DO NOT:

• Measure with a micrometer;

• Mark with a grease pencil; and

• Cut with an axe!!!!!!
THANK YOU !!!!!
“What is pavement design-pavement performance prediction reality? It would seem that only the naive, geniuses or the grossly egotistical would attempt to predict pavement performance. (The author - J. Brown- readily admits to the latter.) The pavement designer must forecast weather, traffic, and the results of a low bid contractor that uses such precise tools as bulldozers and draglines. The traffic forecast must include not only how many trucks but must include size of load and vehicle configurations, including tire pressures and types. Construction materials include those processed by Mother Nature (subgrades) and those semi-processed by the low bid contractor (base and subbase materials). The properties of these materials and the future loadings need to be known twenty-four hours a day, three hundred and sixty-five days per year for so far into the future that most pavement designers will retire before the design life has been reached!”