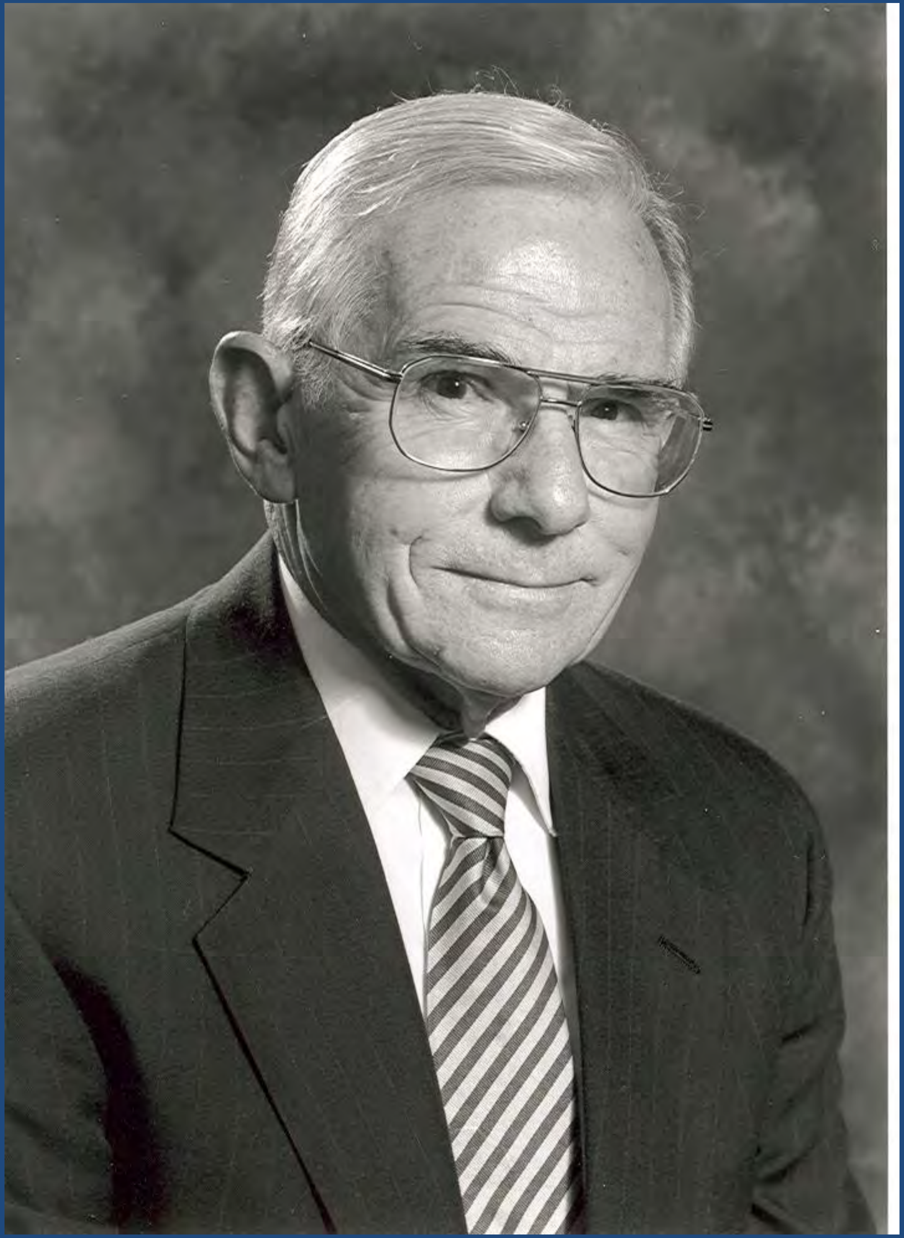
THE MONISMITH LECTURE

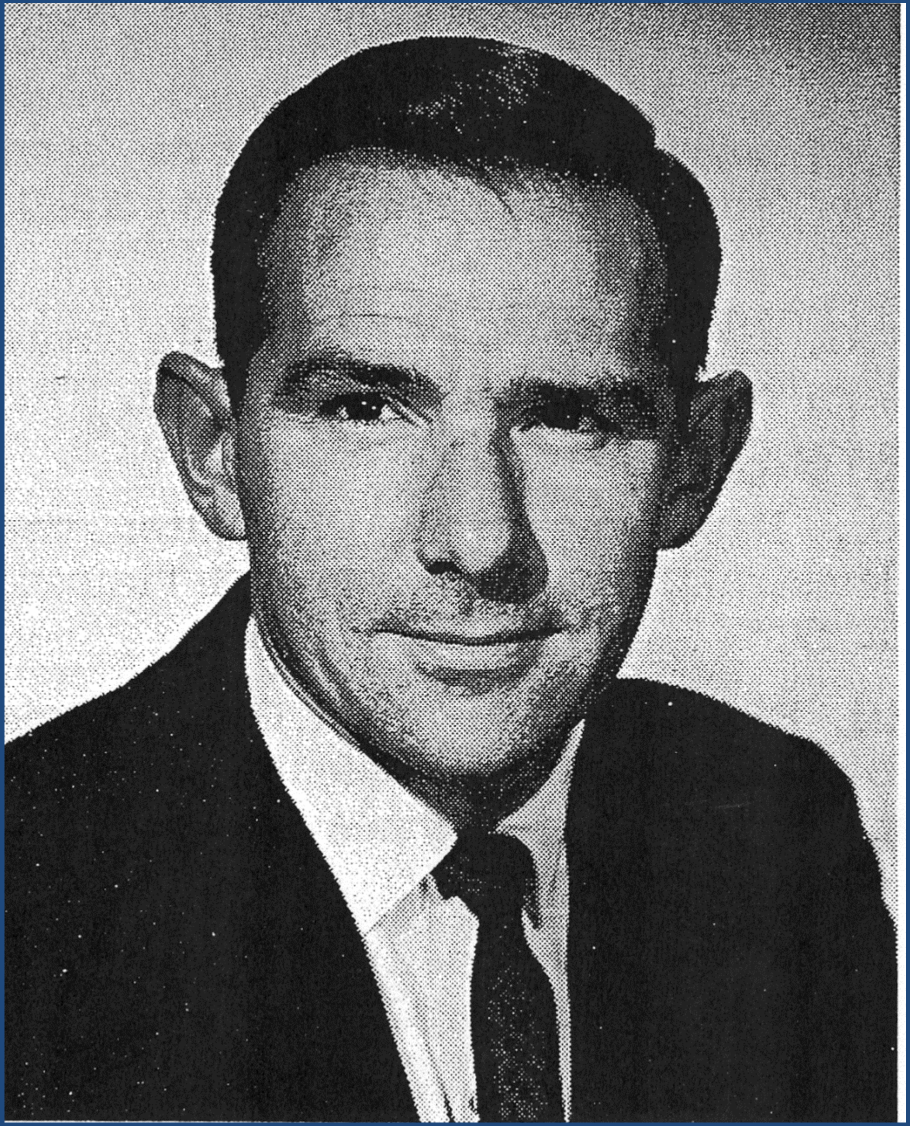
ESTABLISHED
BY
ASCE GEO-INSTITUTE
PAVEMENTS COMMITTEE



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 AASHO ROAD TEST - REPORT 5
PAVEMENT RESEARCH
SPECIAL REPORT 61E

- “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

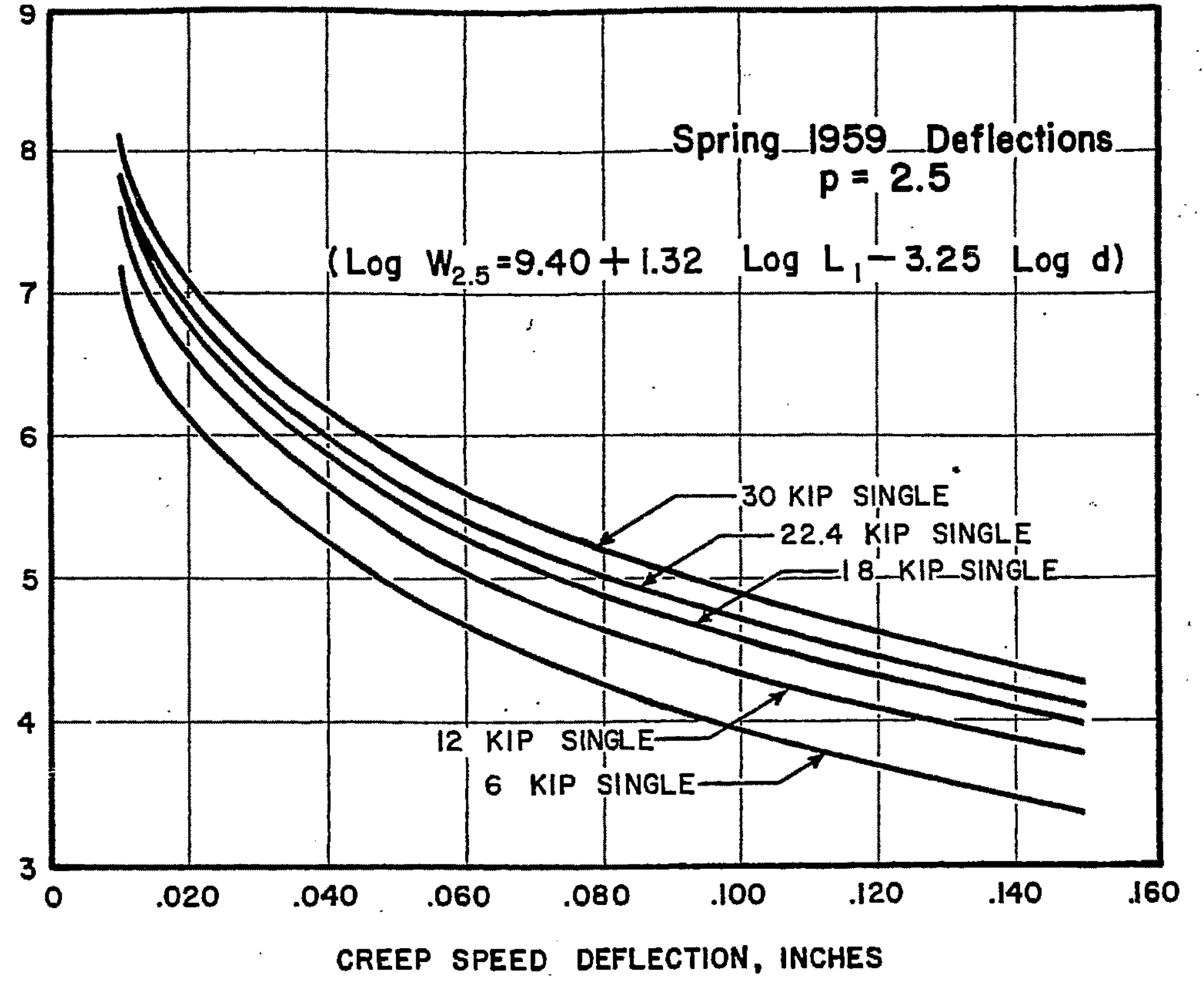
$$\text{LOG } W_{2.5} = 9.4 + 1.32 \text{ LOG } L - 3.25 \text{ LOG } d$$

$$\text{LOG } W_{1.5} = 10.18 + 1.36 \text{ LOG } L - 3.64 \text{ LOG } d$$

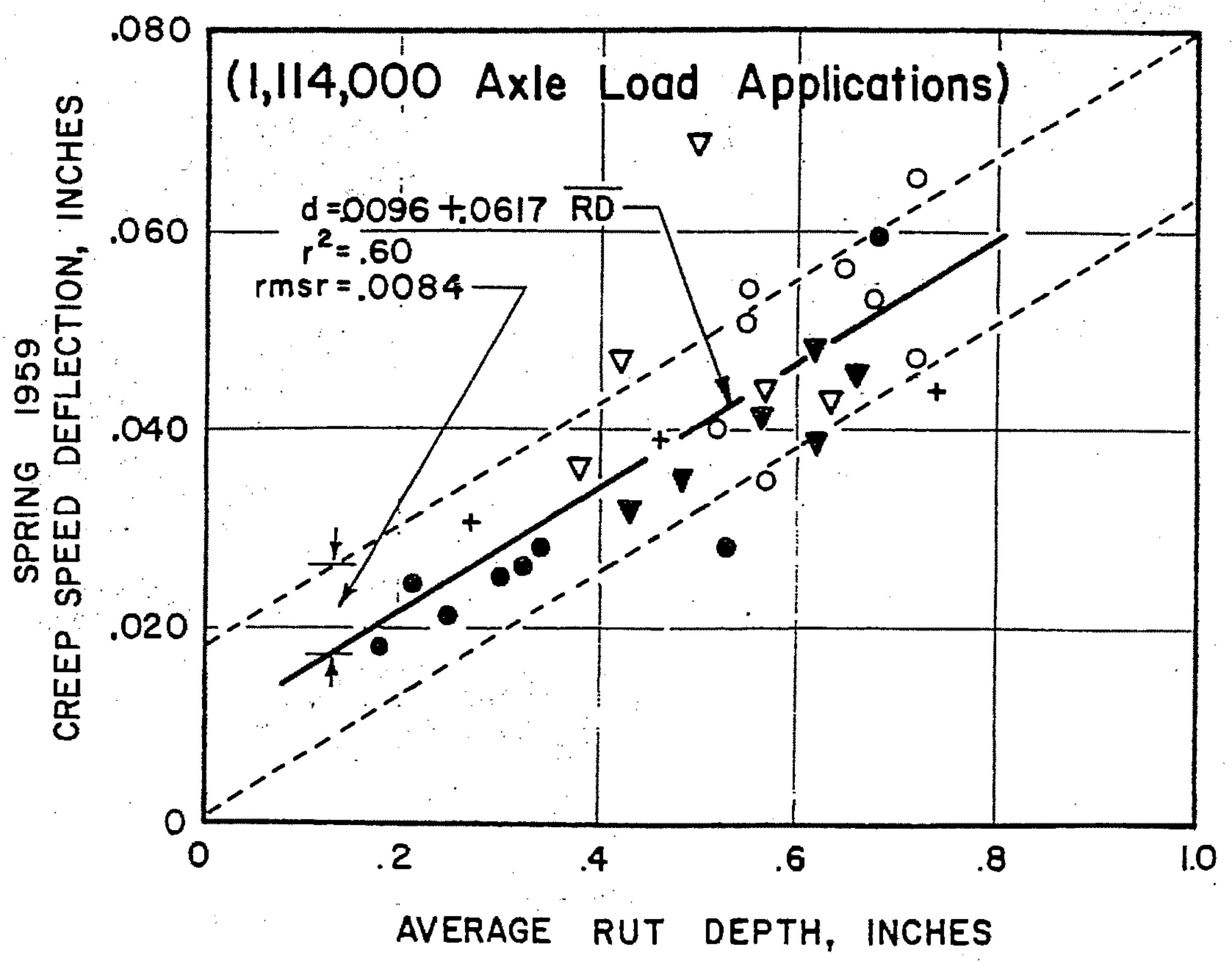
L – Axle Load (kips)

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

NCHRP 1-26 (1987) - NCHRP 1-37-A (1998)
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"

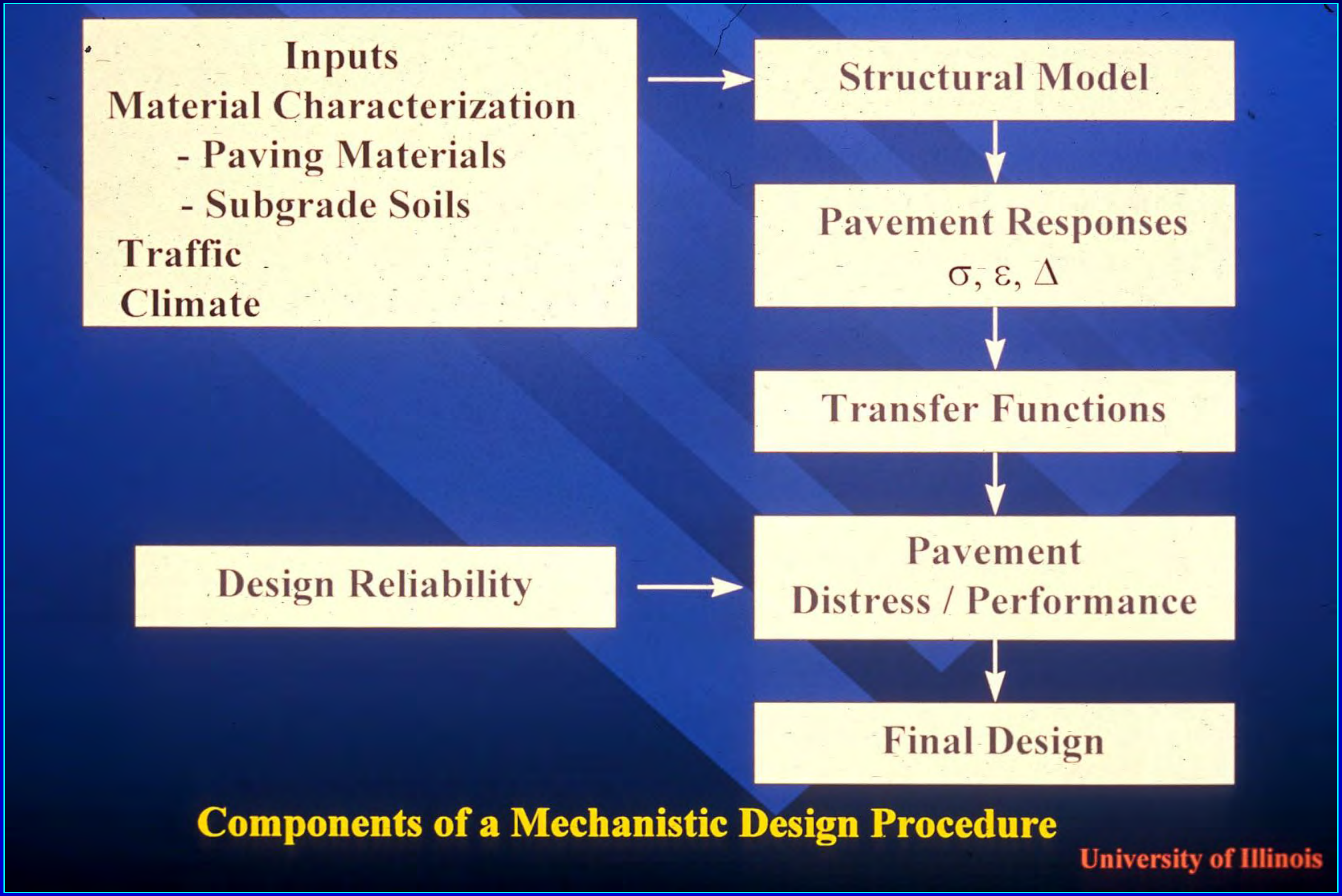
3rd International Conference on the Structural
Design of Asphalt Pavements
London – 1972

ASPHALT INSTITUTE
MS-11 – Third Edition - 1987
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



Inputs
Material Characterization
- Paving Materials
- Subgrade Soils
Traffic
Climate

Structural Model

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

Transfer Functions

Design Reliability

Pavement
Distress / Performance

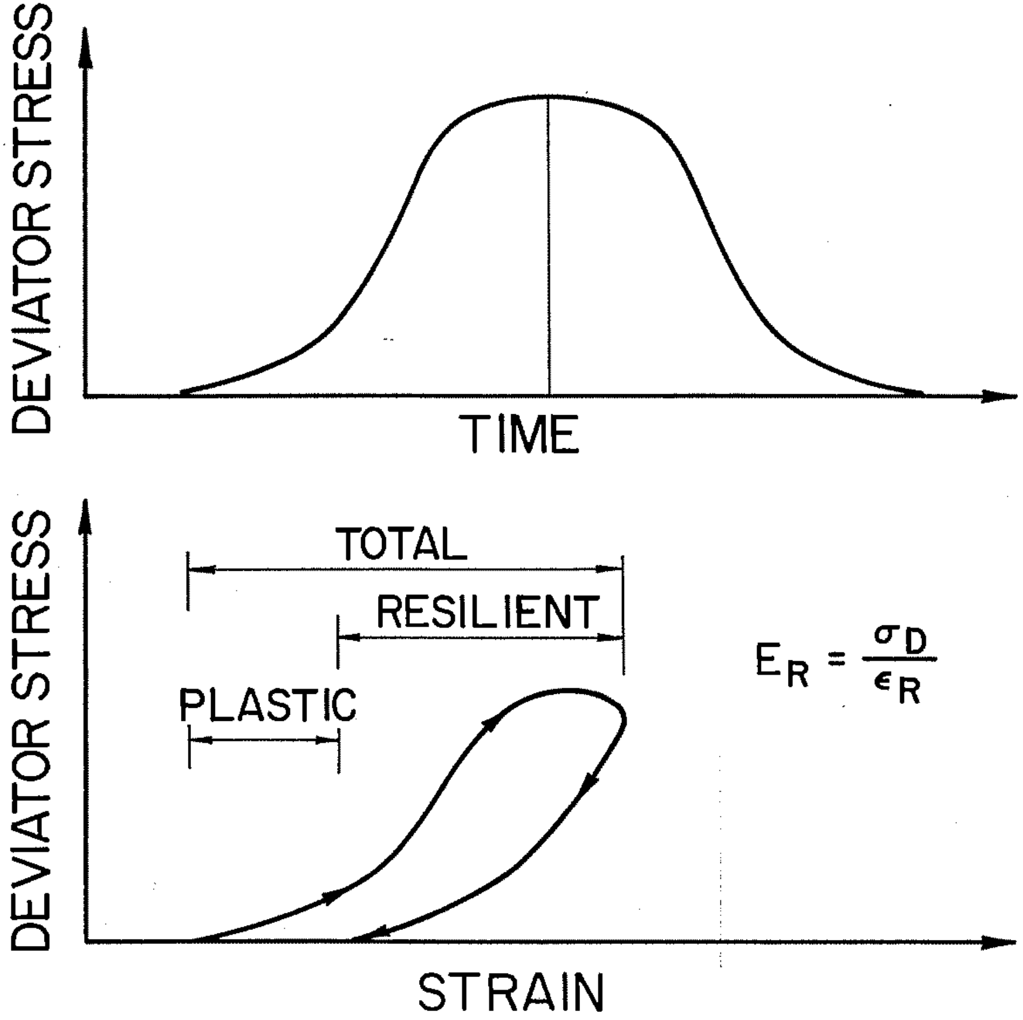
Final Design

Components of a Mechanistic Design Procedure

University of Illinois

RESILIENT MODULI INPUTS FOR PRACTICAL MECHANISTIC PAVEMENT DESIGN

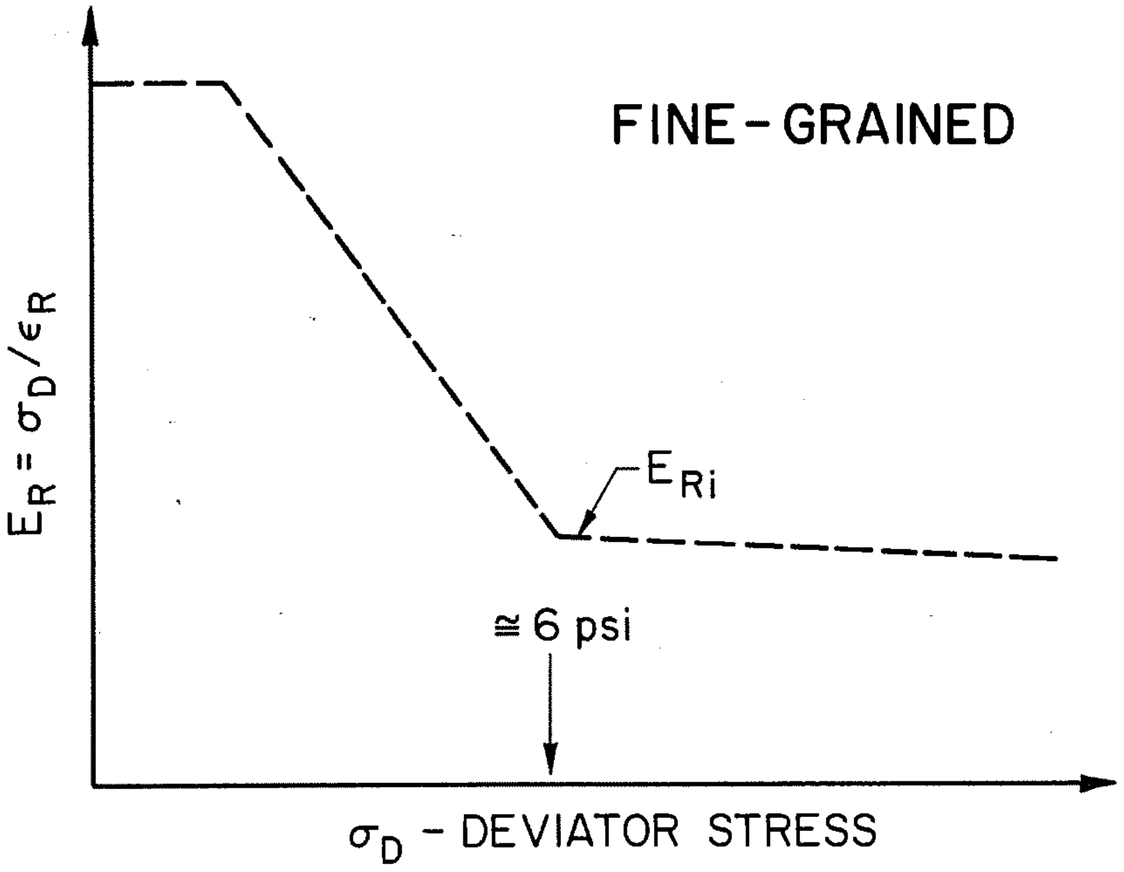
1999 TRB
Resilient Modulus and Mechanistic Pavement Design: Are We There yet??



$$E_R = \frac{\sigma_D}{\epsilon_R}$$

FINE-GRAINED SOILS

“STRESS SOFTENING”

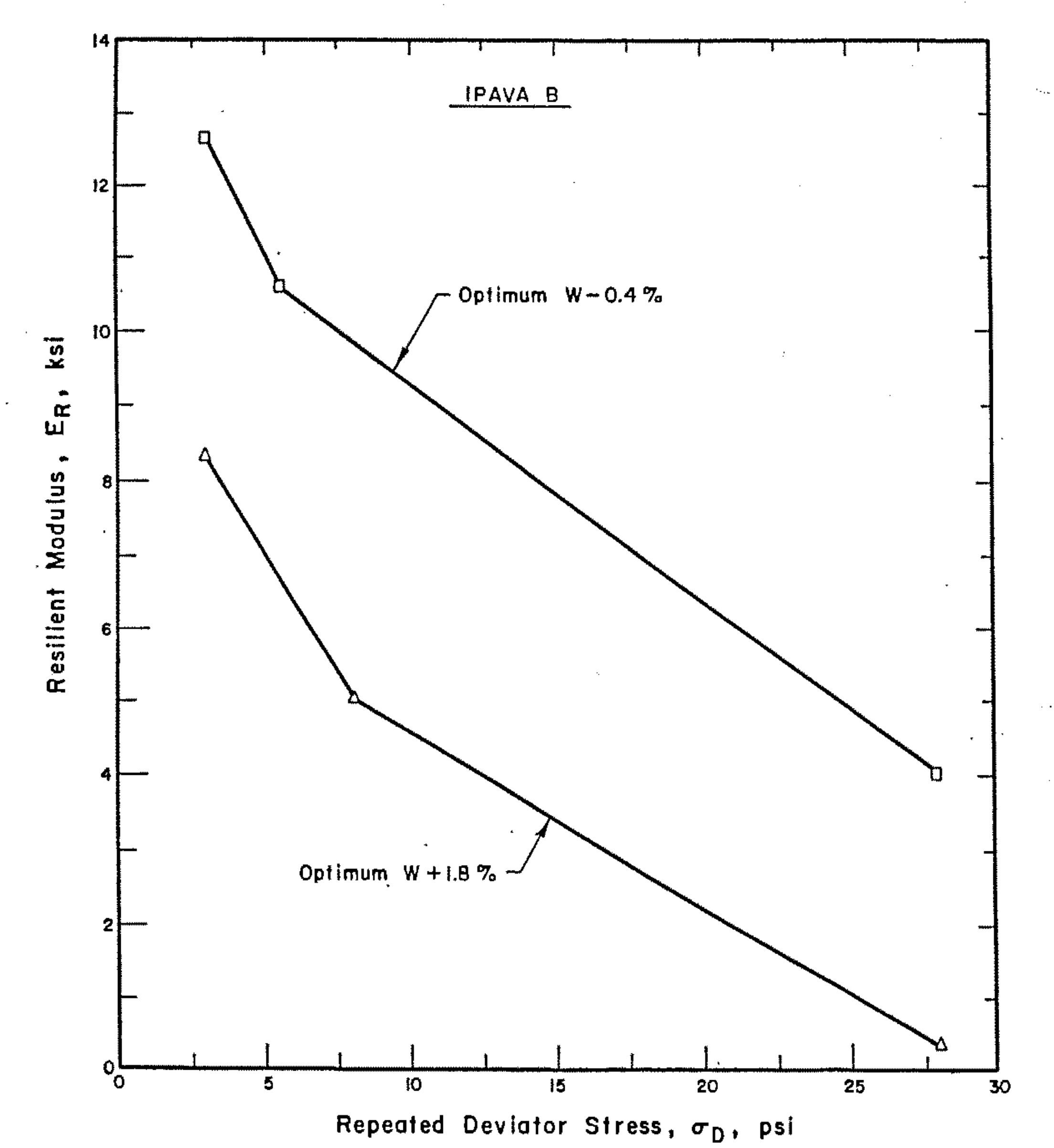


ARITHMETIC MODEL - UOFIL - IL DOT

THE ASPHALT INSTITUTE

MS-11 – Third Edition- 1987

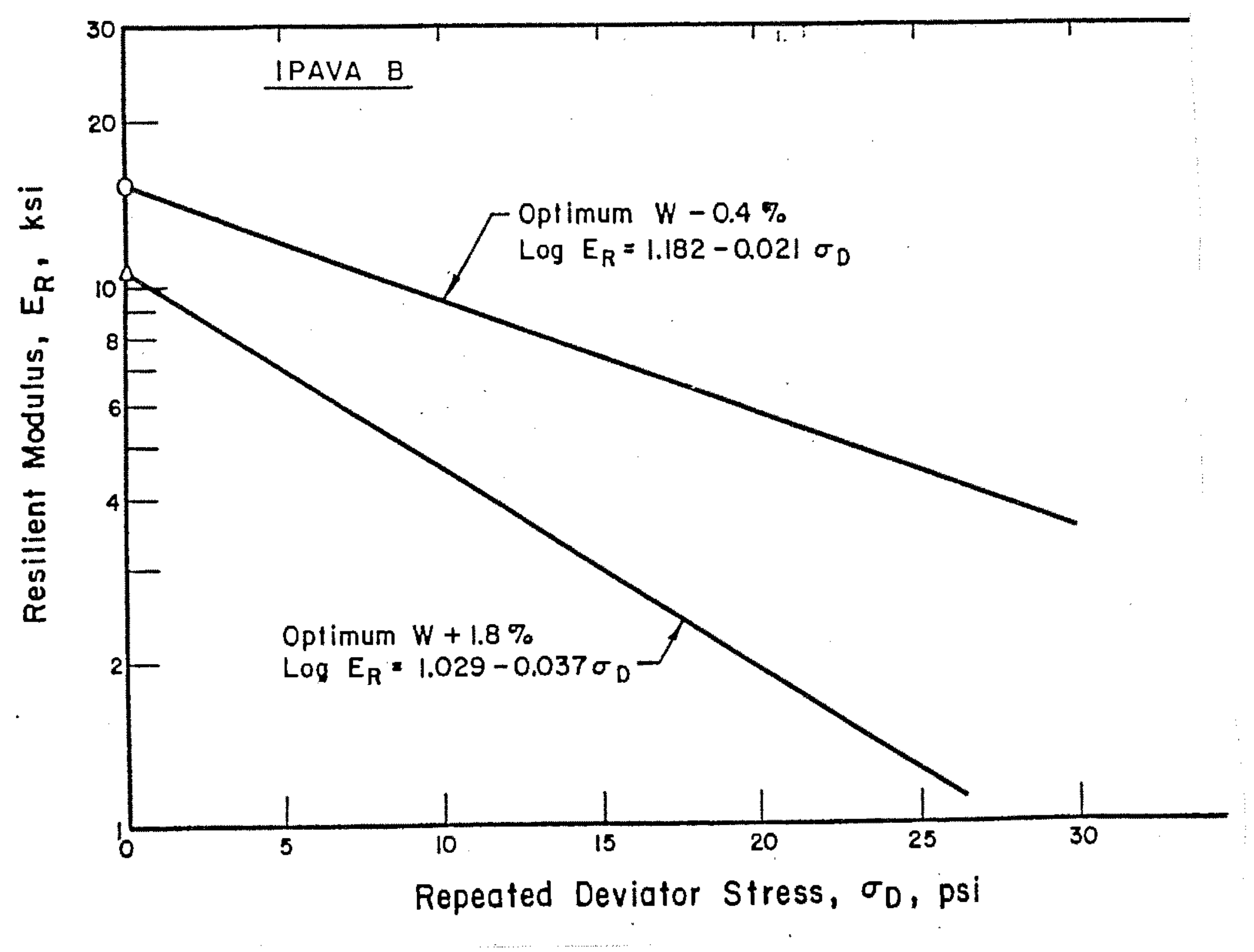
Modulus @ Deviator Stress of 6 psi



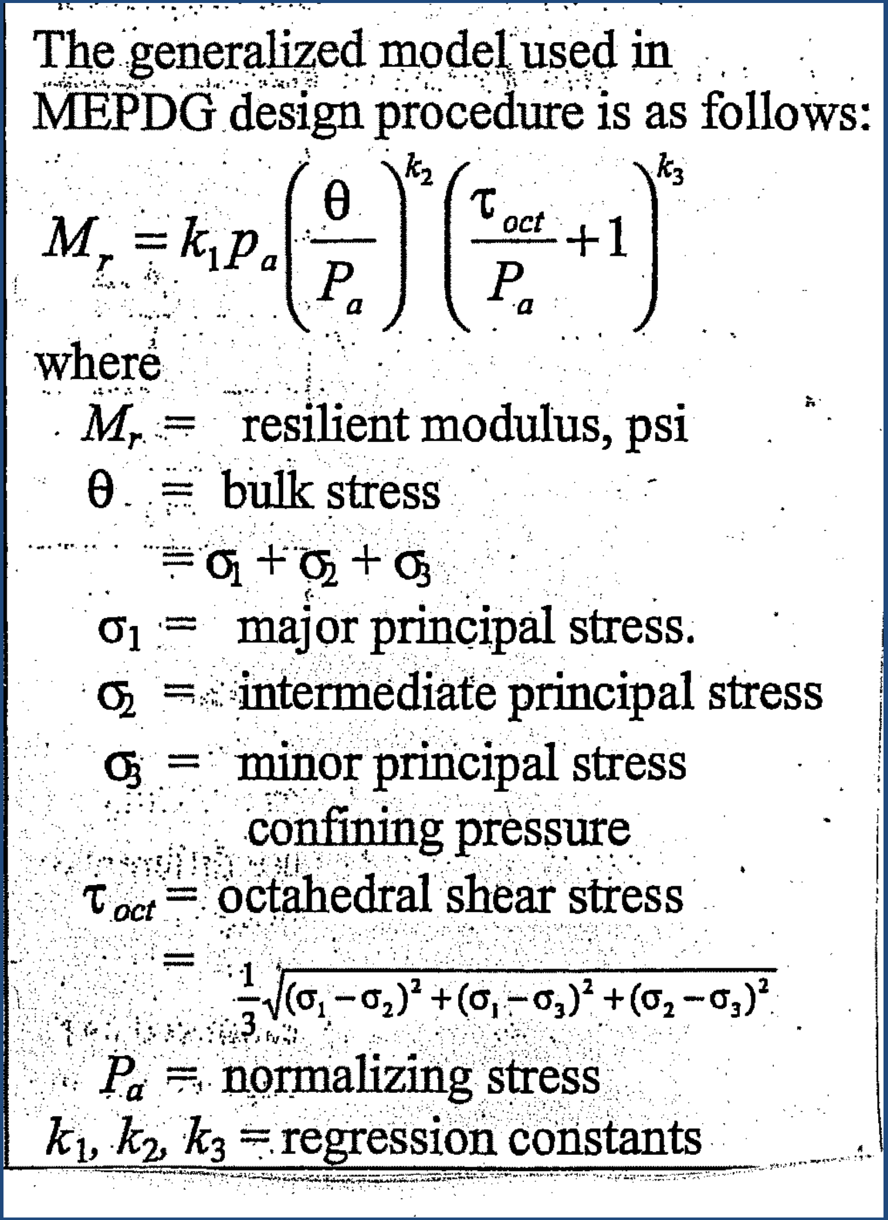
ARITHMETIC MODEL

SEMI-LOG MODEL

$$\mathrm{LOG}\ E_{Ri} = a - (b * \sigma_D)$$



SEMI-LOG MODEL



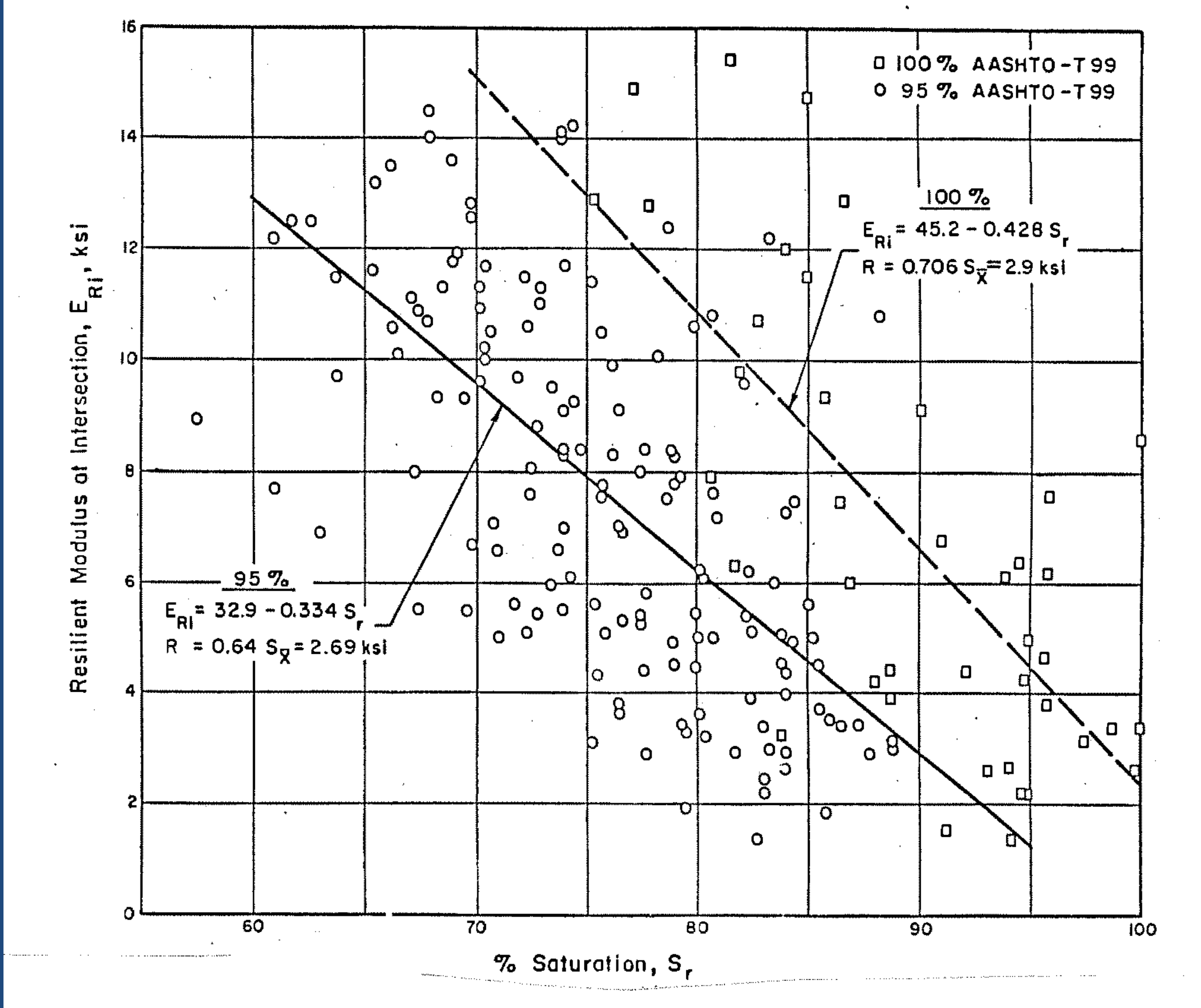
The generalized model used in MEPDG design procedure is as follows:

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

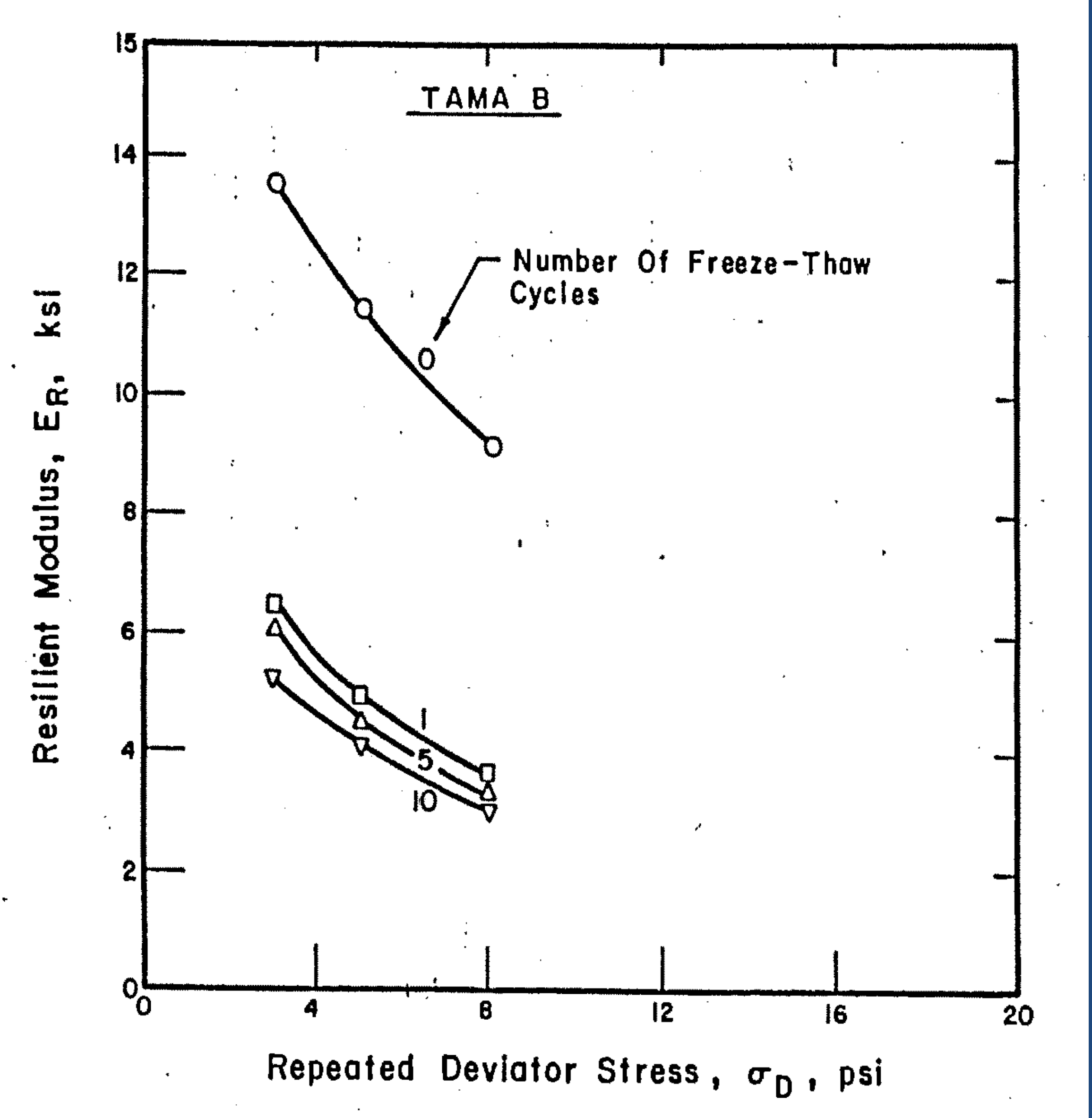
where

M_r = resilient modulus, psi
θ = bulk stress
= $\sigma_1 + \sigma_2 + \sigma_3$
σ_1 = major principal stress.
σ_2 = intermediate principal stress
σ_3 = minor principal stress
confining pressure
τ_{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

AASHTO M-E



DEGREE OF SATURATION EFFECTS



FREEZE-THAW EFFECTS

E_{Ri} INPUTS

* E_R TESTING

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

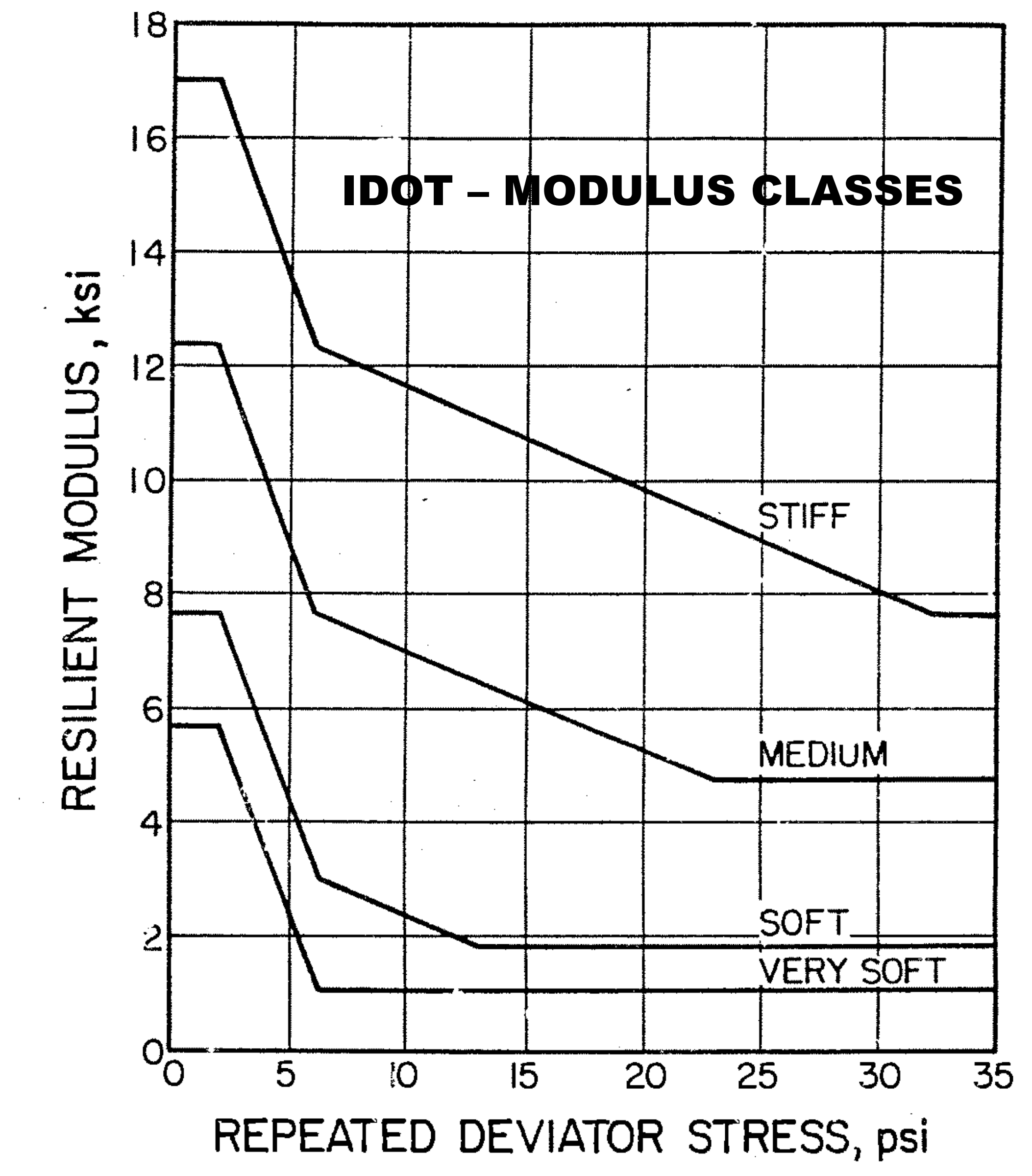
* ESTIMATES
- STRENGTH
(Q$_U$, CBR, DCP)
- σ/ϵ (PURDUE)
- % CLAY/PI
(PEDOLOGY - SOIL SERIES)

*TYPICAL VALUES

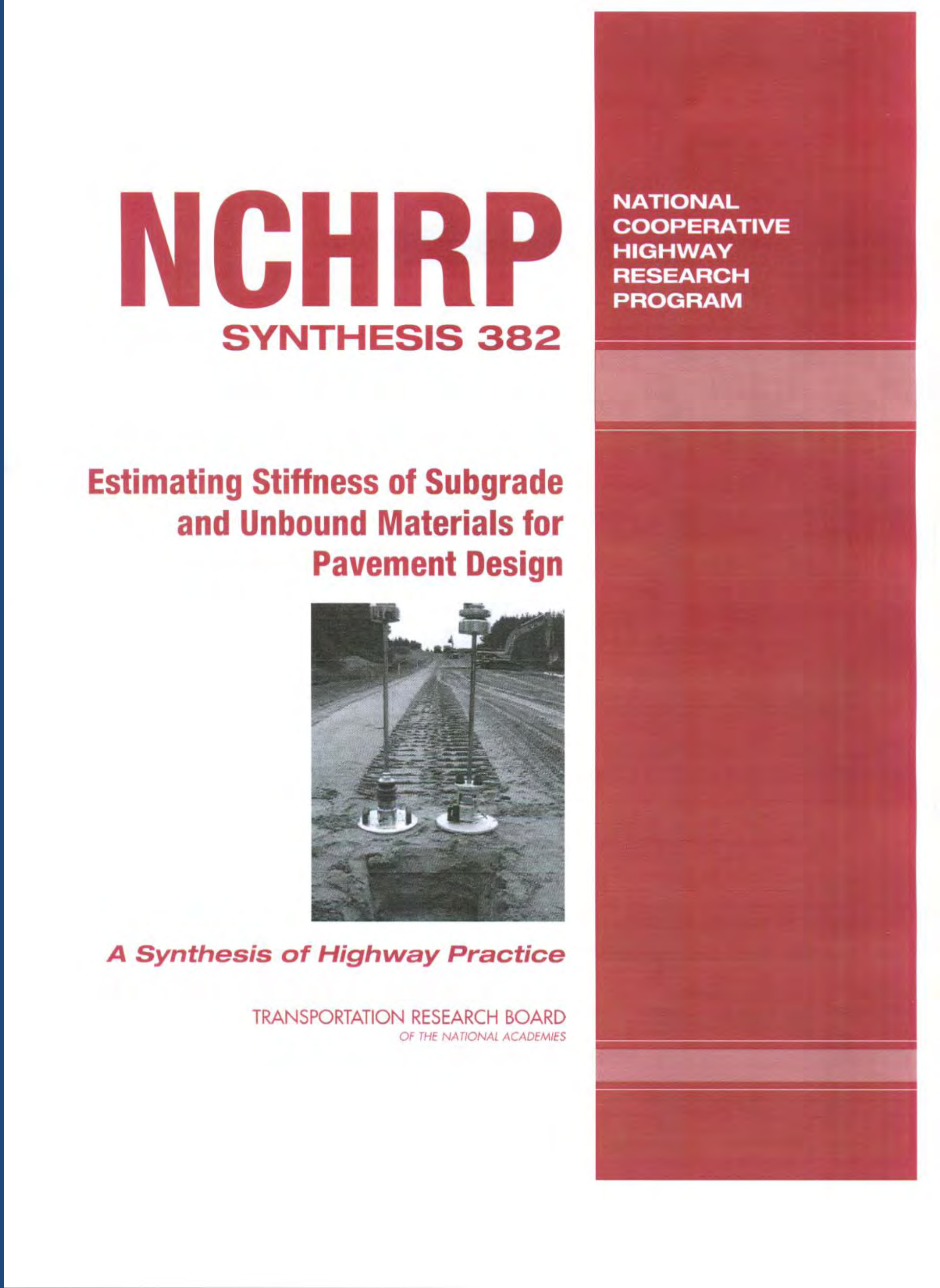
SUBGRADE VARIABILITY

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

* LAB TESTING VARIABILITY ???



ILLI-PAVE DEFAULT SUBGRADE S



NCHRP
SYNTHESIS 382

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design

A Synthesis of Highway Practice

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

A. J. PUPPALA
2008

SUBGRADE MODULUS = ???

GRANULAR MATERIALS

“STRESS-HARDENING”

86 AASHTO GUIDE

GRANULAR MATERIAL MODULI
ARE STRESS DEPENDENT

$$M_R = K_1 \theta^{K2}$$

86 AASHTO GUIDE
TYPICAL THETA MODEL PARAMETERS

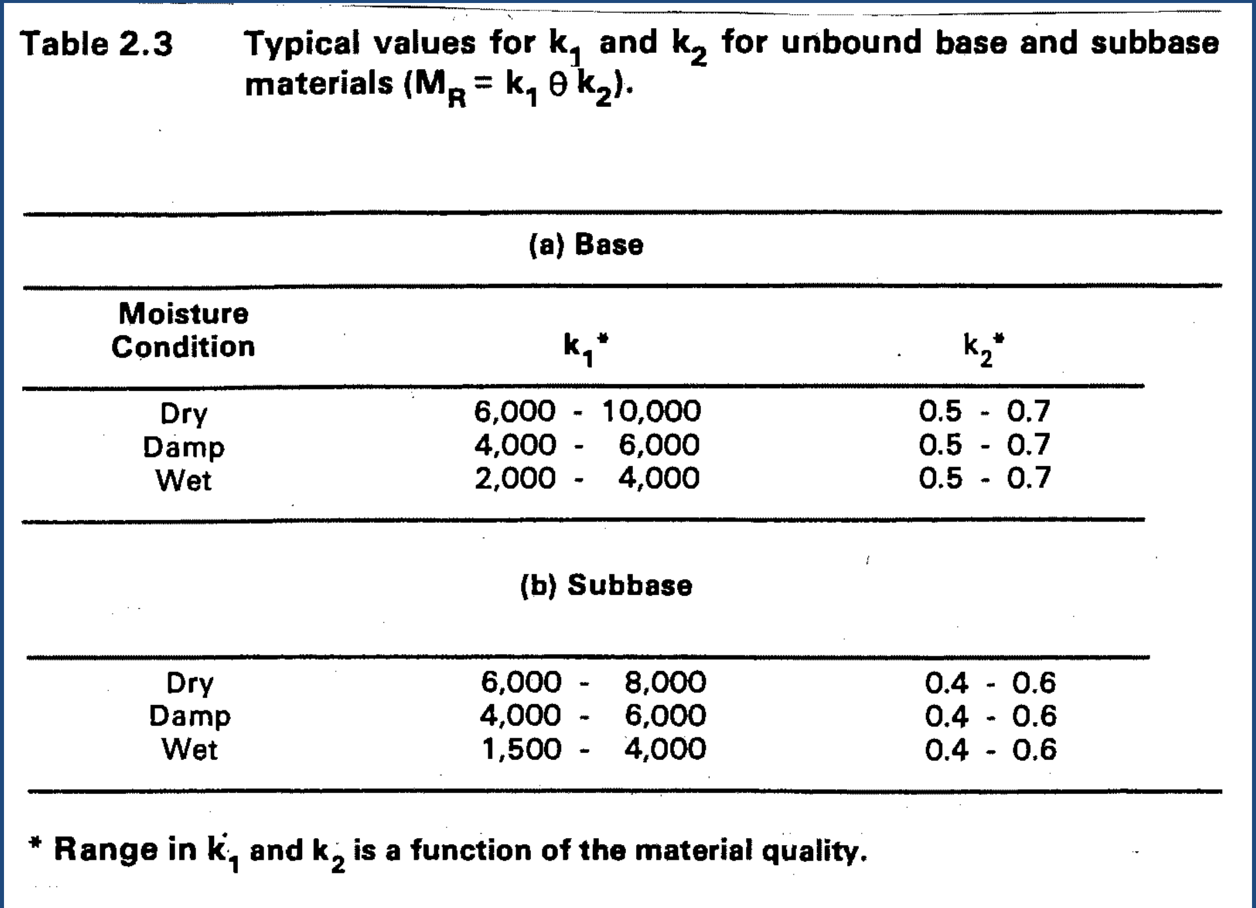


Table 2.3 Typical values for k_1 and k_2 for unbound base and subbase materials ($M_R = k_1 \theta^{k_2}$).

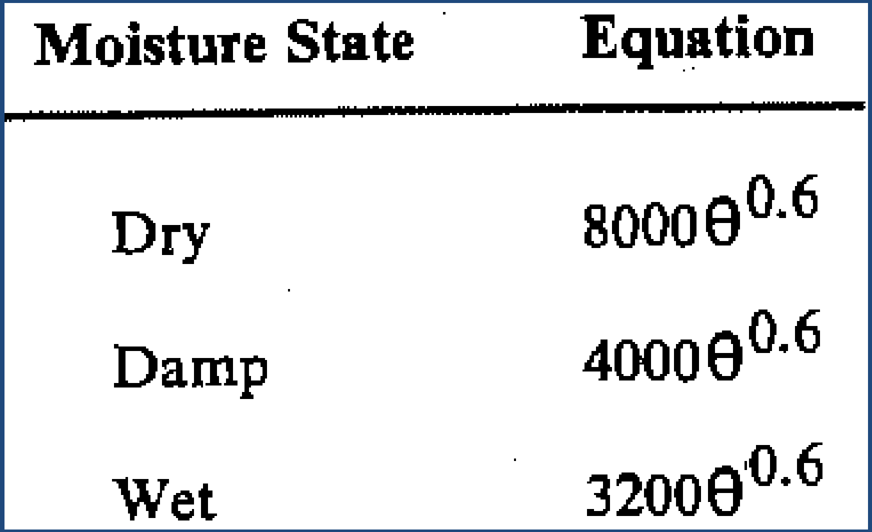
(a) Base

Moisture Condition	k_1*	k_2*
Dry	6,000 - 10,000	0.5 - 0.7
Damp	4,000 - 6,000	0.5 - 0.7
Wet	2,000 - 4,000	0.5 - 0.7

(b) Subbase

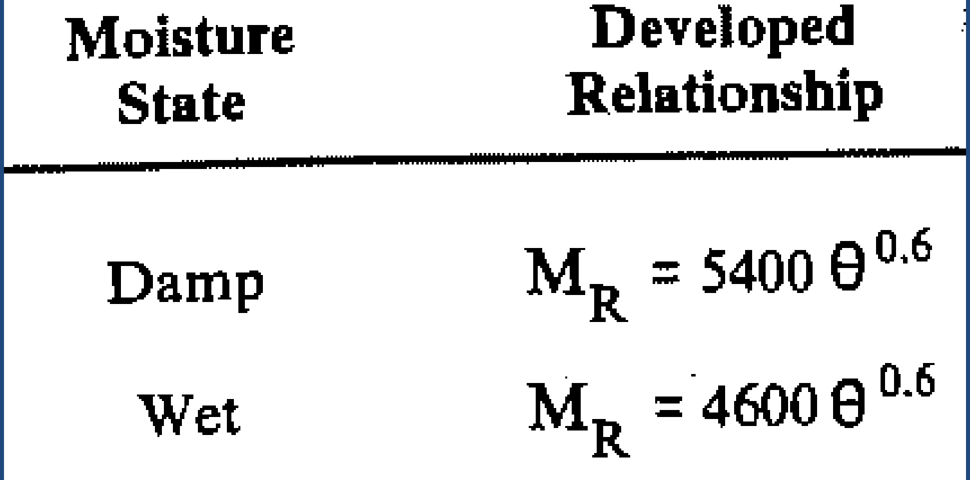
Moisture Condition	k_1*	k_2*
Dry	6,000 - 8,000	0.4 - 0.6
Damp	4,000 - 6,000	0.4 - 0.6
Wet	1,500 - 4,000	0.4 - 0.6

* Range in k_1 and k_2 is a function of the material quality.



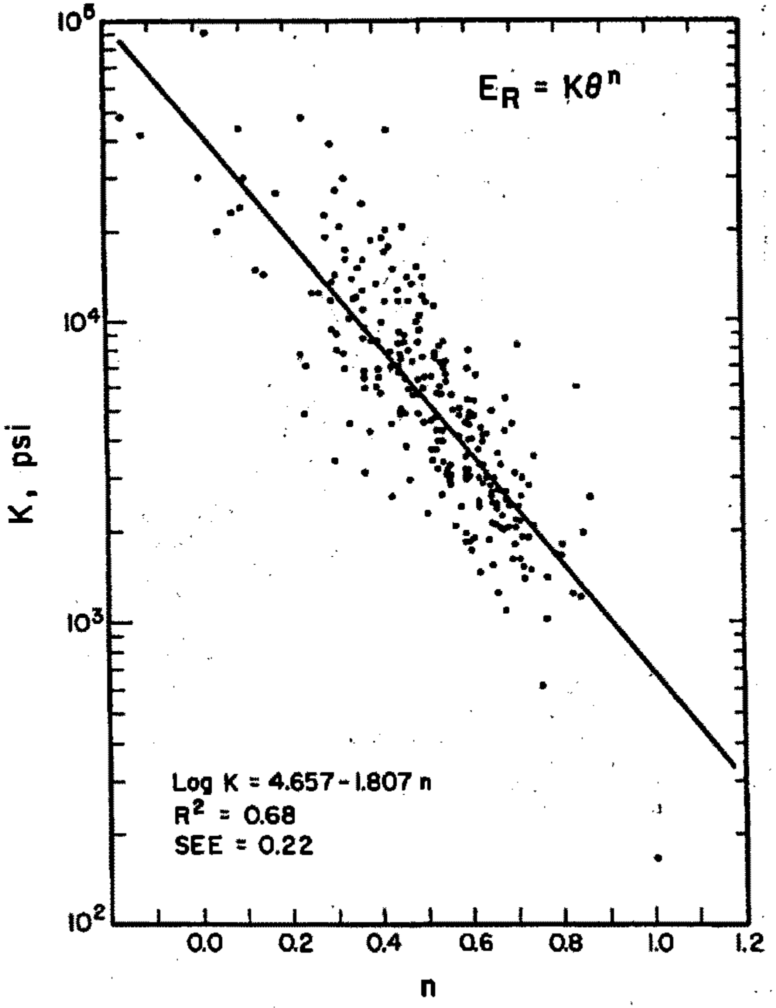
Moisture State	Equation
Dry	$8000\theta^{0.6}$
Damp	$4000\theta^{0.6}$
Wet	$3200\theta^{0.6}$

AASHTO BASE



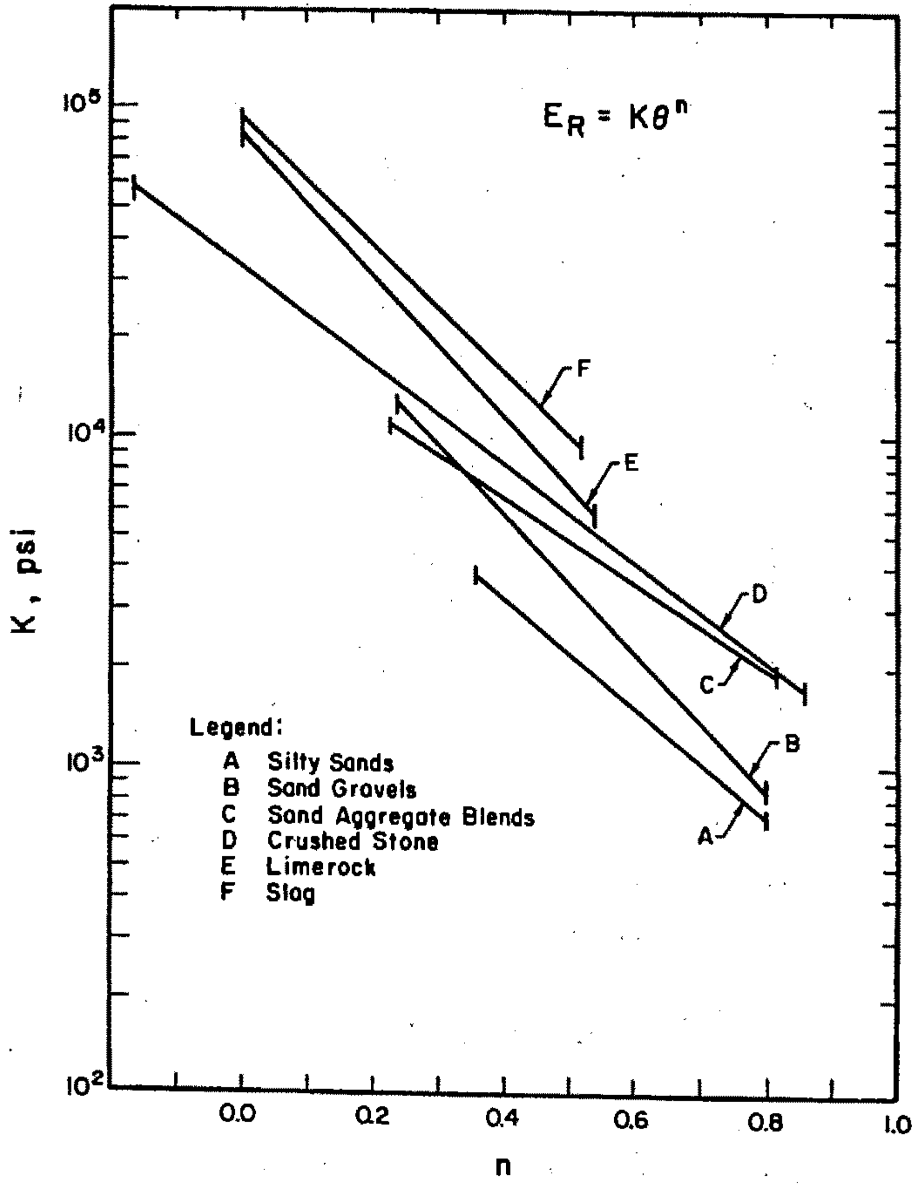
Moisture State	Developed Relationship
Damp	$M_R = 5400\,\theta^{0.6}$
Wet	$M_R = 4600\,\theta^{0.6}$

AASHTO SUBBASE



Rada, G. and M.W. Witczak. "Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Materials." Transportation Research Record 810, 1981.

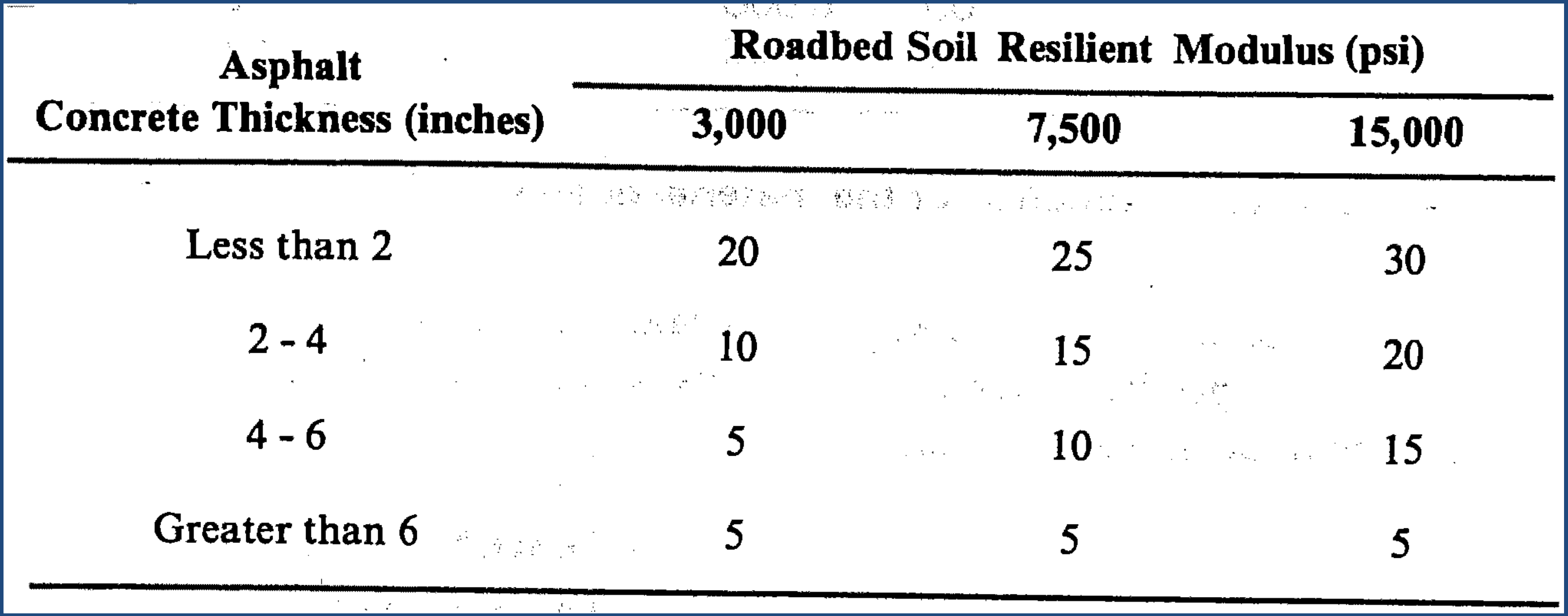
ALL DATA
(271 Data Points)
LOG K = 4.657 –1.807*n



Rada, G. and M.W. Witczak. "Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Materials." Transportation Research Record 810, 1981.

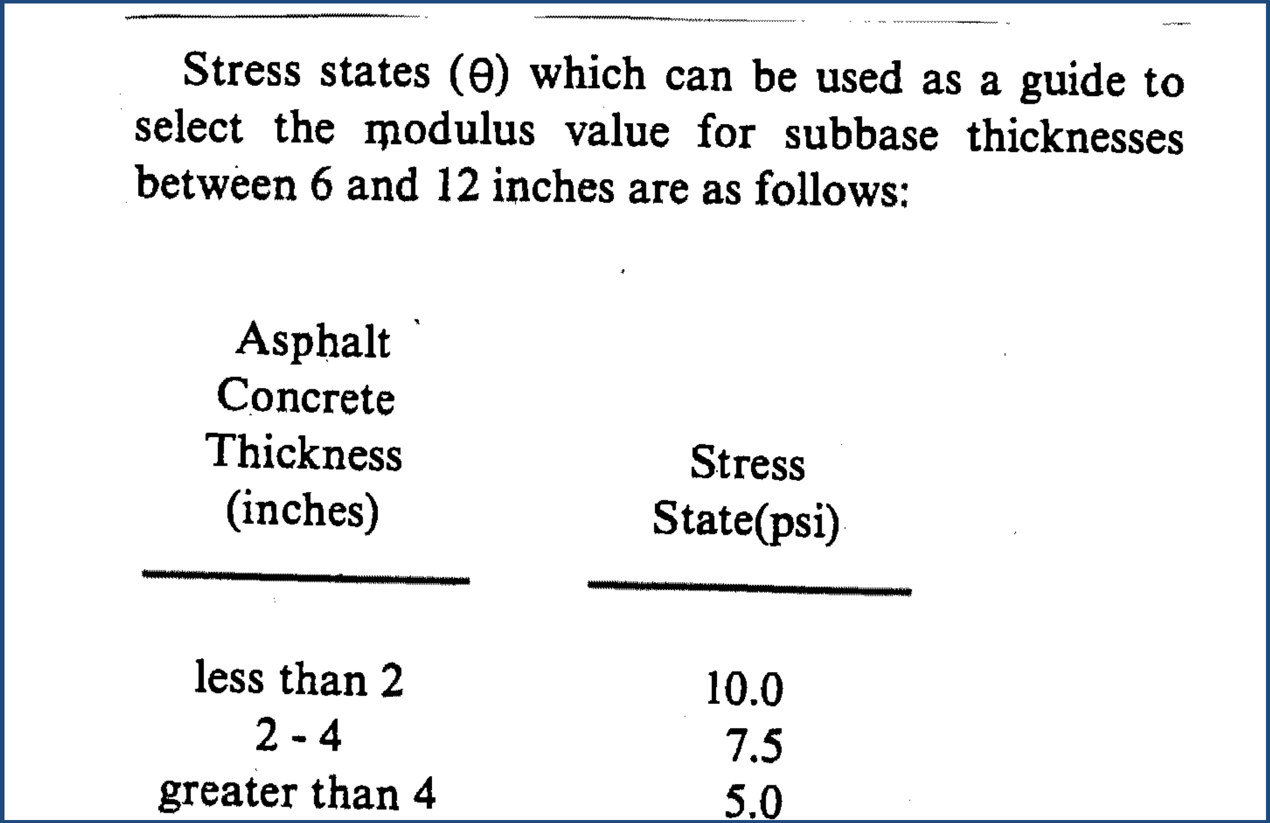
LOG K = A + b*n
(271 Data Points)
A's and b's for
the various materials

86 AASHTO GUIDE
RECOMMENDED THETA VALUES (psi)
BASE COURSE



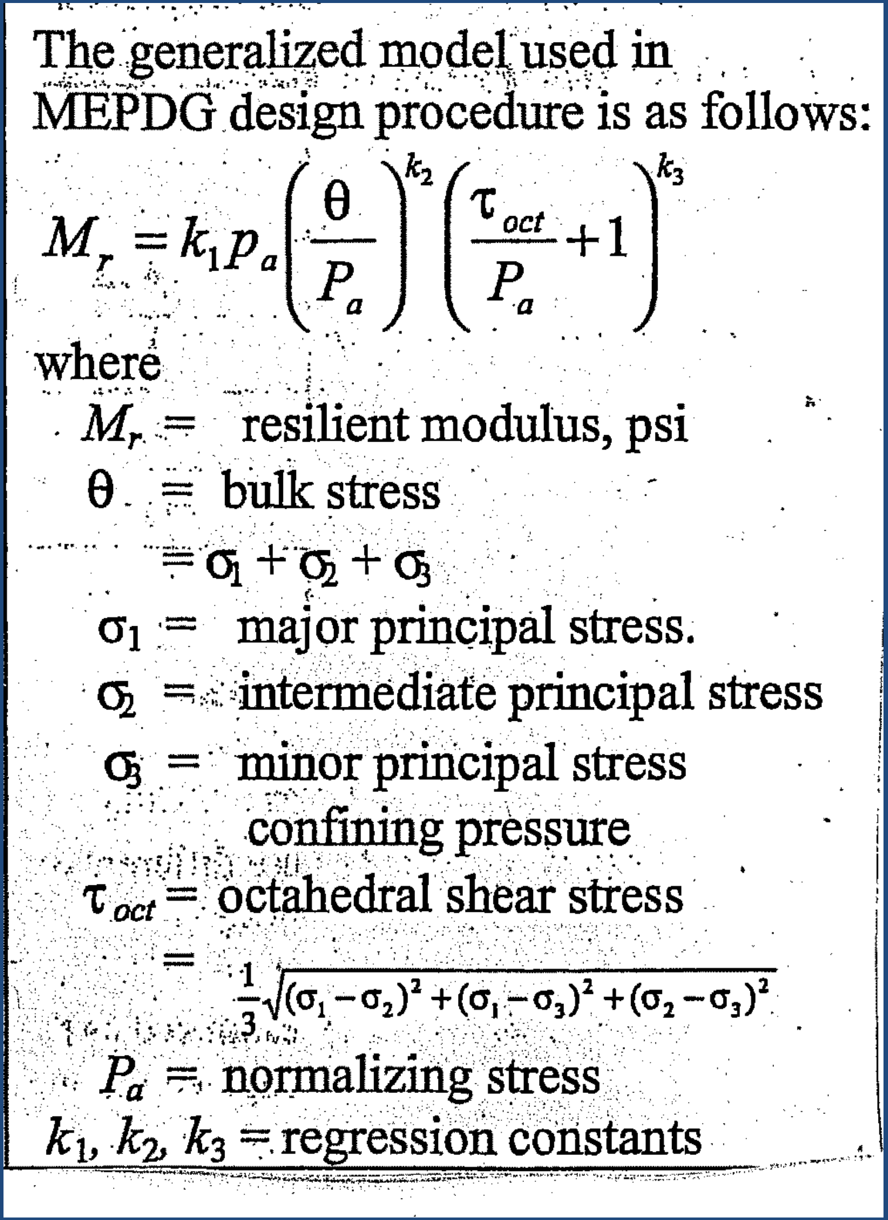
Asphalt Concrete Thickness (inches)	Roadbed Soil Resilient Modulus (psi)		
	3,000	7,500	15,000
Less than 2	20	25	30
2 - 4	10	15	20
4 - 6	5	10	15
Greater than 6	5	5	5

86 AASHTO GUIDE
RECOMMENDED THETA VALUES (psi)
SUBBASE



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:

Asphalt Concrete Thickness (inches)	Stress State(psi)
less than 2	10.0
2 - 4	7.5
greater than 4	5.0



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where

M_r = resilient modulus, psi

θ = bulk stress

= $\sigma_1 + \sigma_2 + \sigma_3$

σ_1 = major principal stress.

σ_2 = intermediate principal stress

σ_3 = minor principal stress confining pressure

τ_{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

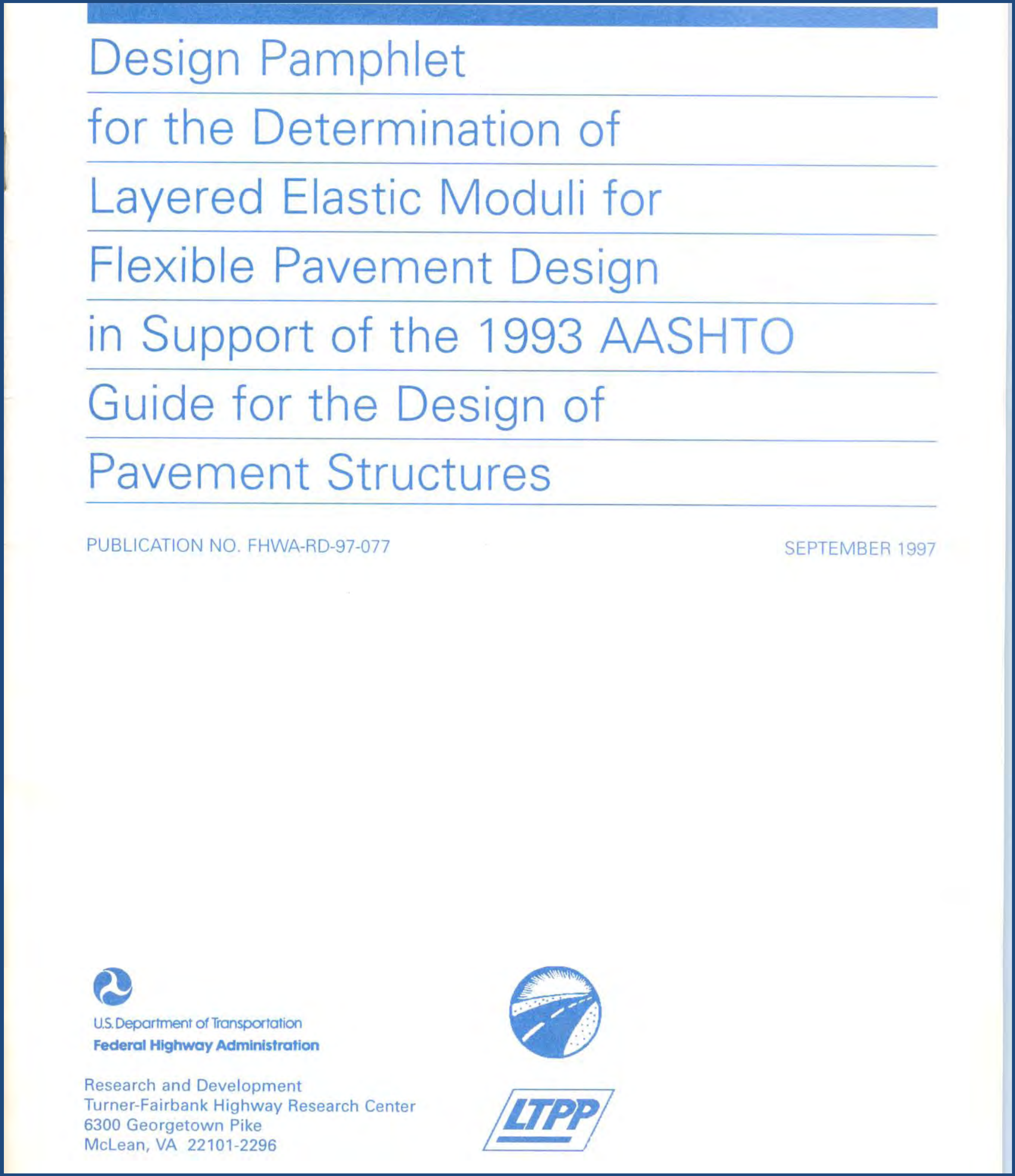
k_1, k_2, k_3 = regression constants

AASHTO M-E

Study of LTPP Laboratory Resilient ModulusTest Data and Response Characteristics: Final Report

FHWA-RD-02-051

A. Yau & H. Von Quintus
B. Fugro-BRE, Inc.



Design Pamphlet for the Determination of Layered Elastic Moduli for Flexible Pavement Design in Support of the 1993 AASHTO Guide for the Design of Pavement Structures

PUBLICATION NO. FHWA-RD-97-077

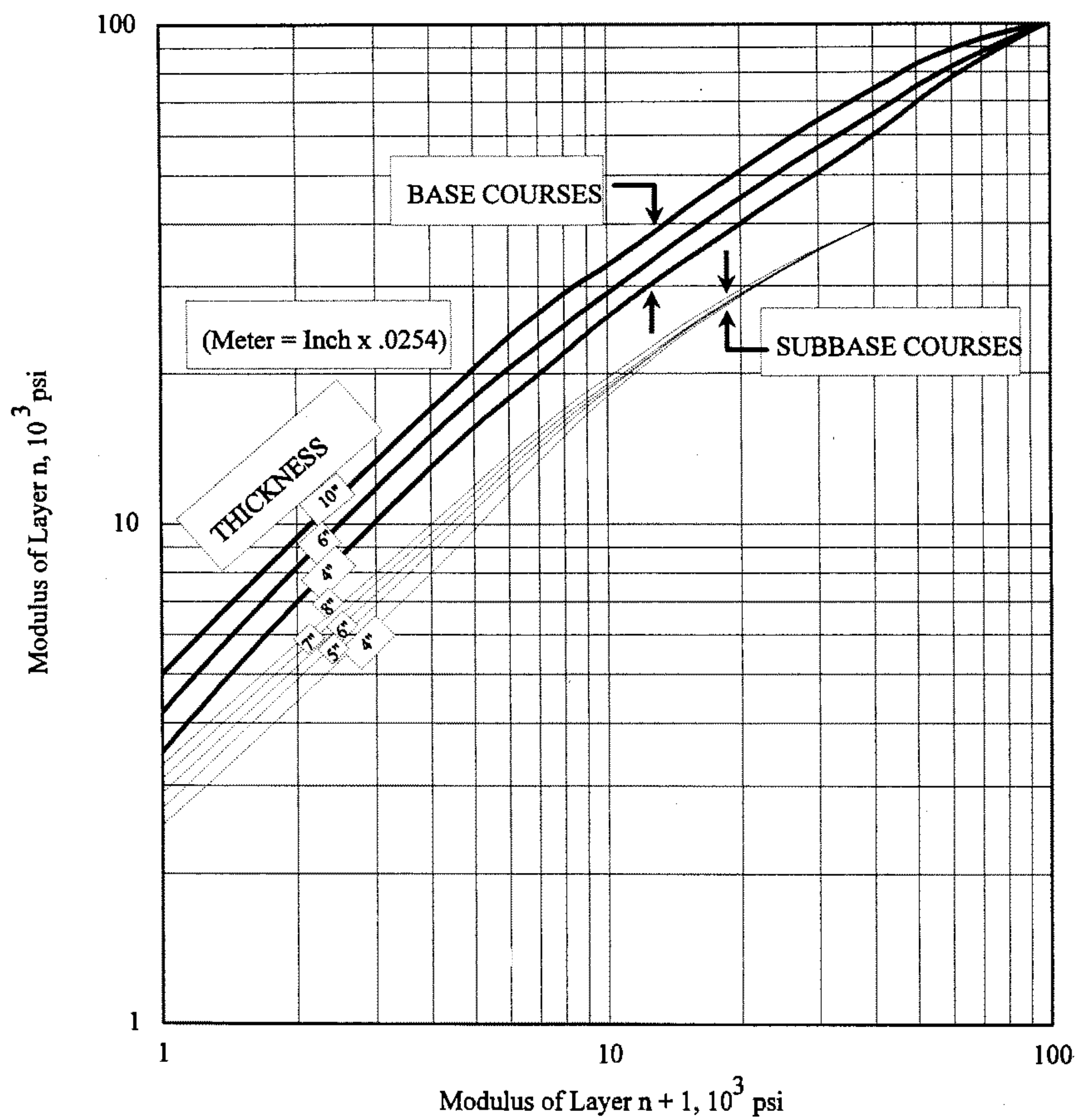
SEPTEMBER 1997

U.S. Department of Transportation
Federal Highway Administration

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

LTPP

H. Von Quintus
&
B. Killingsworth



n+1: Lower Layer

BARKER & BRABSTON
FAA-RD-74-199 (1975)

Addendum to the Shell Pavement Design Manual (1985)

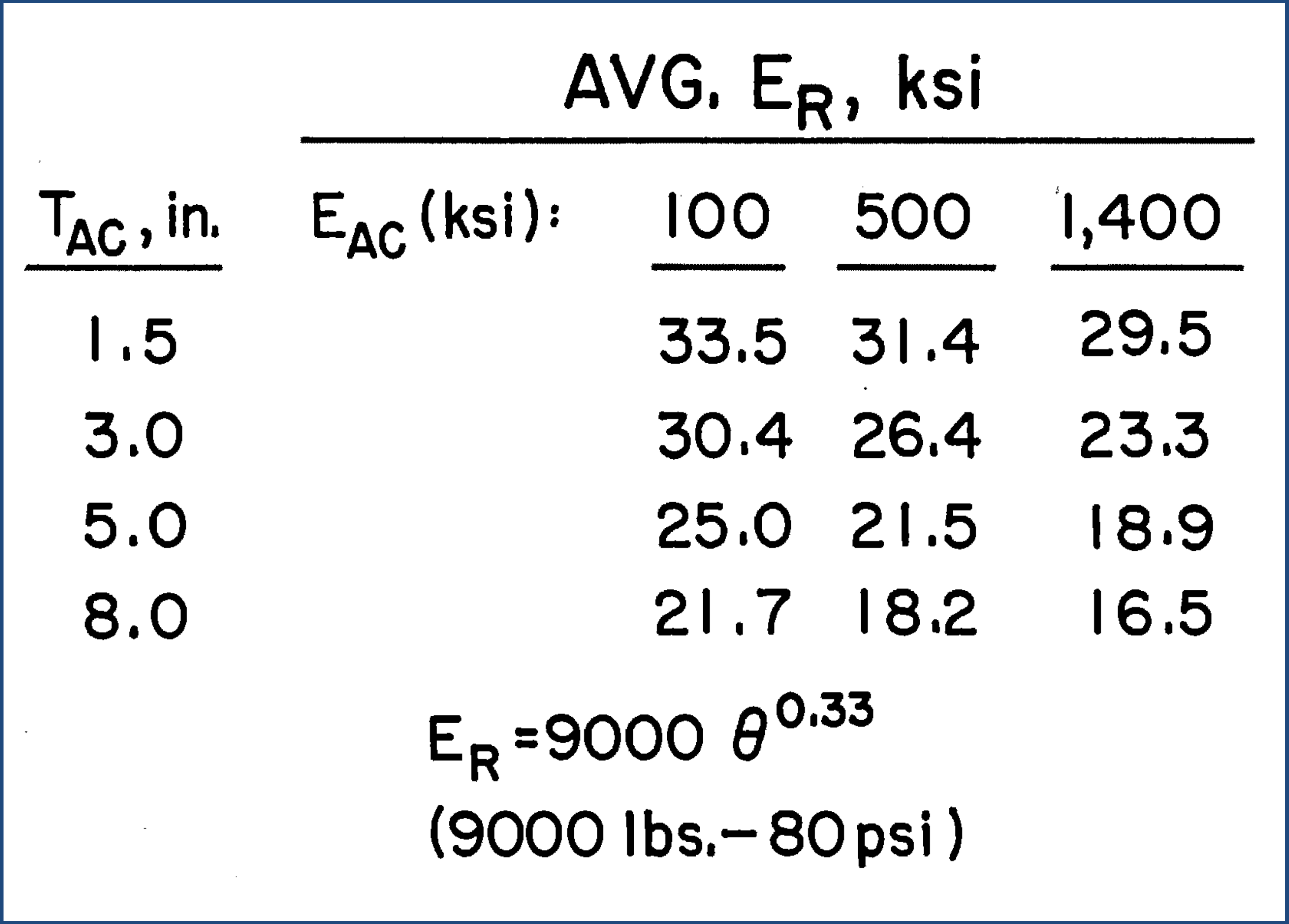
“Field measurements and theoretical considerations have indicated that the dynamic modulus of an unbound vase layer (E_2) must be related to the modulus of the subgrade (E_3).”

The following relationship is utilized:

$$E_2 = k * E_3 \qquad k = 0.2 * h_2^{0.45}$$

$$2<k<4$$

h_2 – thickness of the granular layer (mm)

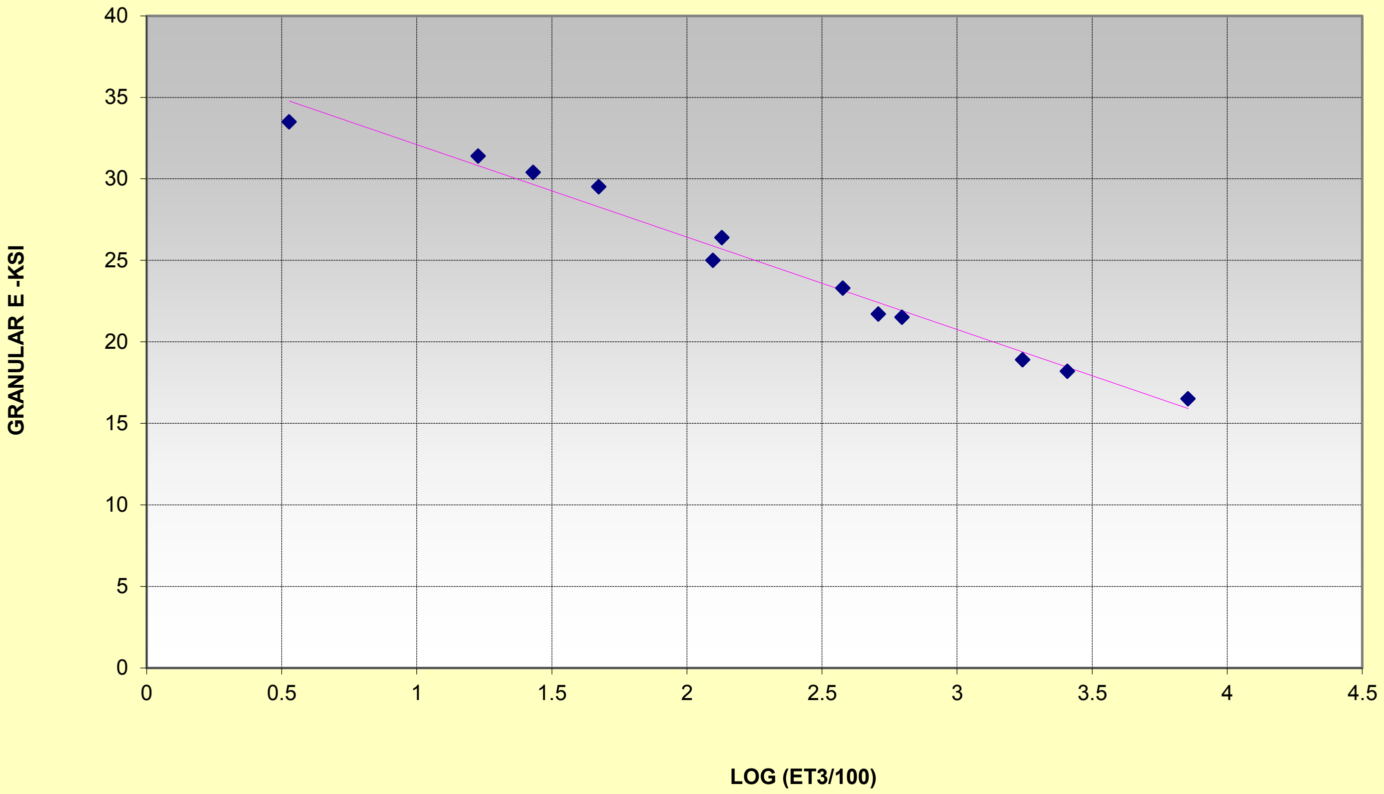


	AVG. E_R, ksi		
T_{AC}, in. — E_{AC} (ksi):	100	500	1,400
1.5	33.5	31.4	29.5
3.0	30.4	26.4	23.3
5.0	25.0	21.5	18.9
8.0	21.7	18.2	16.5

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

(9000 lbs.–80 psi)

THOMPSON - 5th INT. CONF.
DELFT - 1982



$$E\ (\text{ksi}) = 37.8 - (5.7 * [\text{LOG ET}^3 / 100])$$

R^2 = 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

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

* THETA IS FOR THE AVERAGE BASE MODULUS

OFFSET EFFECTS / SOFT SUBGRADE
(4-inch HMA)

OFFSET (inches)	MID-PT MODULUS (ksi)	THETA - θ (psi)
0-6	18.4	13.5
9	16.8	11.3
12	14.3	8.2
15	12.2	6
18	12	5.8
22	11.1	4.9
26	9.5	3.6
31	8.2	2.7
36	7.1	2
42	6.3	1.6

MATERIAL NC DOT DATA	R^2 THETA MODEL	R^2 UZAN MODEL
1	0.992	0.998
2	0.998	0.999
3	0.993	0.999
4	0.992	0.998
5	0.994	0.994
6	0.996	0.999
7	0.996	0.998
8	0.994	0.994
9	0.992	0.994
10	0.989	0.989

THETA: $M_R = K1*\theta^{K2}$

UZAN: $M_R = K1*\theta^{K2} * (\sigma_D)^{K3}$

BOTH MODELS CAPTURE THE STRESS HARDENING EFFECT

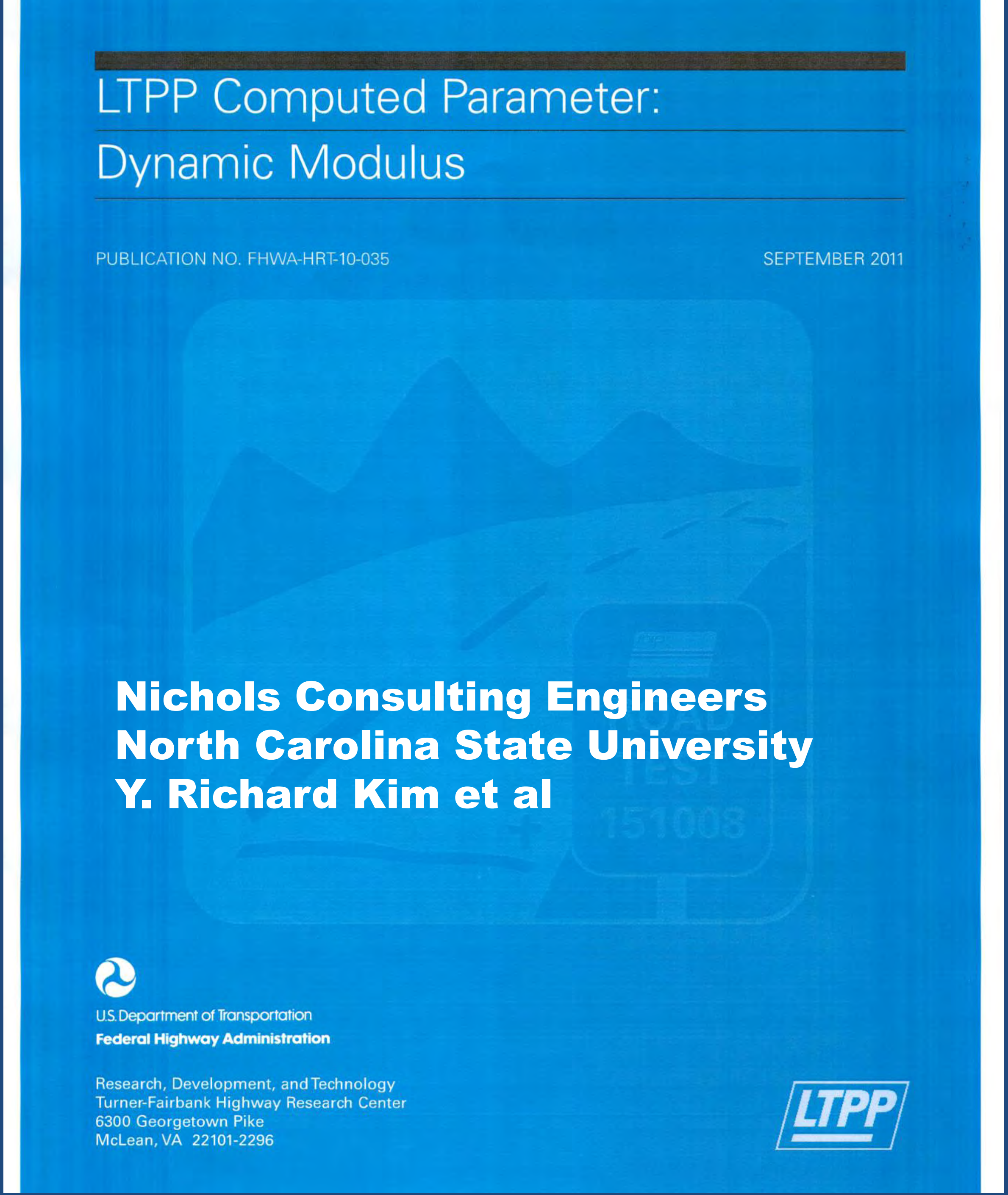
HMA MODULUS

* E_{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



LTPP Computed Parameter:
Dynamic Modulus

PUBLICATION NO. FHWA-HRT-10-035 SEPTEMBER 2011

Nichols Consulting Engineers
North Carolina State University
Y. Richard Kim et al

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

STRUCTURAL MODELS

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

* AXYSYMETRIC 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!!!

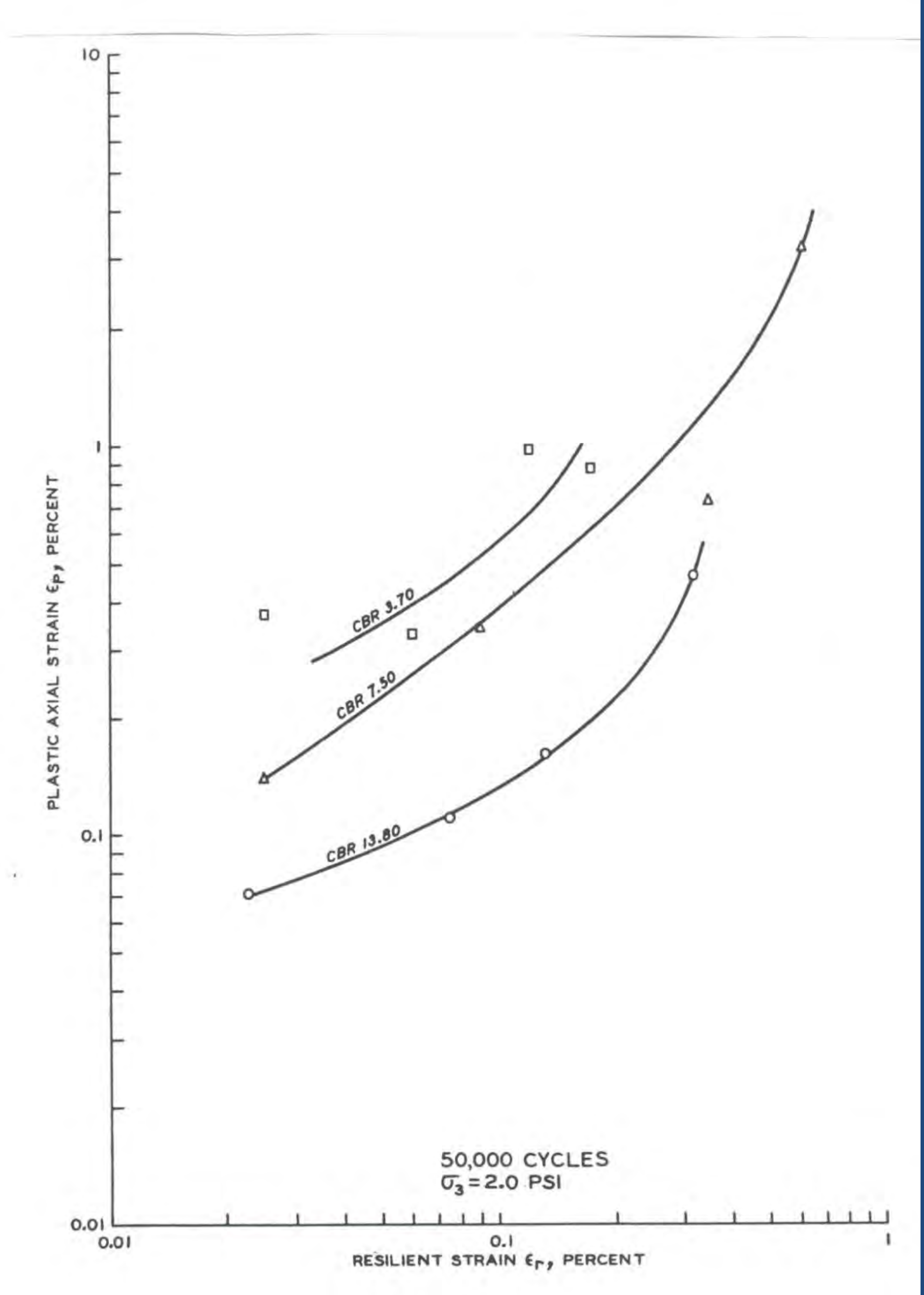
SUBGRADE RUTTING

SUBGRADE TRANSFER FUNCTIONS

- SUBGRADE VERTICAL STRAIN
- SUBGRADE STRESS RATIO (SSR)

  SSR= DEV STRESS / Q_U

TOWNSEND-CHISOLM
WES - NOV. 1976
Vicksburg Buckshot Clay



VERTICAL STRAIN CRITERIA

$$\varepsilon = L\,(1/N)^{m}$$

AGENCY	L	m	RD (INS)
AI	$1.05*10^{-2}$	0.223	0.5
SHELL			PSI / 2.5
50%	$2.8*10^{-2}$	0.25	
85%	$2.1*10^{-2}$	0.25	CROW
95%	$1.8*10^{-2}$	0.25	
TRL/1132 (85%)	$1.5*10^{-2}$	0.253	0.4

MIKE NUNN / TRL 615 (2004)

“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.”

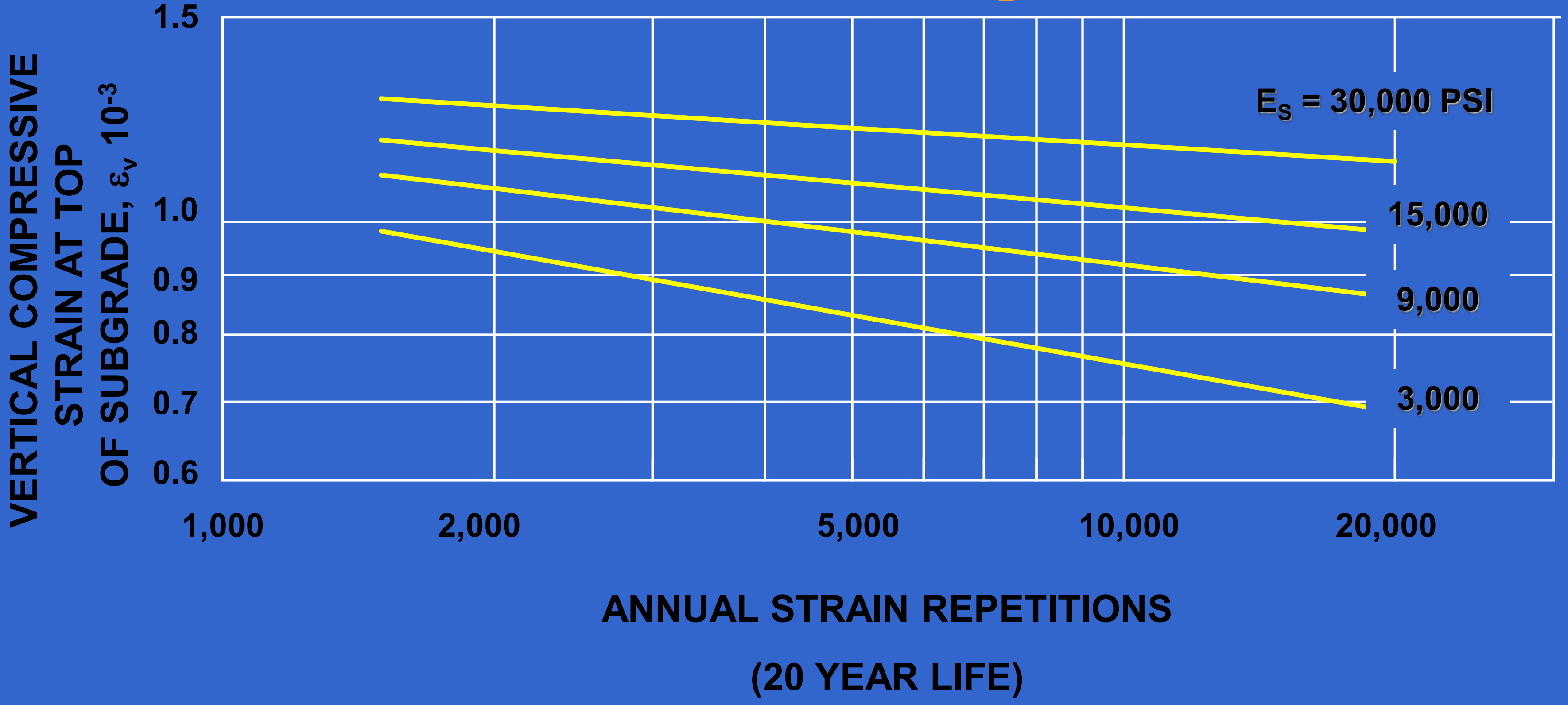
NOTE: The procedure utilizes “foundation classes” and “equivalent elastic half spaces” to characterize the composite foundation support.

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)

CURRENT FAA SUBGRADE STRAIN CRITERIA

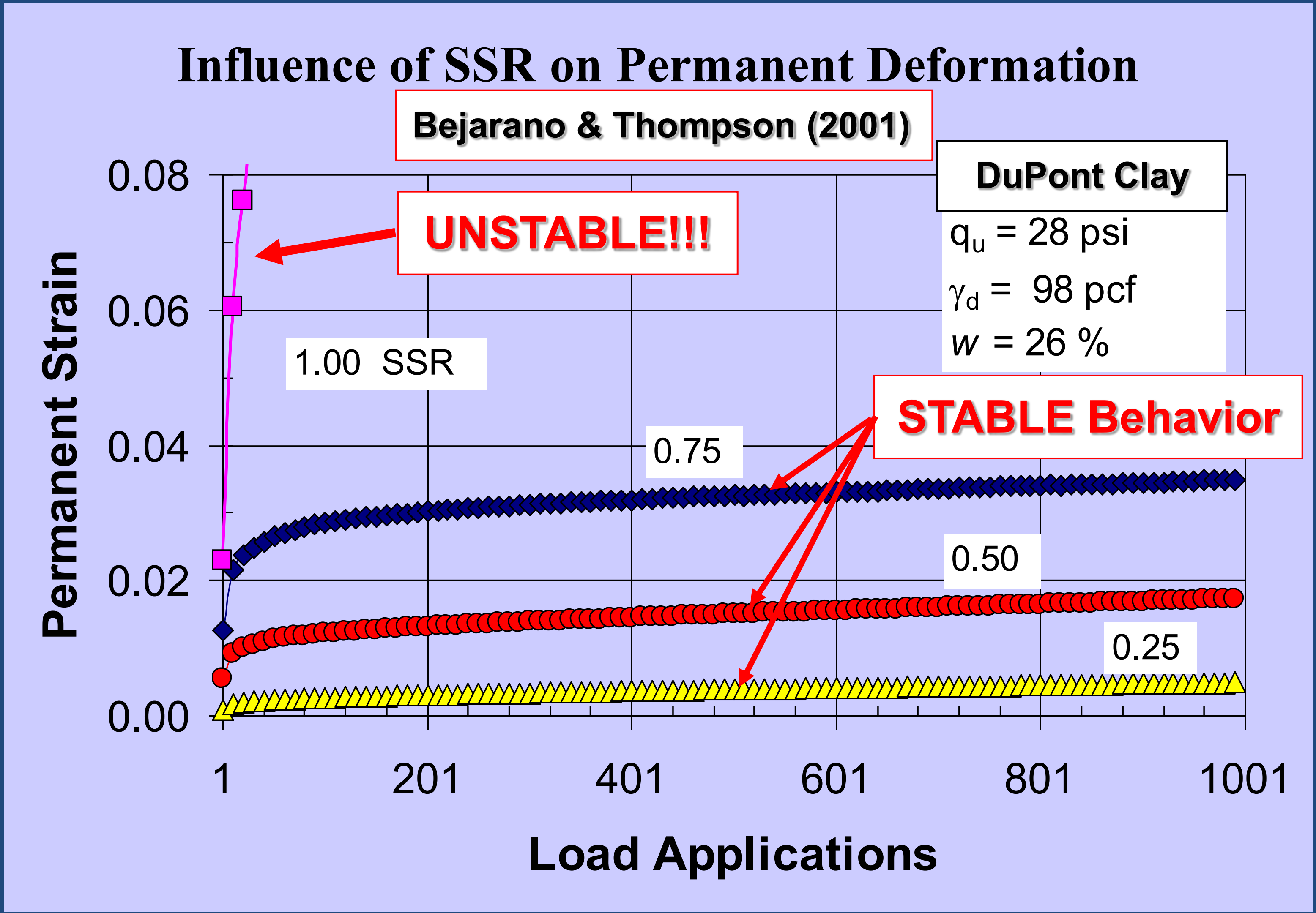
$C < 12{,}100$: $C = (0.004/\varepsilon_V)^{8.1}$

$C > 12{,}100$: $C = (0.002428/\varepsilon_V)^{14.21}$

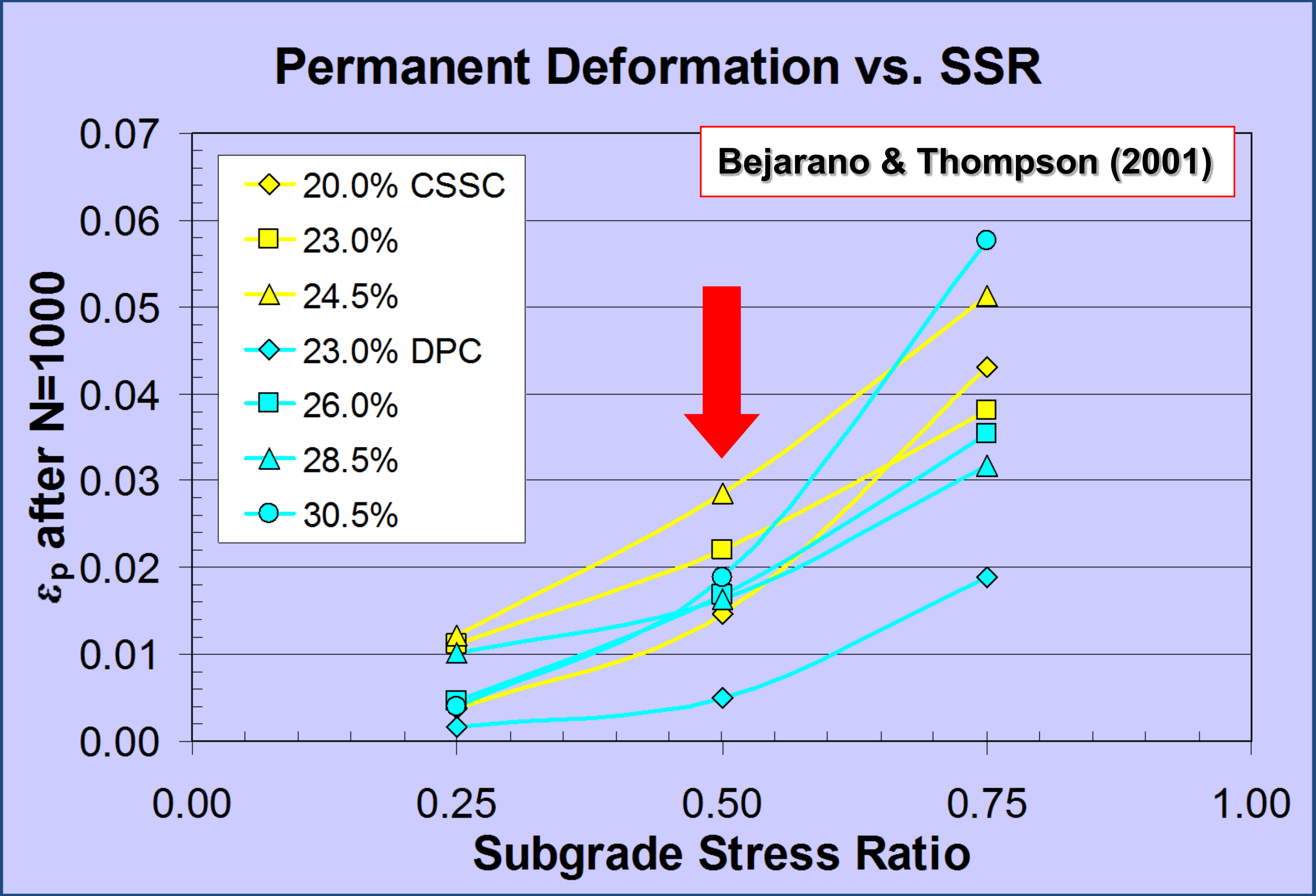
SUBGRADE STRESS RATIO (SSR)

SSR = SUBGRADE DEVIATOR STRESS / Q_U

Transfer Functions: Subgrade Rutting-SSR



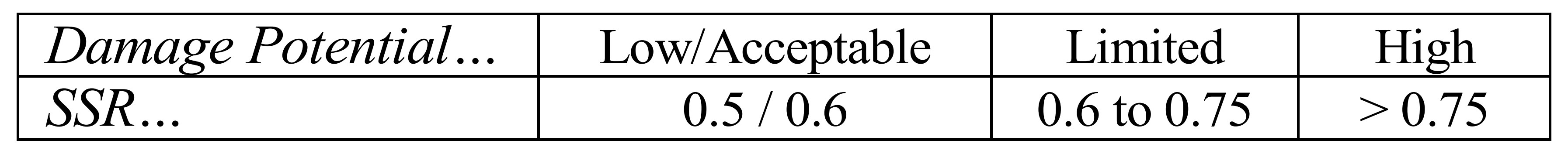
Transfer Functions:
Subgrade Rutting-SSR



Transfer Functions: Subgrade Rutting-SSR

SSR General Guidelines

UNIVERSITY OF IL R&D



Damage Potential…	Low/Acceptable	Limited	High
SSR…	0.5 / 0.6	0.6 to 0.75	> 0.75

WES: BETA – COVERAGE – SSR RELATIONS

C: COVERAGES

SSR: SUBGRADE STRESS RATIO

LOG (BETA)=((1.7782 + (0.2397* LOG C))/ ((1 + (0.5031 * LOG C))

BETA = (3.14 * SUBGRADE VERTICAL STRESS) / CBR

CBR~ Q_U (psi) / 4.5

SSR= SUBGRADE VERTICAL STRESS / Q_U

THUS: SSR = BETA / 14.1

COVERAGES	BETA	SSR
100	13.34	0.946
1,000	9.89	0.701
10,000	8.10	0.574
20,000	7.72	0.548
50,000	7.30	0.518
100,000	7.02	0.498

HIGH ESAL PAVEMENTS

- SUBGRADE RUTTING –
  NORMALLY NOT A PROBLEM
- “WORKING PLATFORM” –
  ESSENTIAL FOR PAVING!!!!!

GRANULAR MATERIAL RUTTING

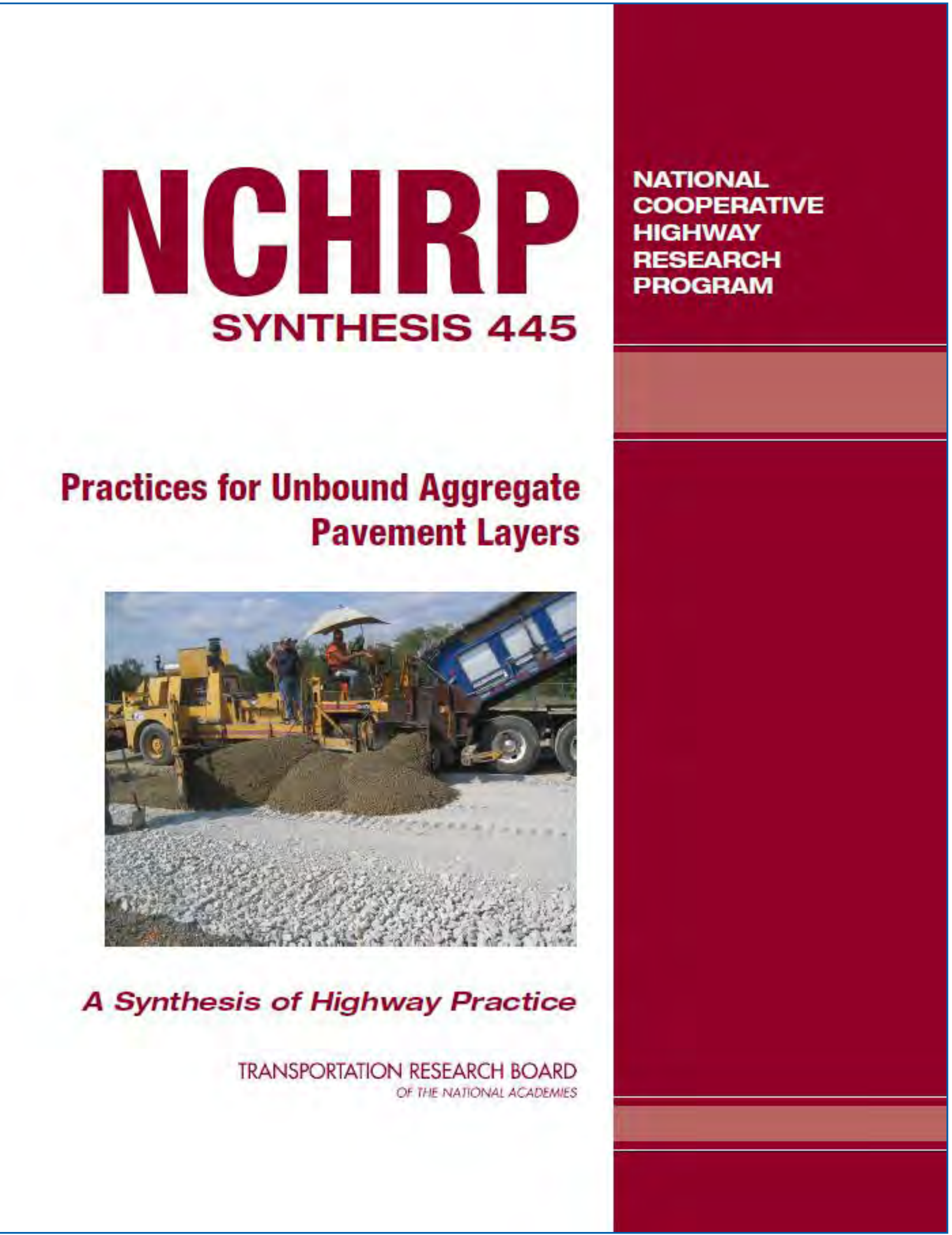
MINIMUM HMA “COVER THICKNESS”

MEPDG RUTTING MODEL

GRANULAR MATERIALS

Permanent Deformation Models

NCHRP
Synthesis 445
(Tutumluer - 2013)



NCHRP
SYNTHESIS 445

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Practices for Unbound Aggregate Pavement Layers

A Synthesis of Highway Practice

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

APPENDIX E

Review of Current Permanent Deformation Models

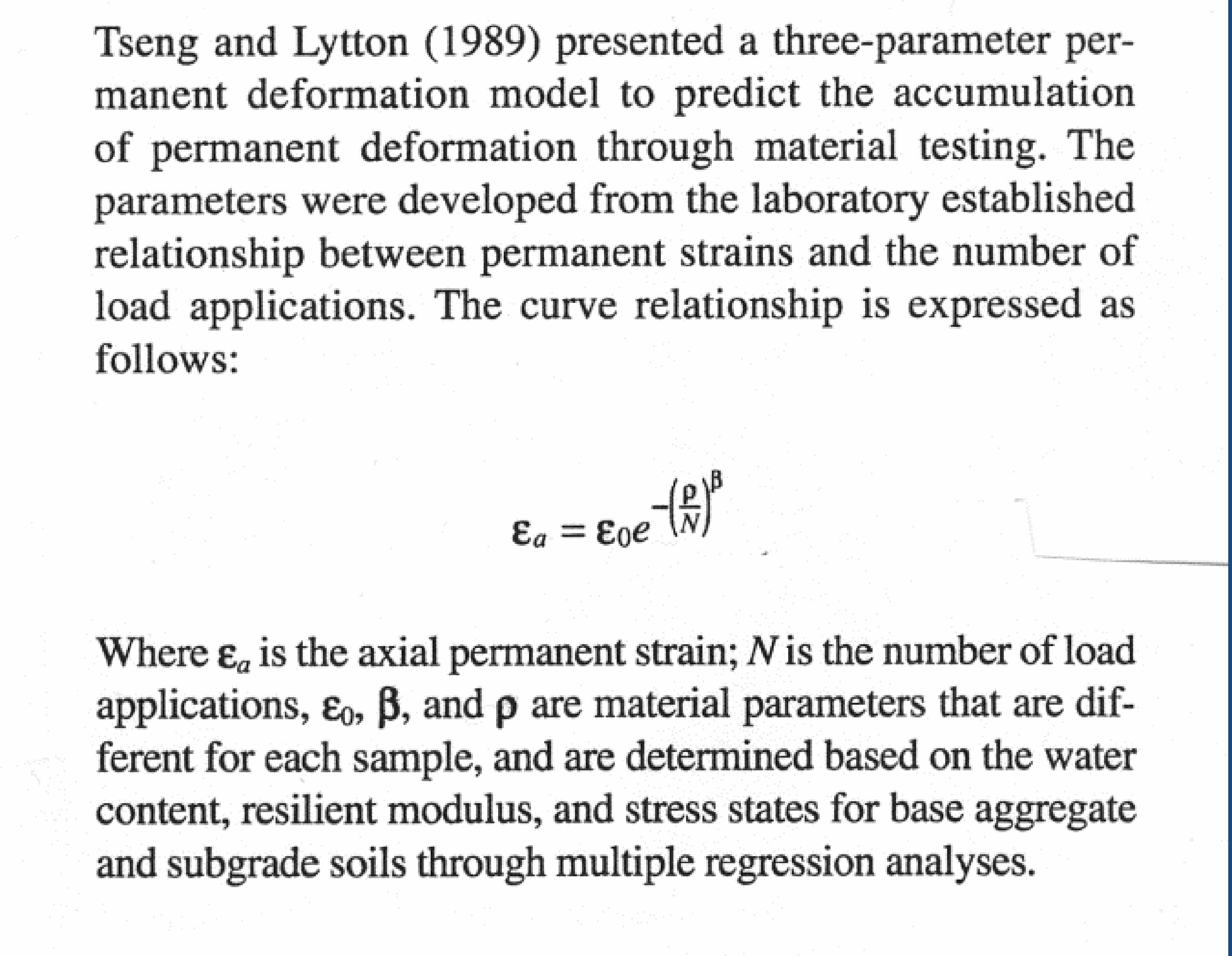
Typical Model Forms

$$\varepsilon_p = a + b(\mathrm{LOG}\ N)$$

$$\varepsilon_p = A\ N^b$$

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 ε_a is the axial permanent strain; N is the number of load applications, ε_0, β, and ρ 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.

Basic Form for MEPDG
TSENG & LYTTON
ASTM STP 1016 (1989)

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
β_1 = Field calibration factor
ε_0, ρ = Material properties
ε_r = Resilient strain from lab tests to determine material properties
ε_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

I L L I N O I S
1867
UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN

ILLINOIS CENTER FOR TRANSPORTATION

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
β_1 = Field calibration factor
ε_0, ρ = Material properties
ε_r = Resilient strain from lab tests to determine material properties
ε_v = Vertical resilient strain computed for sublayer
h = Sublayer thickness

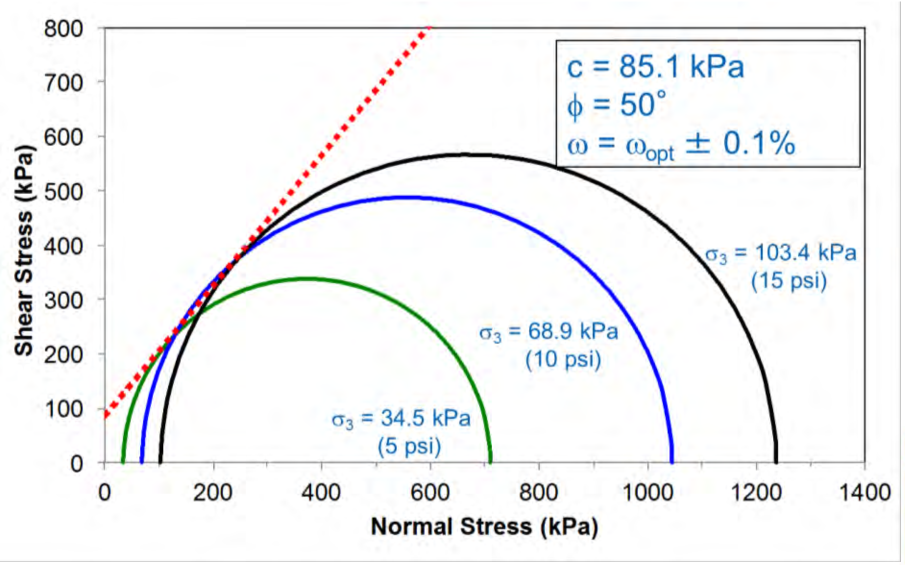
$f\,(N$, thickness, M_R, W_c, $\varepsilon_r)$ ➡ **No stress state!**

Aggregate Shear Strength Properties

Label	Cohesion c		Friction Angle f	Compaction Water Content
	psi	kPa	degree	%
Material G1	12.4	85.1	50	$\omega_{opt} \pm 0.1$
Material G2	8.6	59.4	45	$\omega_{opt} \pm 0.8$
Material B	0.2	1.1	51	$\omega_{opt} \pm 0.2$
Material L	0.3	2.4	45	$\omega_{opt} \pm 0.1$

w_{opt} = Optimum moisture content

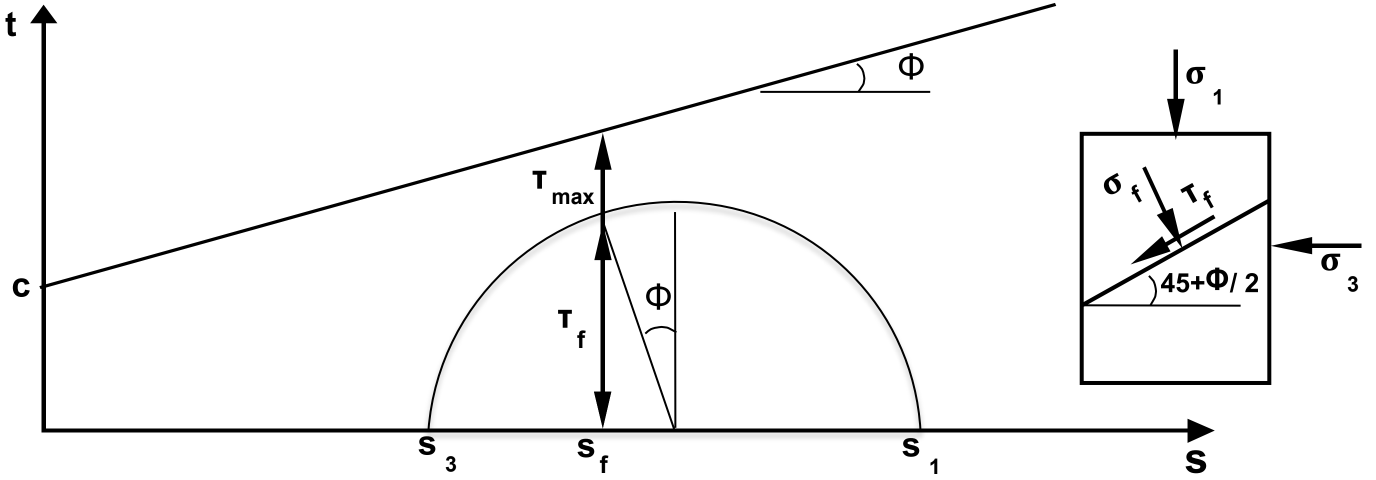
N.C. DOT data



Shear Stress Ratio (SSR) Concept

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

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



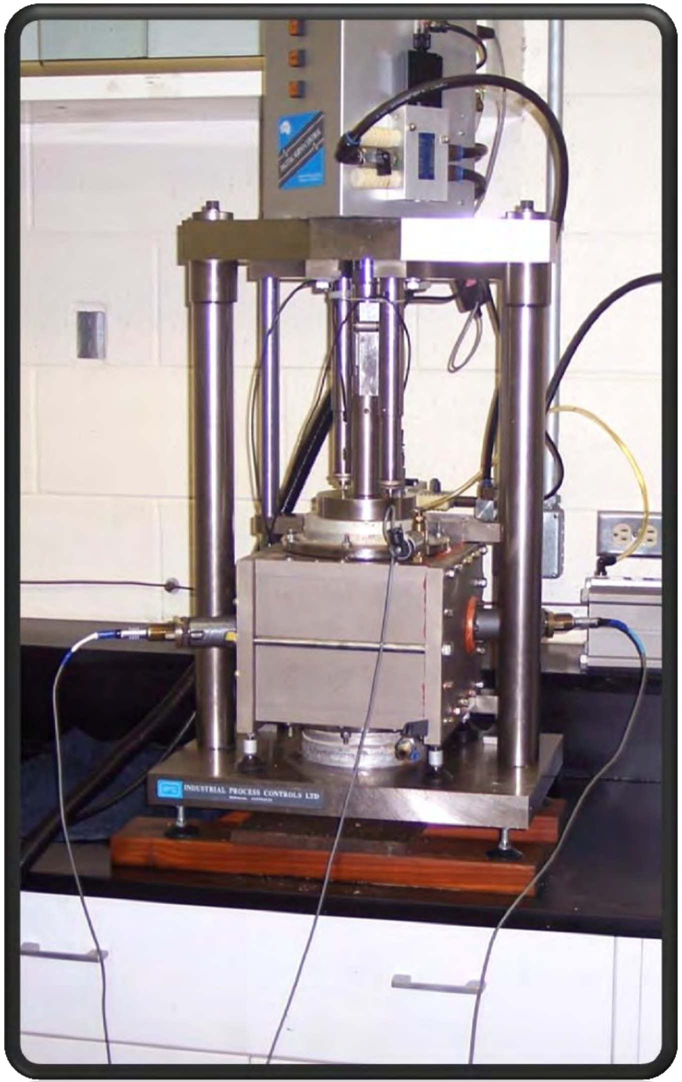
$$\sigma_f = \frac{2\sigma_3 + 2\tan^2\phi\,\sigma_3 + \sigma_d + \tan^2\phi\,\sigma_d - \sqrt{\tan^2\phi\,\sigma_d{}^2 + \tan^4\phi\,\sigma_d{}^2}}{2\,(1+\tan^2\phi)}$$

$$\tau_f = \sqrt{(\sigma_d/2)^2 - (\sigma_f - (\sigma_3 + \sigma_d/2))^2}$$

Repeated Load Triaxial Testing for Permanent Deformation Characterization

Specimen Preparation and Setup

```
[FD22] V3.04 User Programmable Test                    TEST RUNNING
                                                   Load kN: 0.7131
 BLK1
           Test Time: 00:11:26
        Cycle Number    686                   Actual Specimen Height, mm 149.85
 Dynamic(vert), psi    5.05                     Resilient Strain % 0.00039
 Dynamic(horiz), psi   0.51                     Permanent Strain % 0.00098
 Confining(vert), psi   4.81                   Resilient Modulus % 14.964
 Confining(horiz),psi   4.68
   Resilient Modulus 19.680
```

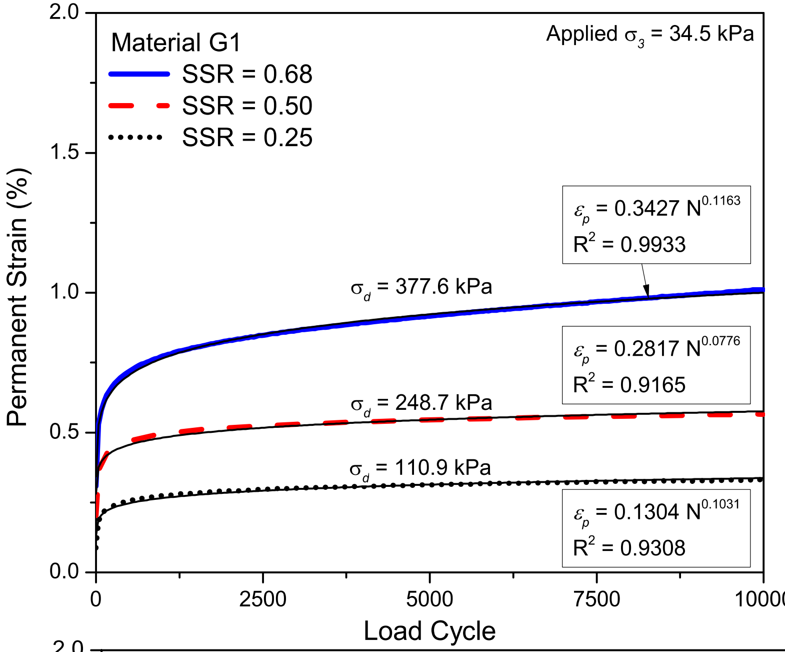


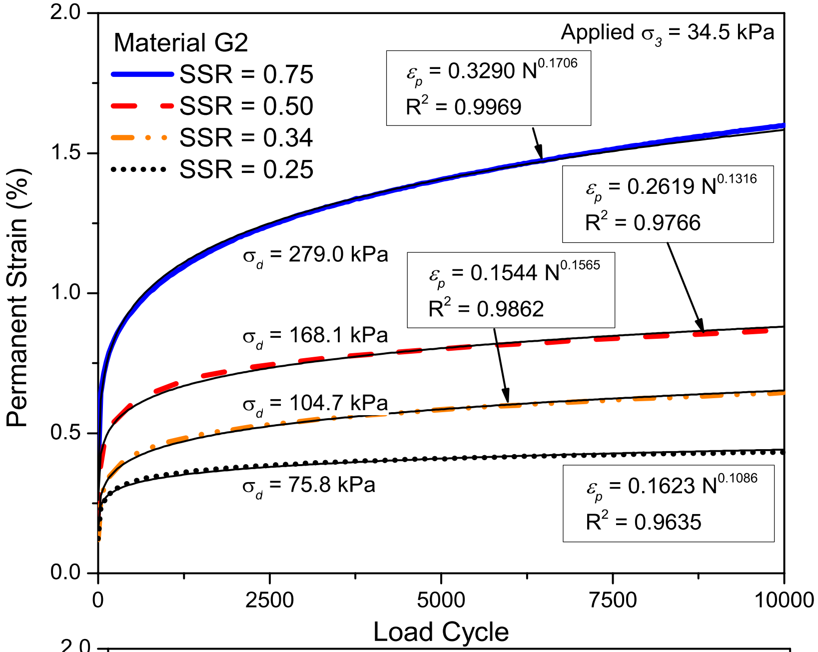
University of Illinois – FastCell

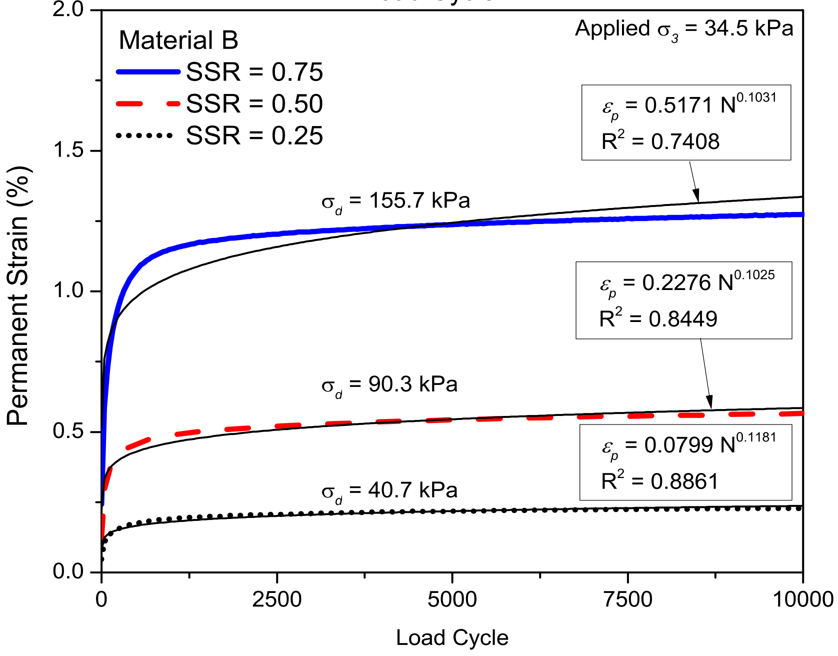
Test Protocol

- 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

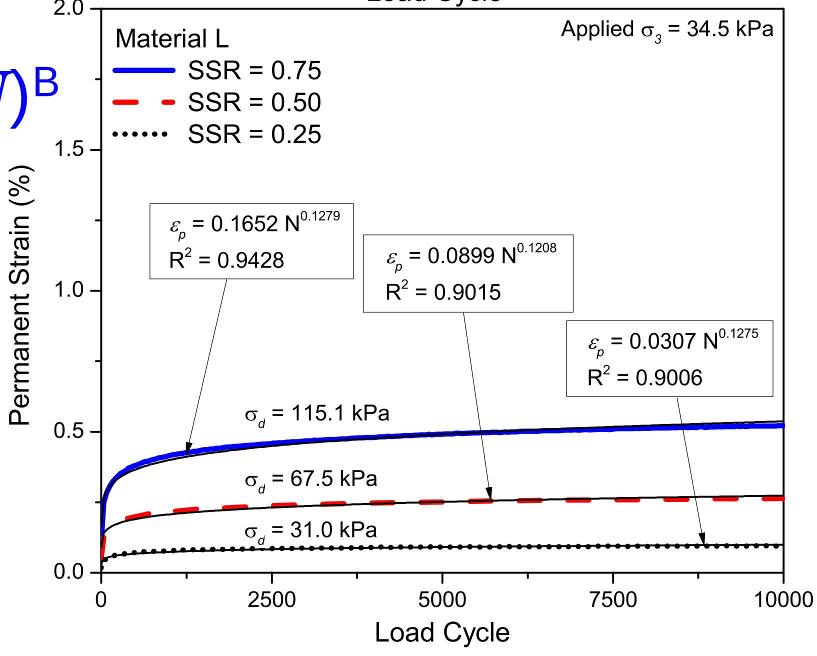
Permanent Deformation Test Results







$$\varepsilon_p = A\,(N)^B$$



Development of Improved Rutting Model

$$\boxed{\varepsilon_p = \mathrm{A}\,(N)^{\mathrm{B}}\,(\sigma_{\mathrm{d}})^{\mathrm{C}}\,(\tau_f/\tau_{\mathrm{max}})^{\mathrm{D}}}$$

A, B, C, D = Regression parameters

ε_p = Permanent strain

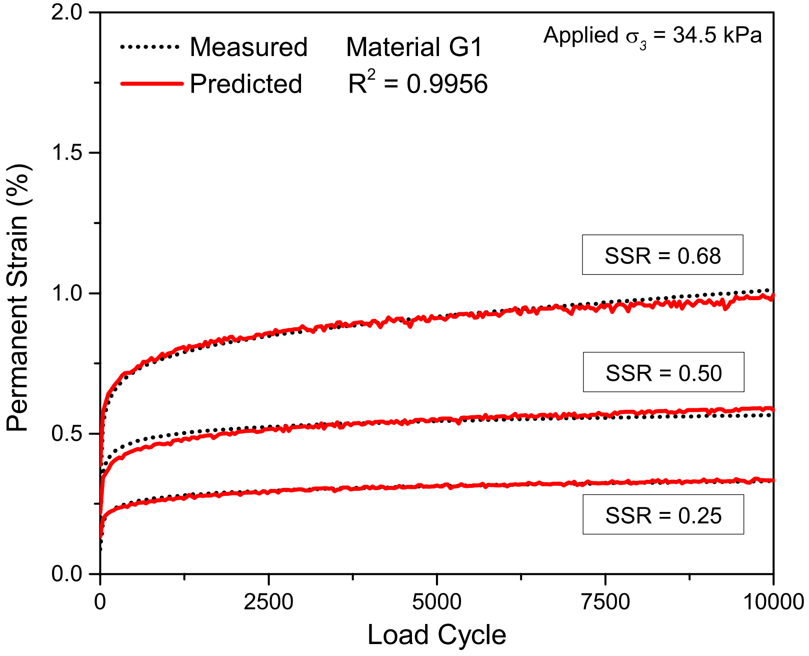
N = Load cycle

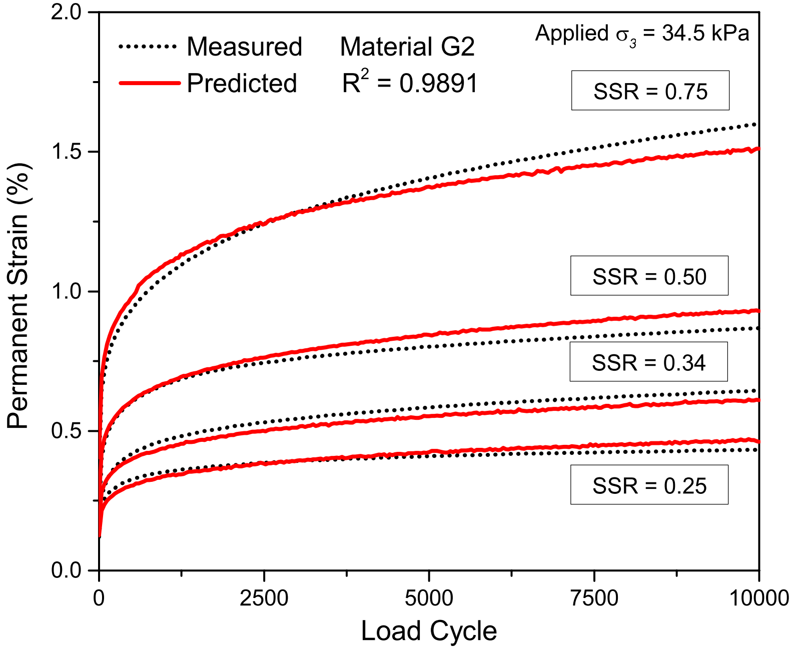
σ_{d} = Applied stress

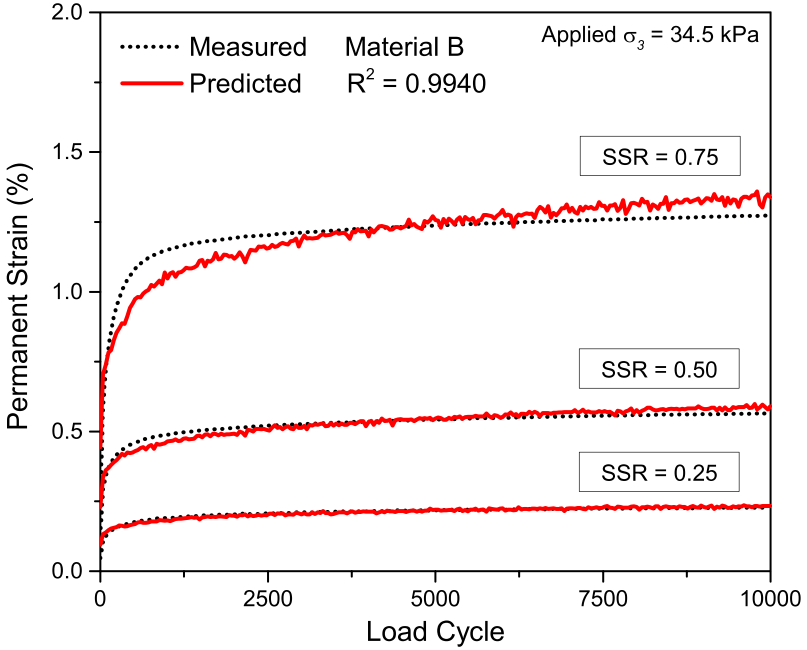
τ_f = Applied shear stress

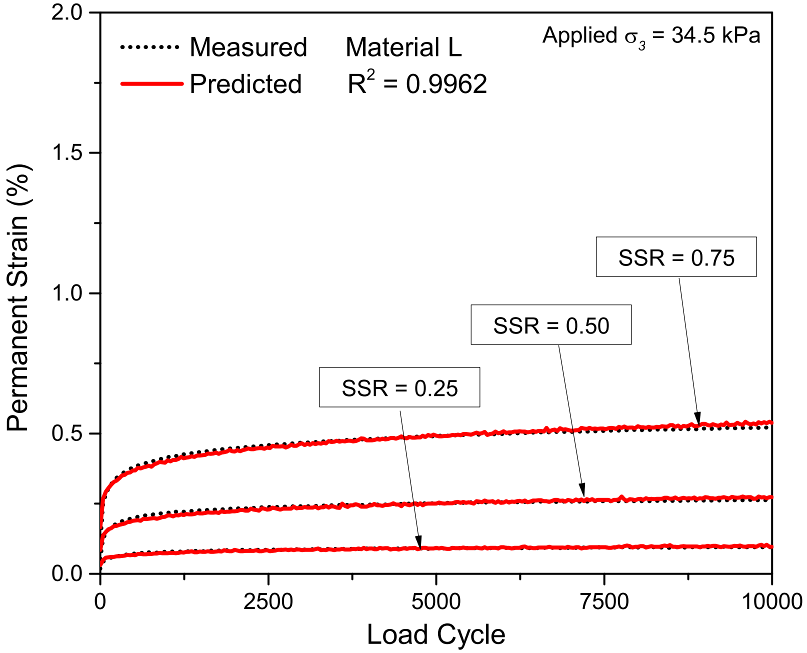
τ_{max} = Shear strength at failure

$$\varepsilon_p = A\,(N)^B\,(\sigma_d)^C\,(\tau_f/\tau_{max})^D$$





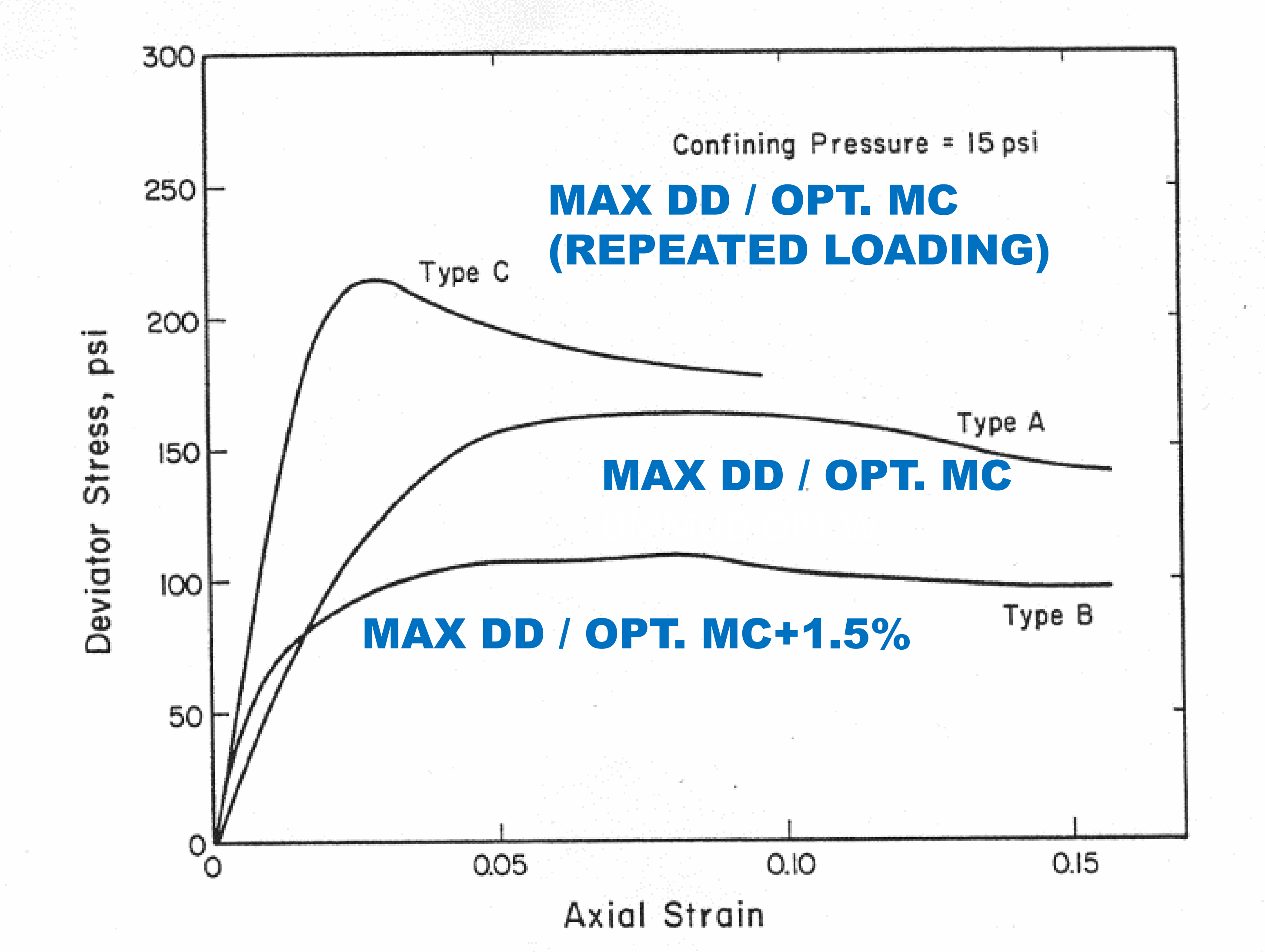




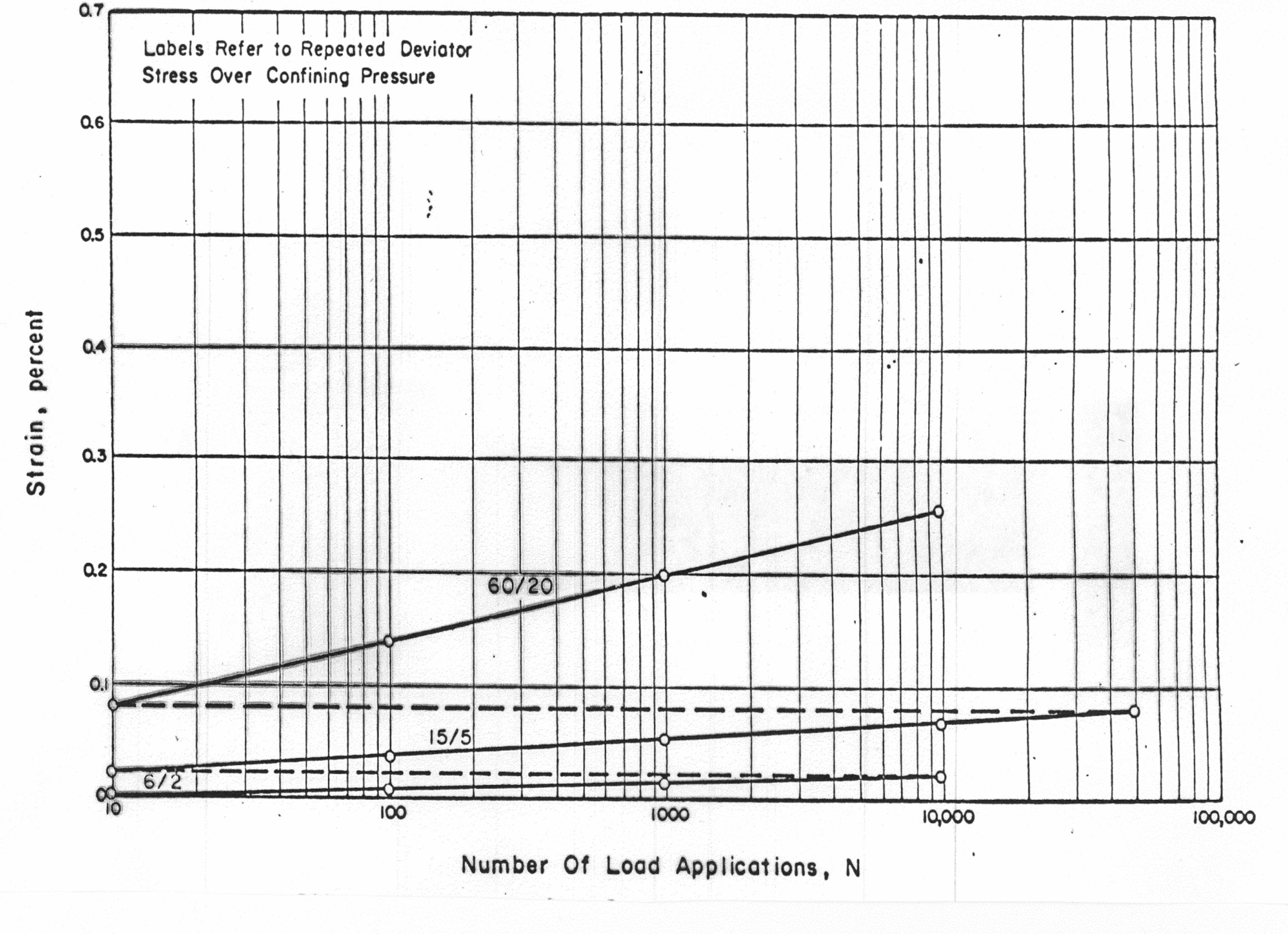
STRENGTH PARAMETERS ARE IMPORTANT FACTORS IN PREDICTING PERMANENT DEFORMATION OF GRANULAR MATERIALS!!!!

COMPLICATING FACTORS

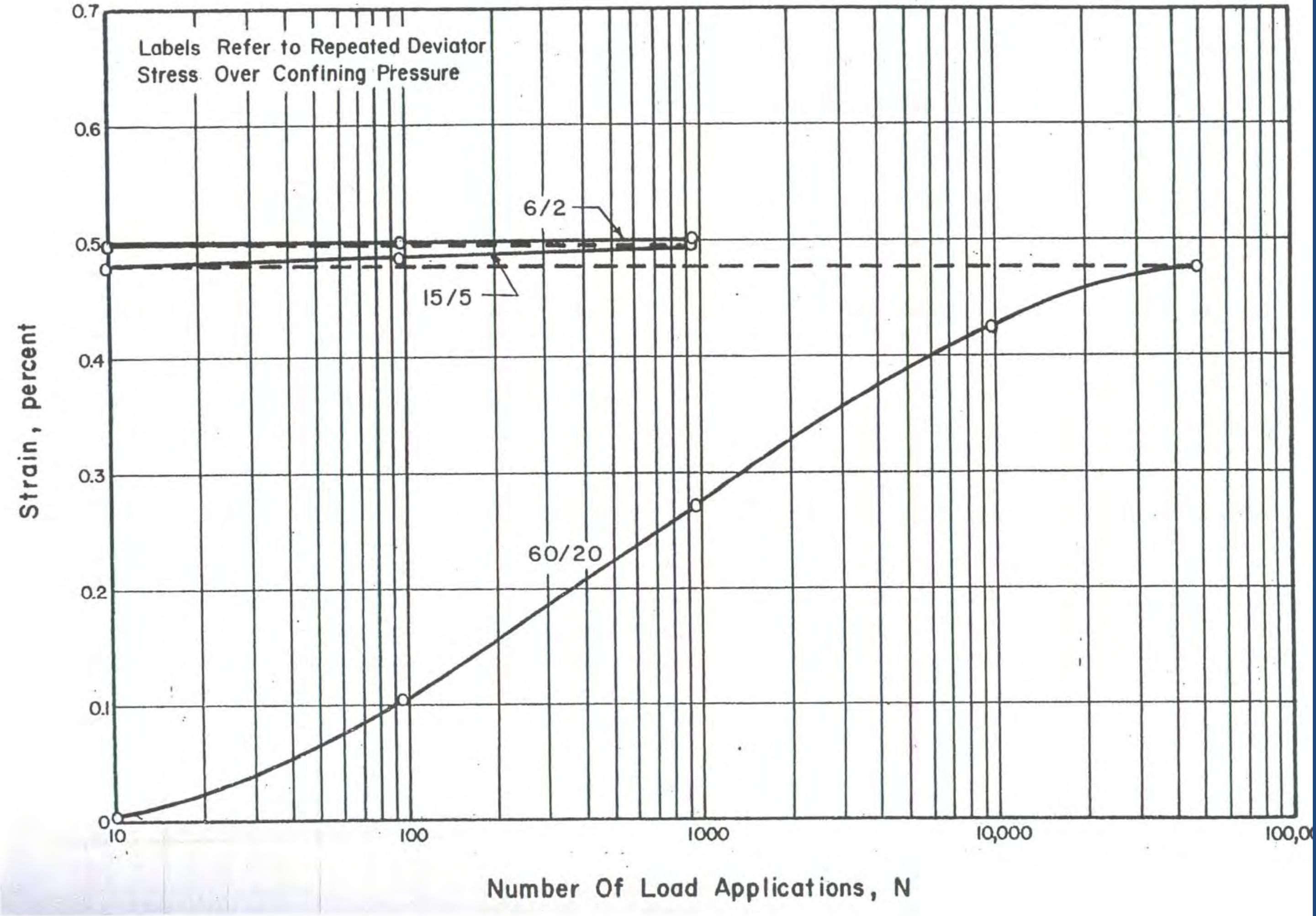
- STRENGTH INCREASE WITH LOADING
- STRESS HISTORY EFFECTS



SHEAR STRENGTH INCREASE WITH REPEATED LOADING
DENSE GRADED CRUSHED GRAVEL BASE
Thompson & Smith (TRR 1278)



STRESS HISTORY: LOW TO HIGH



STRESS HISTORY: HIGH TO LOW

CUMULATIVE DAMAGE

???

AASHTO Ware – ADVISORY
AASHTO Ware Pavement ME Design

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

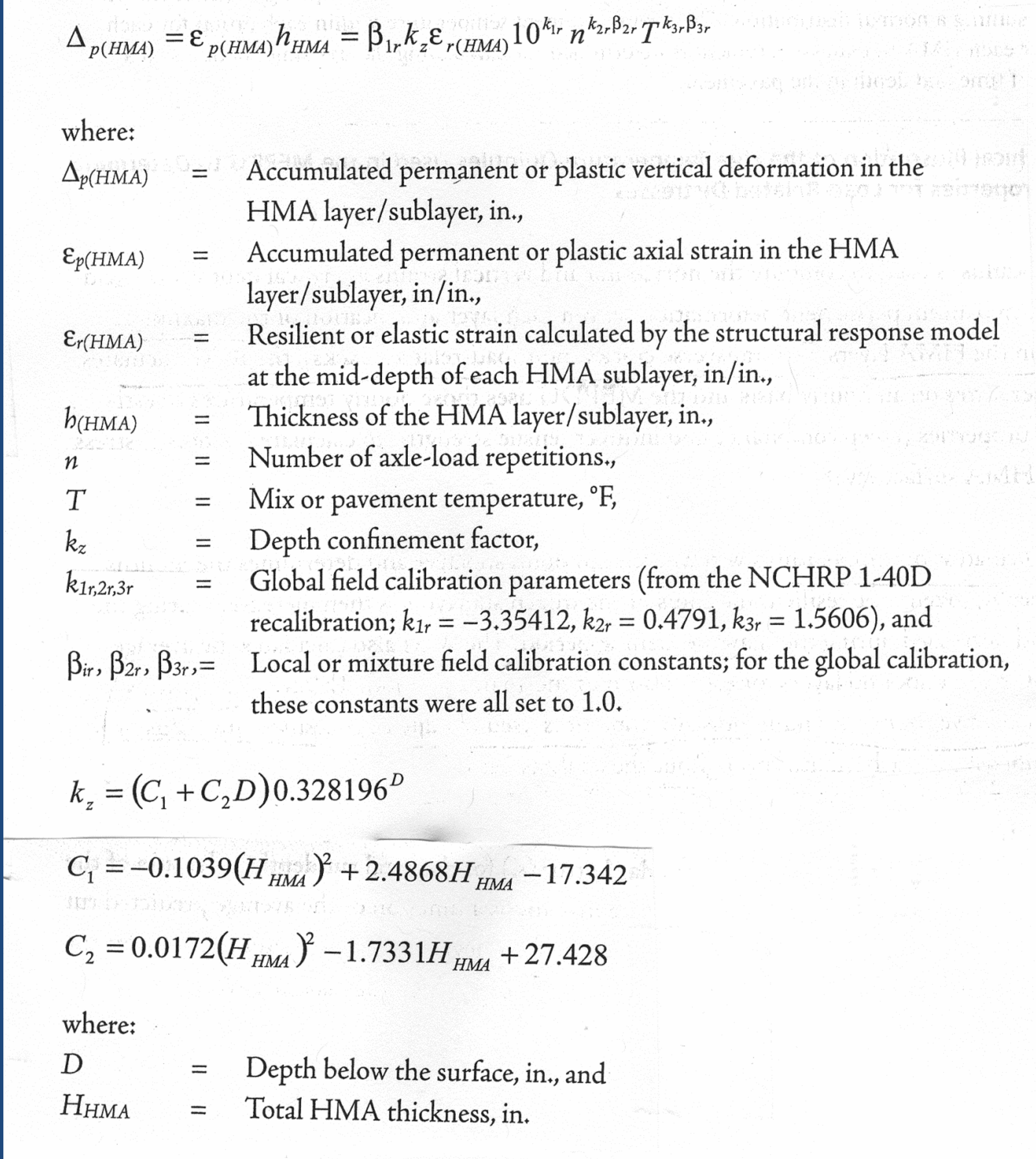
“AASHTO encourages each licensing agency to calibrate and validate using local materials”

* NCHRP 01-53: Proposed Enhancements to Pavement ME Design: Improved Consideration of the Influence of Subgrade and Unbound Layers on Pavement Performance.

* NC DOT Project @ University of Illinois

HMA RUTTING

MEPDG HMA RUTTING MODEL



$$\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{HMA} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} n^{k_{2r}\beta_{2r}} T^{k_{3r}\beta_{3r}}$$

where:

$\Delta_{p(HMA)}$ = Accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.,

$\varepsilon_{p(HMA)}$ = Accumulated permanent or plastic axial strain in the HMA layer/sublayer, in/in.,

$\varepsilon_{r(HMA)}$ = Resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA sublayer, in/in.,

$h_{(HMA)}$ = 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

$\beta_{ir}, \beta_{2r}, \beta_{3r}$,= Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

$$k_z = (C_1 + C_2 D) 0.328196^D$$

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868 H_{HMA} - 17.342$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331 H_{HMA} + 27.428$$

where:

D = Depth below the surface, in., and

H_{HMA} = Total HMA thickness, in.

NATIONAL RUTTING MODEL

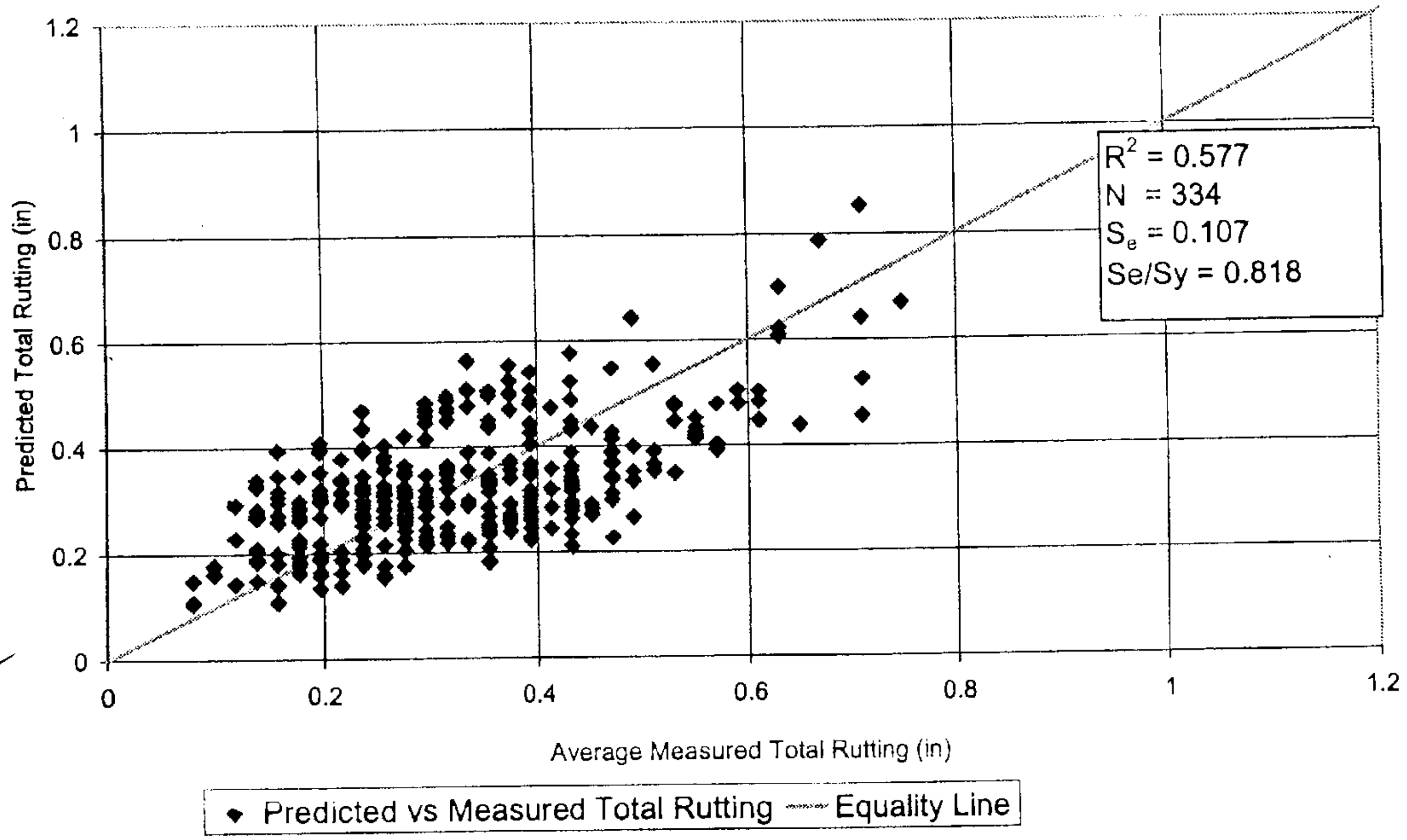
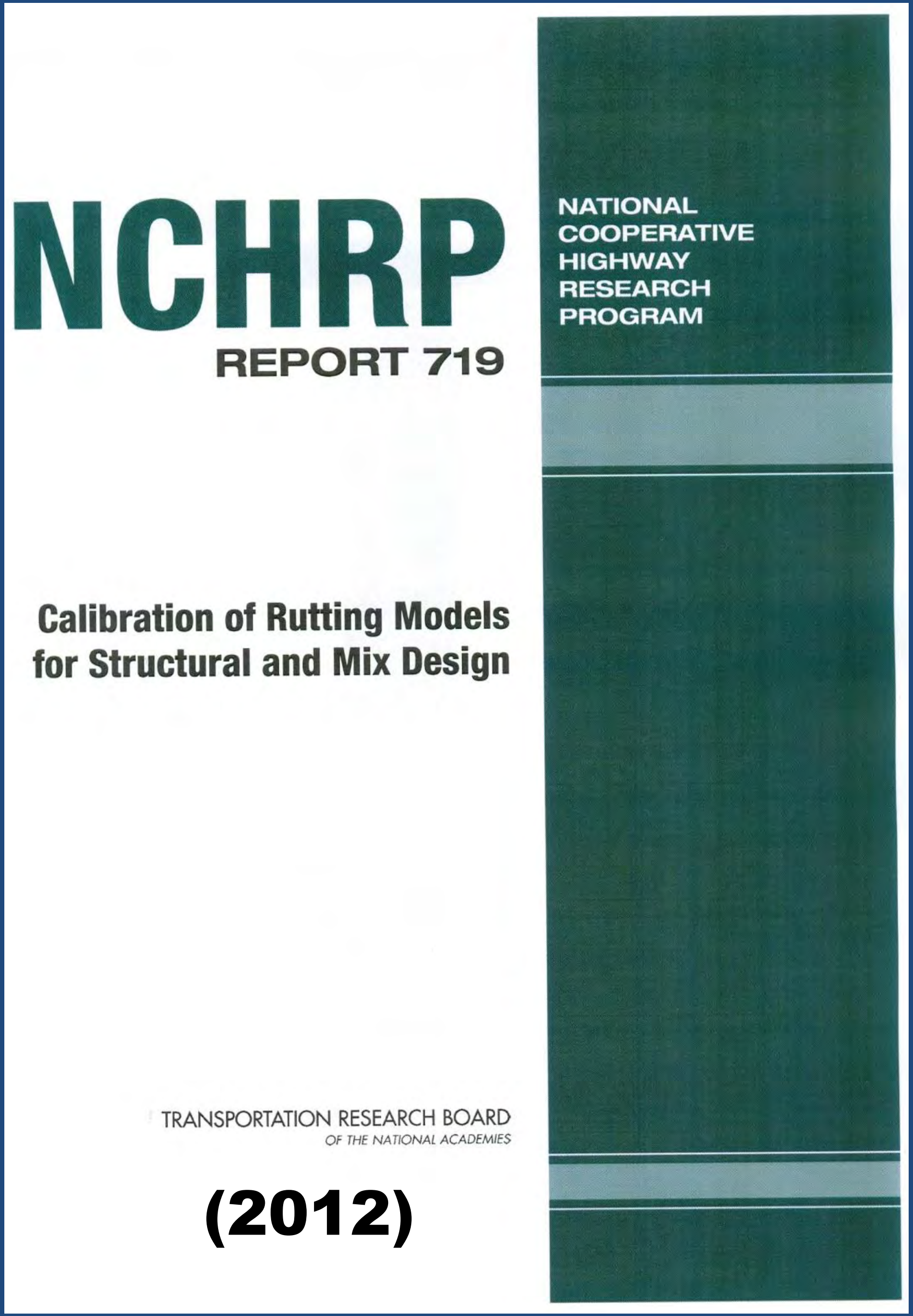


Figure 10. **Comparison of Measured and Predicted Total Rutting Resulting from Global Calibration Process**



NCHRP

REPORT 719

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Calibration of Rutting Models
for Structural and Mix Design

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

(2012)

“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 (σ_{DEV})
- Asphalt Institute - Modified Leahy (σ_{DEV} and ε_V)
- WesTrack (shear strain and stress)

TRANSFER FUNCTION	R^2	S_e – in.	S_e / S_y
MEPDG	**0.583**	**0.1085**	**0.665**
Modified Leahy	**0.699**	**0.1045**	**0.611**
WesTrack	**0.712**	**0.091**	**0.585**

“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

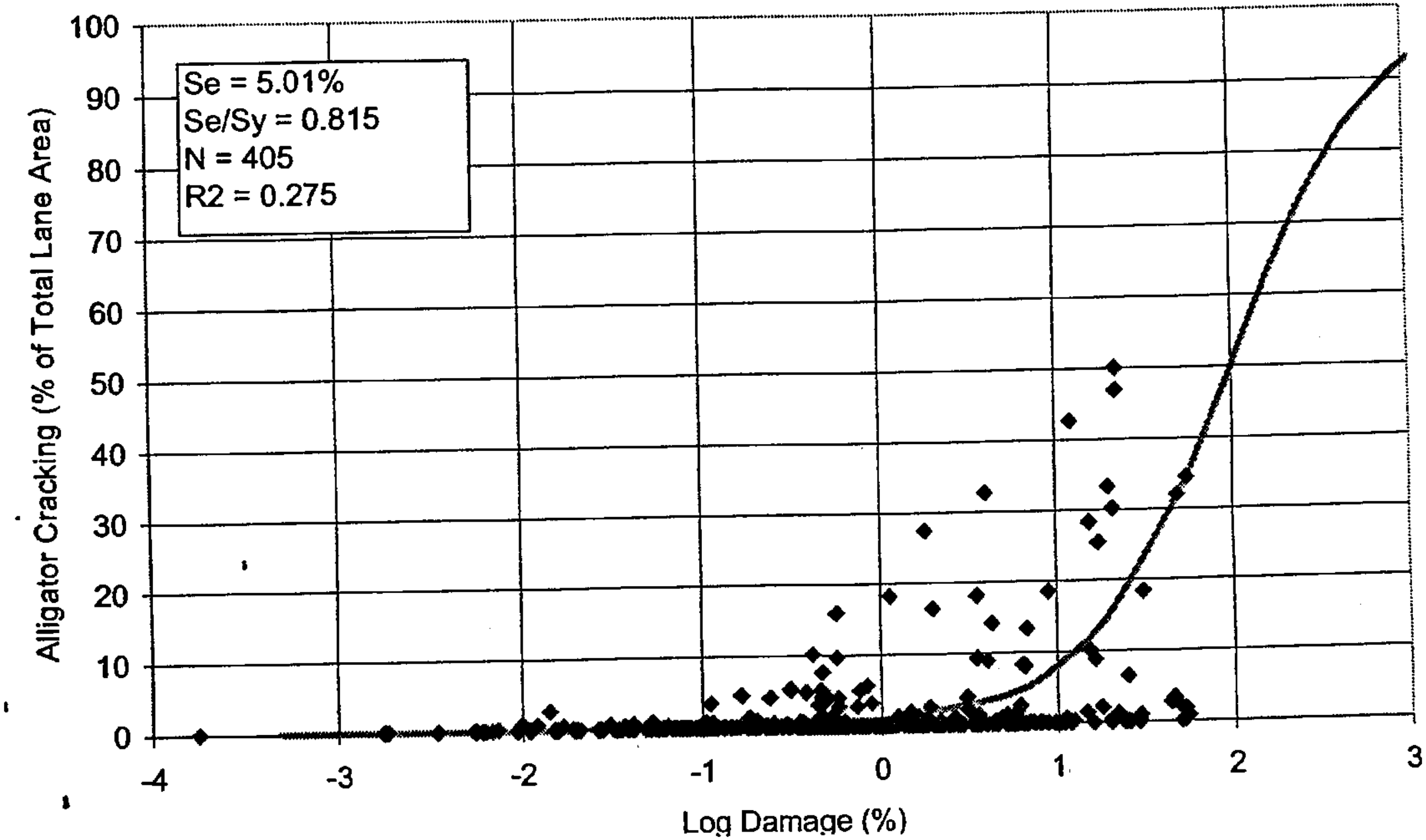


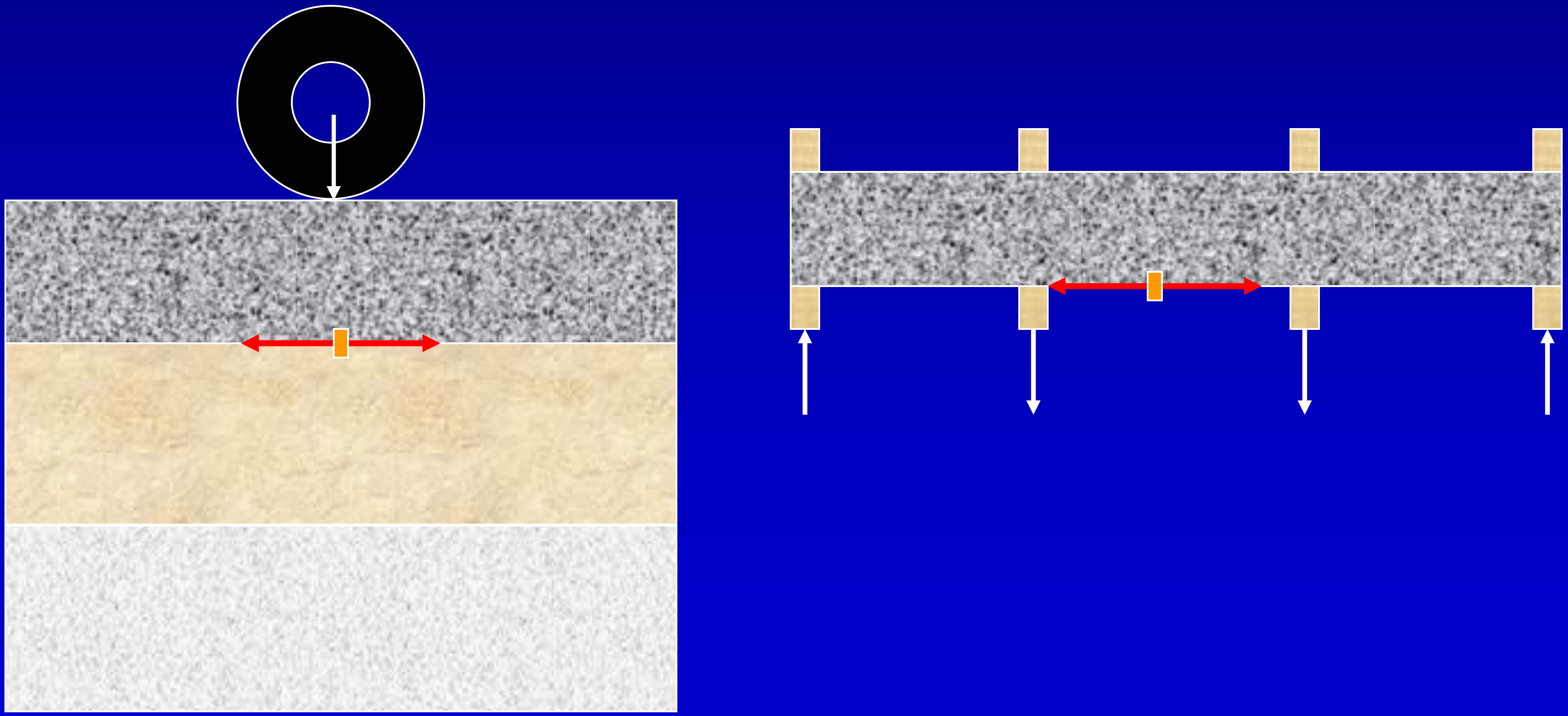
Figure 11 Comparison of Cumulative Fatigue Damage and Measured Alligator Cracking Resulting from Global Calibration Process

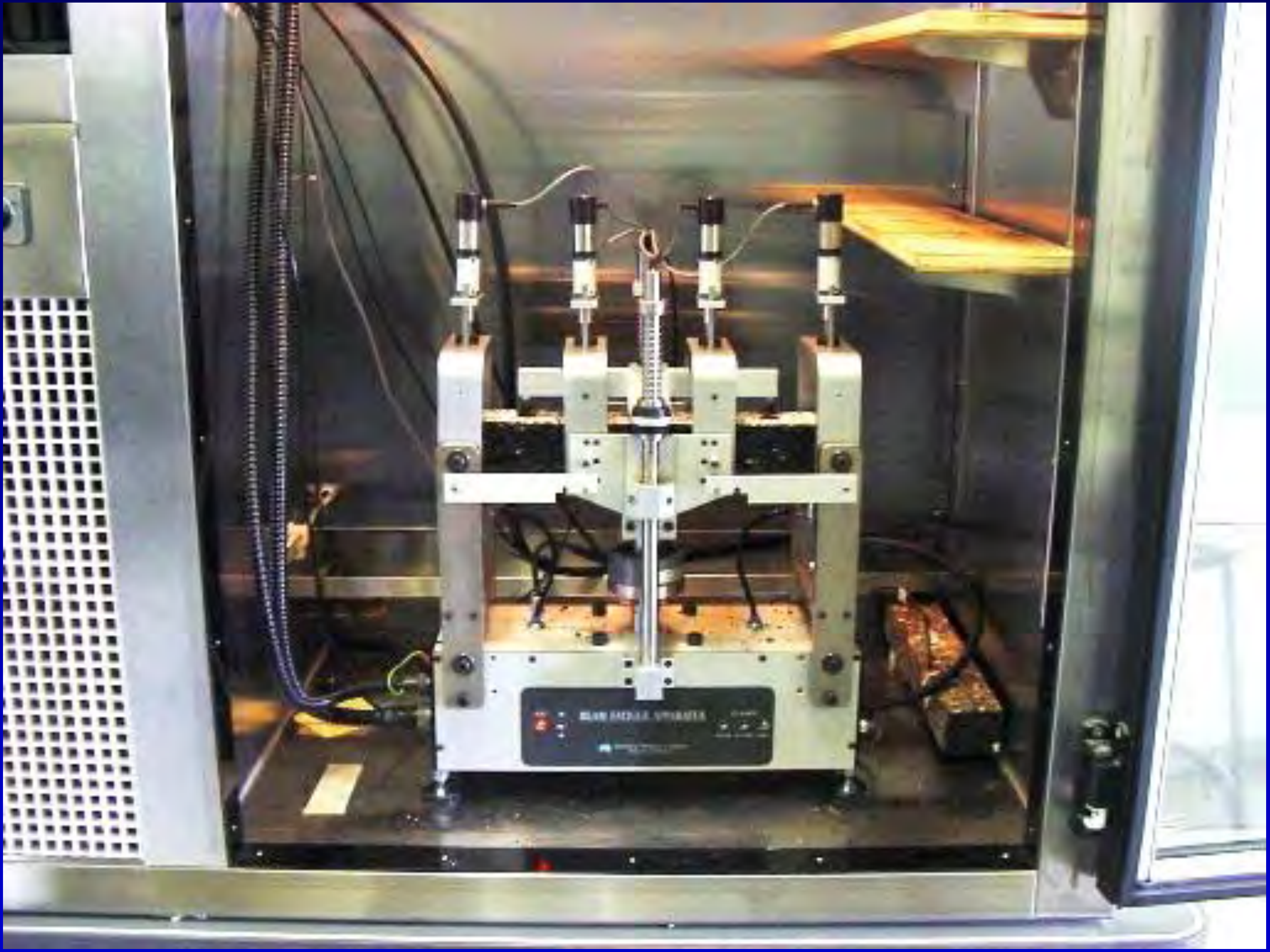
AASHTO TP 8-94

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

FATIGUE DESIGN

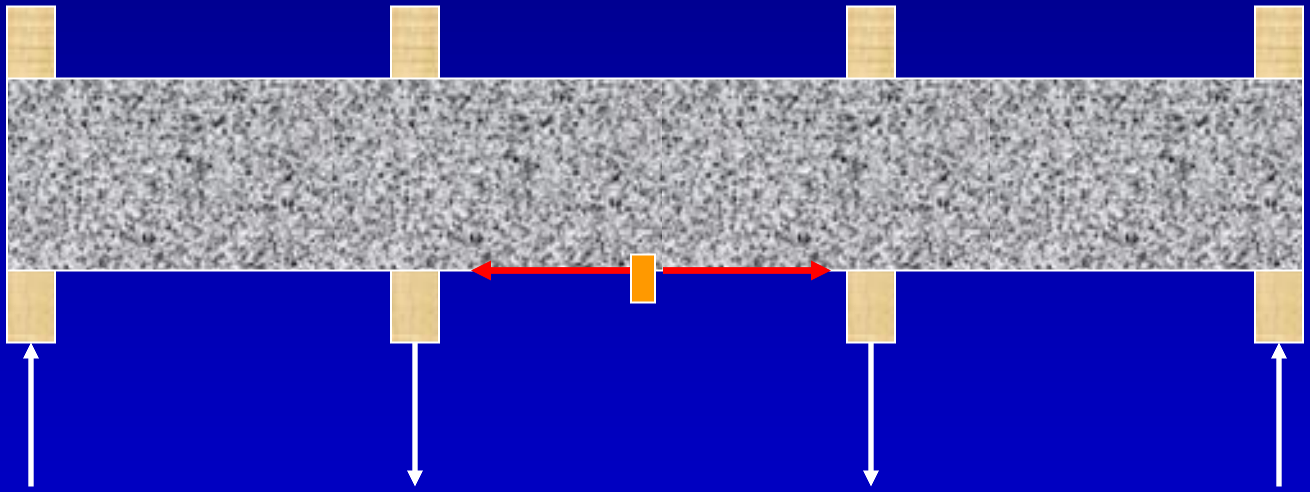
- Tensile Strain at Bottom of Asphalt
- Tensile Strain in Flexural Beam Test
  - Other Configurations





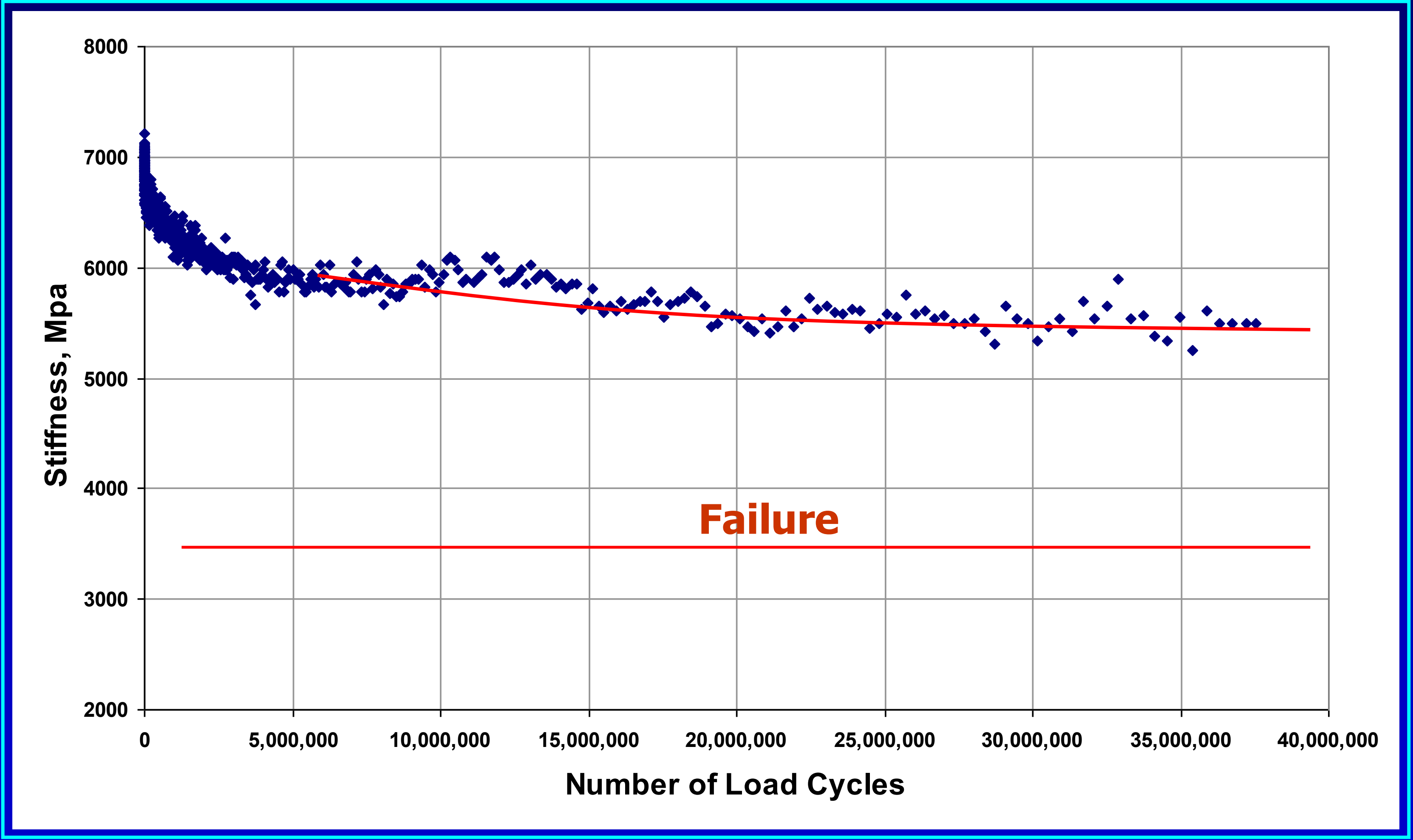
FATIGUE TESTING

- **Tensile Strain in Flexural Beam Test**
  - **Other Configurations**

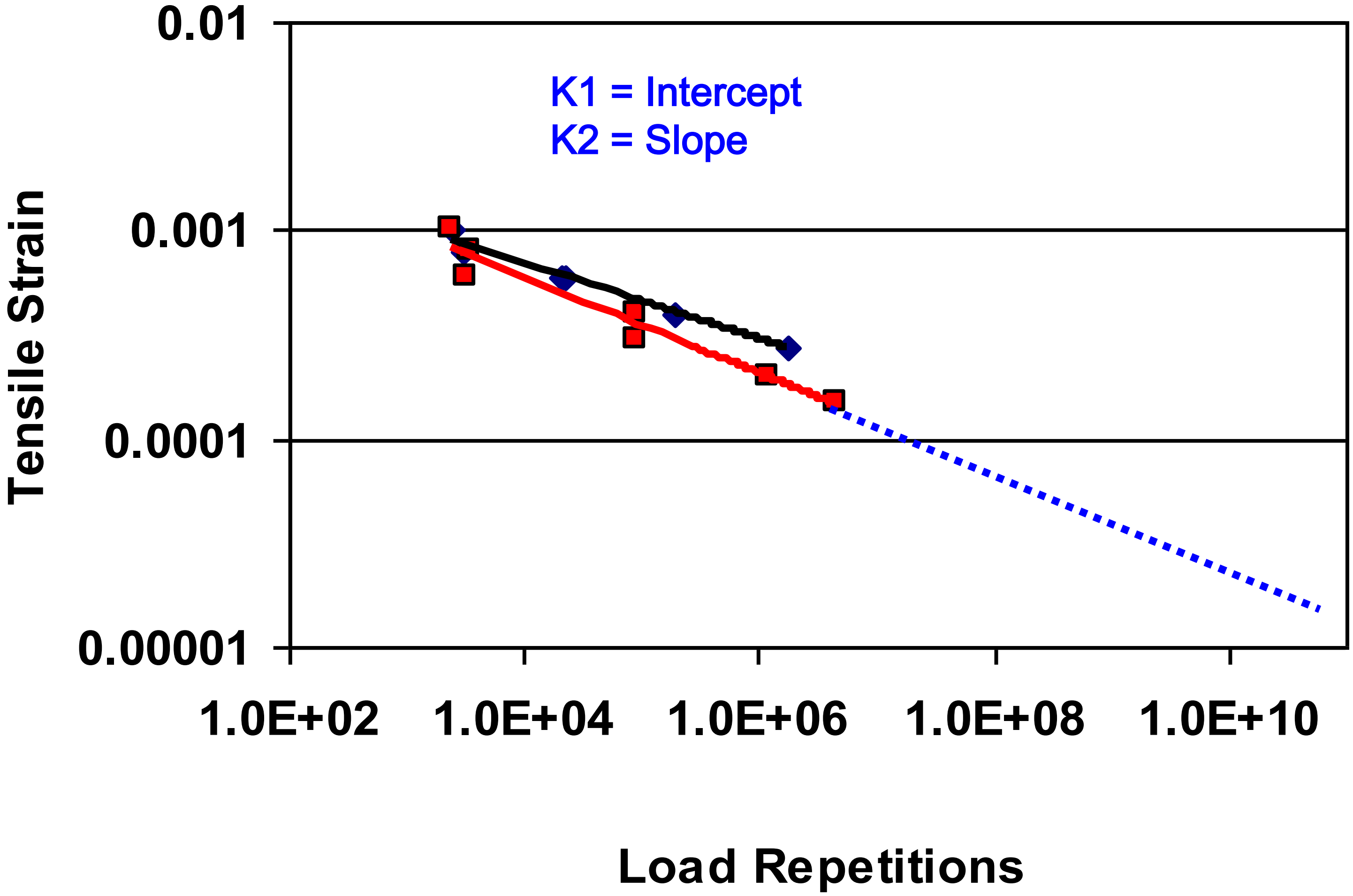


  - **10 Hz Haversine Load, 20° C, Controlled Strain**

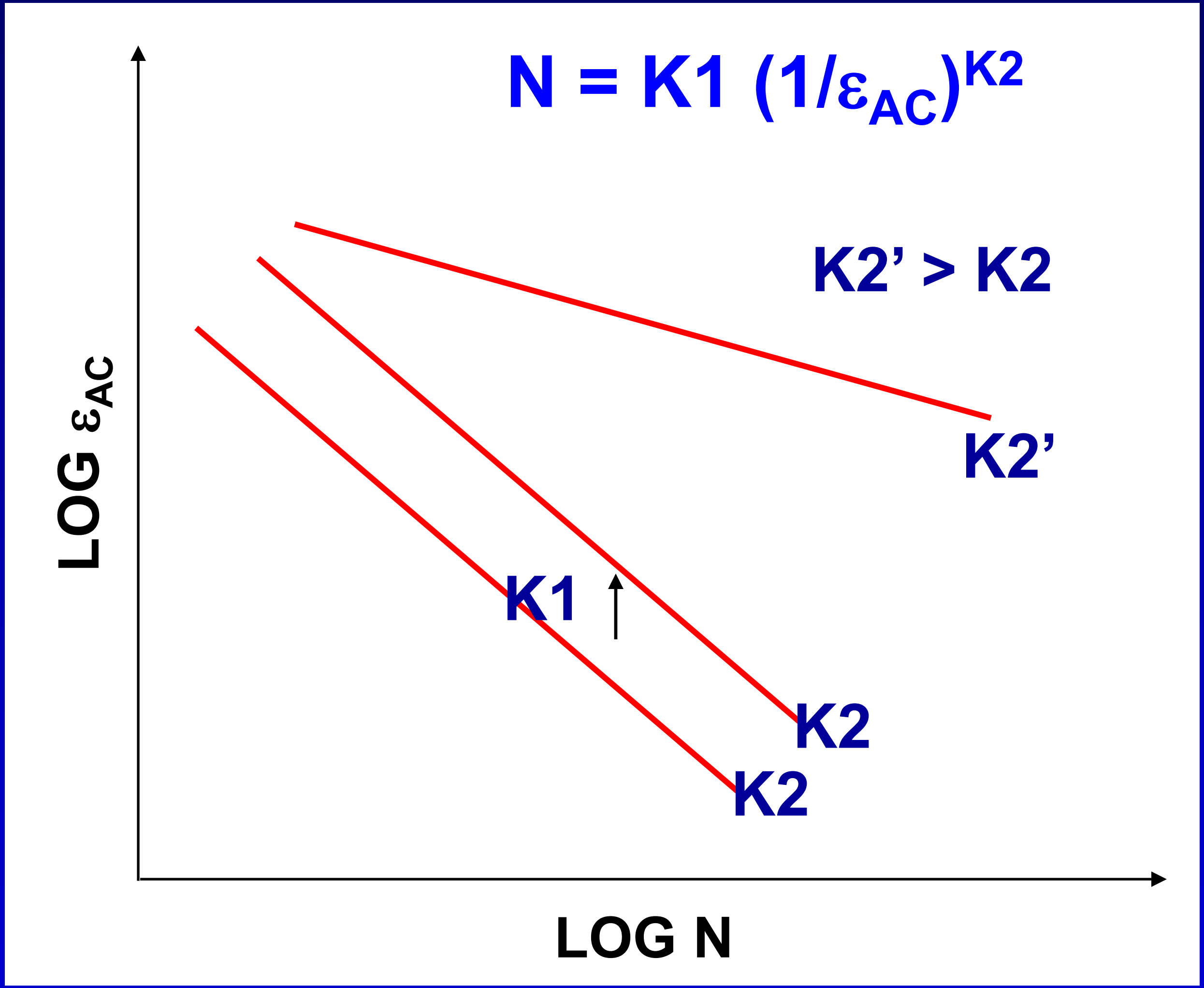
STIFFNESS CURVE



LABORATORY ALGORITHM



AC FATIGUE



$$N = K1\,(1/\varepsilon_{AC})^{K2}$$

K2’ > K2

FATIGUE ALGORITHMS

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

AASHTO MEPDG FORMAT

$$N_f = 0.00432 * k`_1 * C(1/\varepsilon)^{k2} (1/E_{HMA})^{k3}$$

$K`_1$ - 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

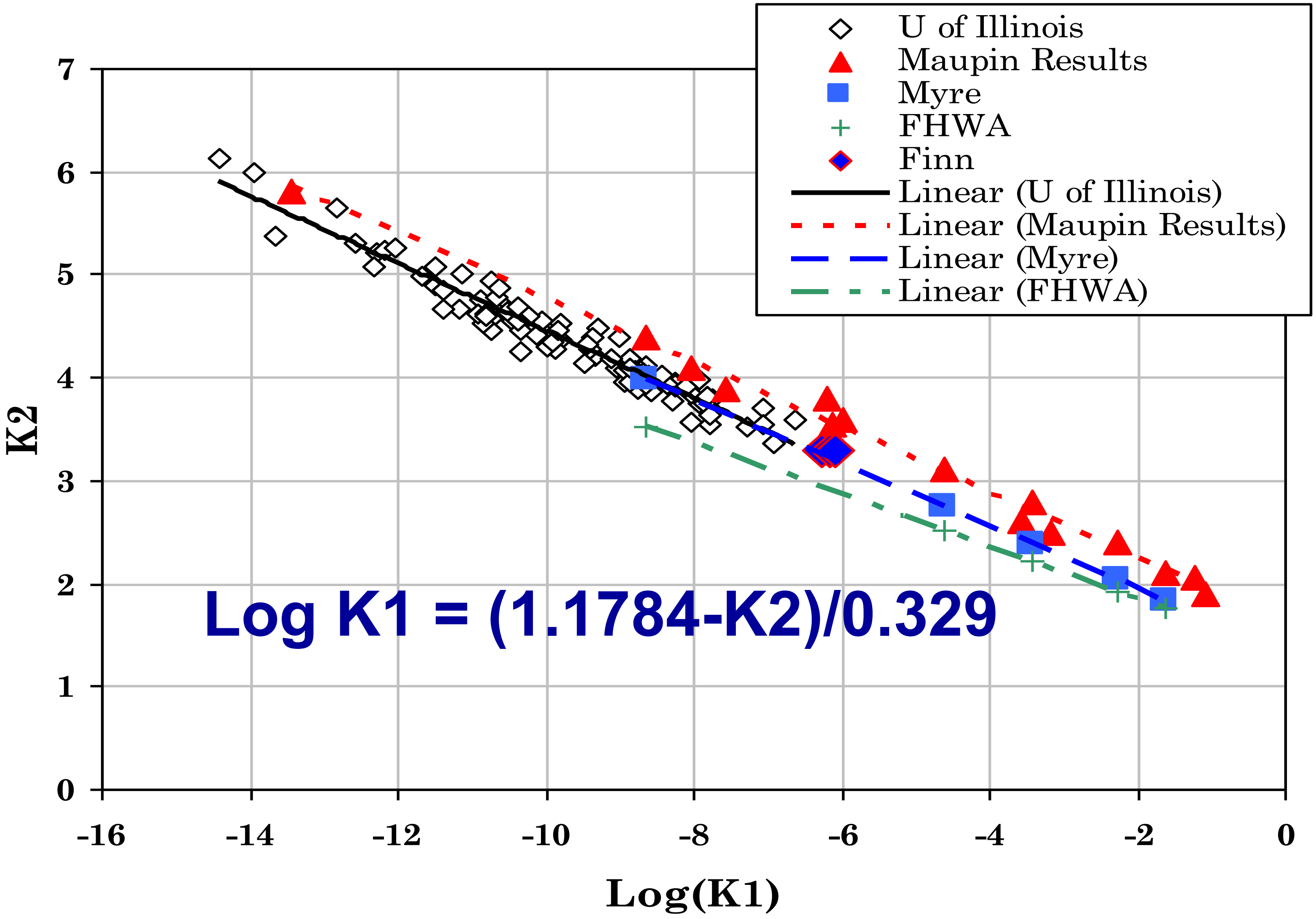
$$N = K1\,(1/\varepsilon)^{K2}$$

Minimum K2: 3.5

90% K2: 4.0

Average K2: 4.5

OTHER STUDIES



$$\text{Log K1} = (1.1784 - \text{K2})/0.329$$

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

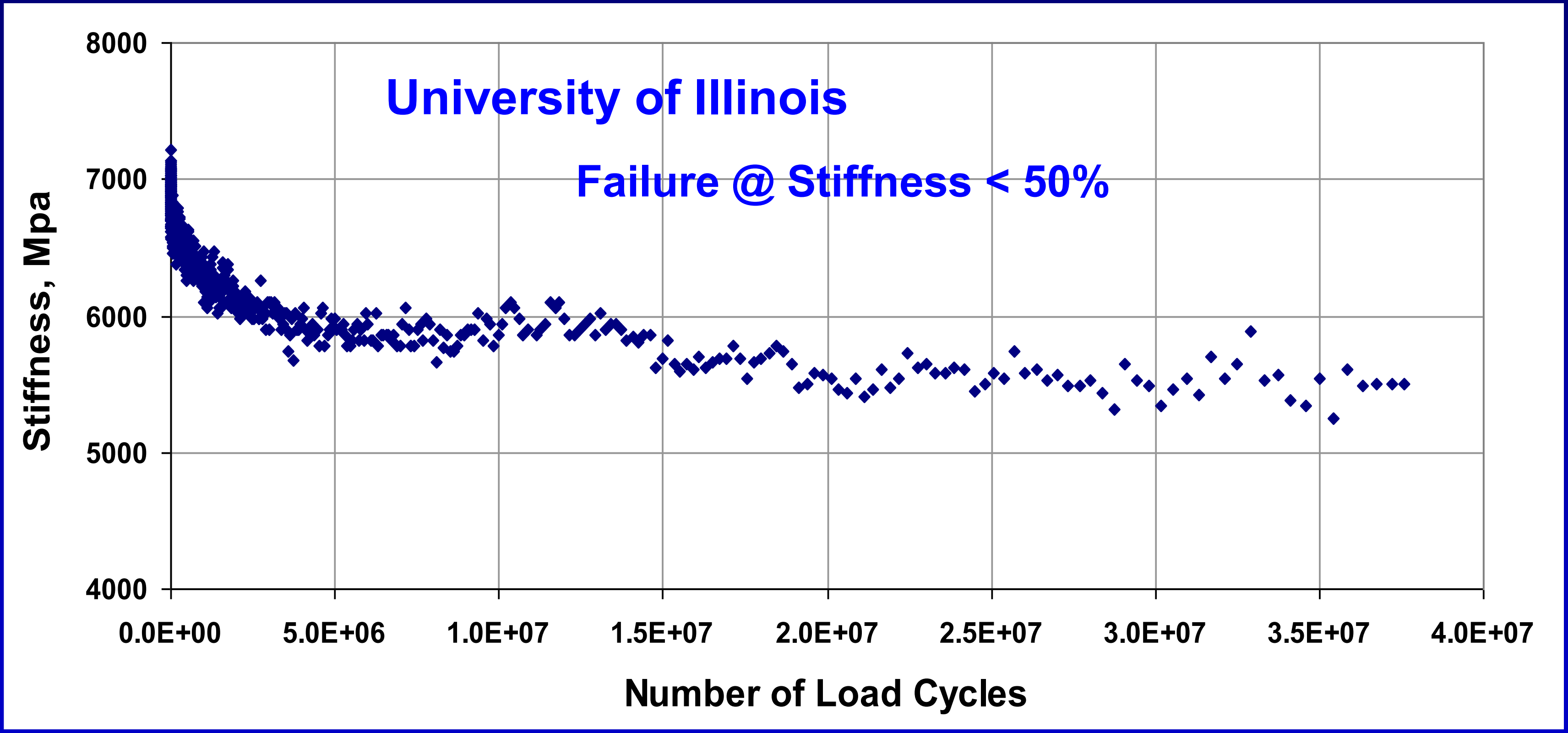
Monismith & McLean

“Technology of Thick Lift Construction: Structural Design Considerations”

1972 AAPT Proceedings

70 Micro-Strain Endurance Limit!!

70 Micro Strain Test



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.

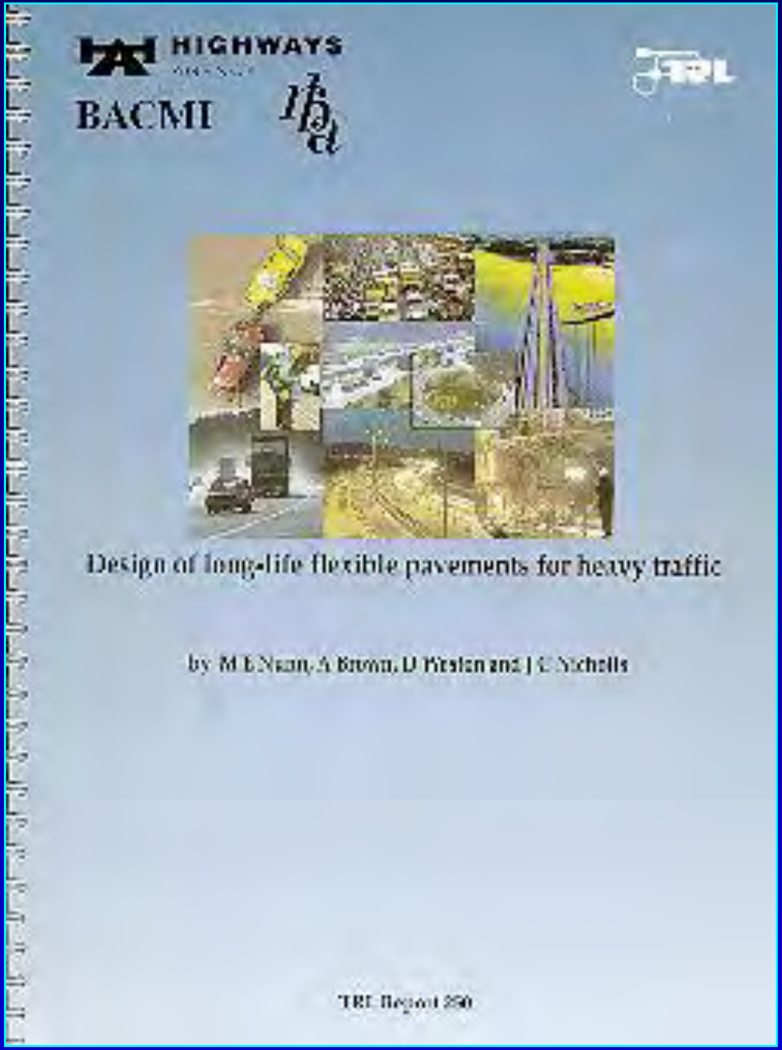
- Carpenter – UofI
  21 HMAs / Range: 90 – 300 με/ AVG: 125

Michael Nunn
“Long-Life Flexible Pavements”
8th ISAP Conference
Seattle, WA - 1997

ASPHALT PAVEMENT ALLIANCE
(2000)

“PERPETUAL PAVEMENTS”

Huddleston – Buncher – Newcomb



HIGHWAYS

BACMI

TRL

Design of long-life flexible pavements for heavy traffic

by M E Nunn, A Brown, D Weston and J C Nicholls

TRL Report 250

TRL Report 250
Nunn, Brown, Weston
& Nicholls

Design of Long-Life Flexible Pavements for Heavy Traffic

http:\\www.trl.co.uk

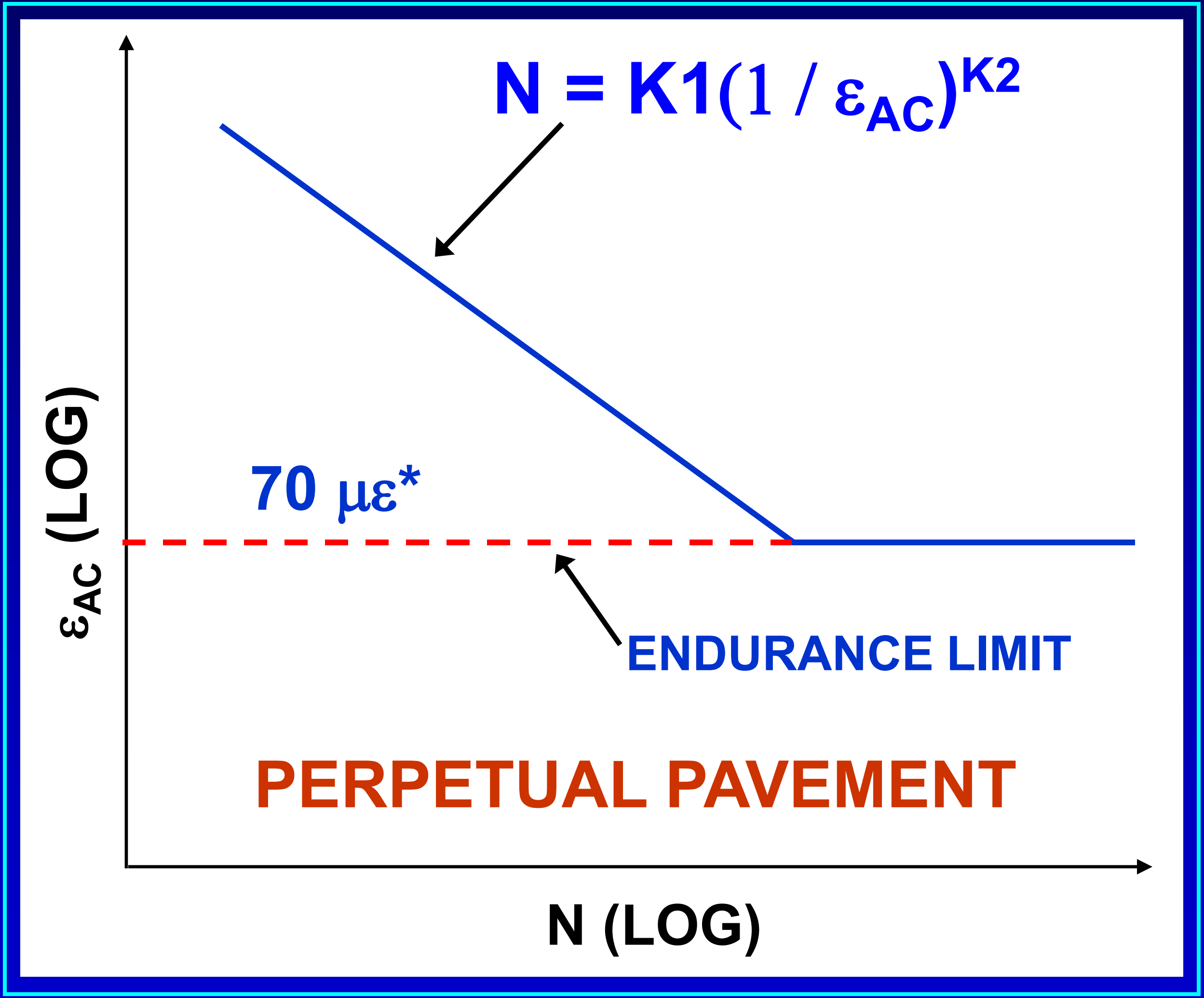
“Design Principles for Long Lasting HMA Pavements”

Thompson & Carpenter

ISAP Symposium
Design & Construction of
Long Lasting Asphalt Pavements

Auburn, AL
June -2004

HMA FATIGUE



*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)

NCHRP 9-44A
Validating an Endurance Limit for HMA Pavements: Laboratory Experiment and Algorithm Development
(ASU –WITCZAK/MAMLOUK – et al)
NCHRP REPORT - 762

* 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)

HMA MODULUS (ksi)	FEL: µ-STRAIN (RP – 1 SEC.)	FEL: µ-STRAIN (RP – 5 SEC.
300	46	122
600	37	102
1000	31	89
1500	27	80
2000	25	75
3000	21	66

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
(σ – ε – Δ)

↓

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

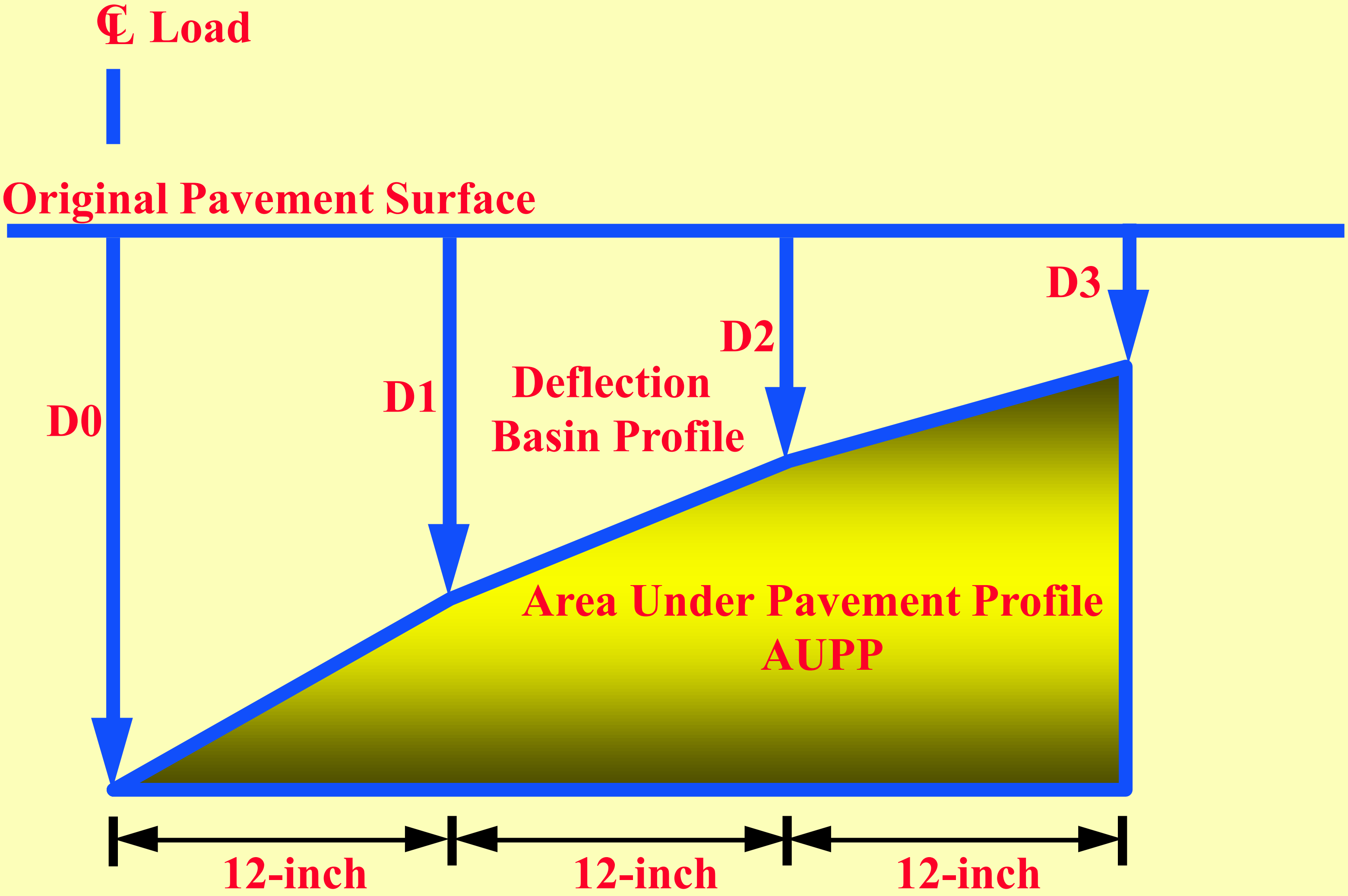
VARIABILITY IN Δ AND "BASIN SHAPE PARAMETERS"

SOME SIGNIFICANT PARAMETERS

Δ_0

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

AUPP (AREA UNDER PAVEMENT PROFILE)



AUPP = (5*D0 - 2*D1 - 2*D2 - D3) / 2

All Ds in mils

IDOT-FULL-DEPTH HMA

$$\text{LOG } \varepsilon_{\text{HMA}} = 1.53 \text{ LOG } \Delta_0 + 0.319$$

$$\text{LOG SSR} = 1.28 \text{ LOG } \Delta_0 - 2.21$$

$$(\text{SSR} = \text{SUB DEV } \sigma / Q_U)$$

$$\text{LOG } \varepsilon_{\text{HMA}} = 1.001 + 1.024 \text{ LOG (AUPP)}$$

Δ_0 : mils

ε_{HMA} : micro-strain

IDOT-CONVENTIONAL FLEXIBLE PAVTS

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

$$\text{LOG SSR} = 1.67 \text{ LOG } \Delta_0 - 2.88$$

$$(\text{SSR} = \text{SUB DEV } \sigma / Q_U)$$

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

Δ_0 : mils

ε_{HMA} : 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 MOELS

- \+ 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 !!!!

JIM BROWN
PAVEMENTS ENGINEER
TX DOT
CHAIRMAN – AASHTO JOINT TASK FORCE ON PAVEMENTS

Proceedings - Workshop on Resilient Modulus Testing
Oregon State University - March, 1989

“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!”