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Development of Drainage Coefficients and Loss of Support Values for Pavement Design in Nebraska

A Cooperative Research Project

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16. Abstract

A chart of drainage time to achieve 50 percent saturation for bases and subbases with edge drains was developed. Using this chart recommended values for drainage coefficients for portland cement concrete (PCC) and asphalt cement (AC) pavements can be determined from the 1993 American Association of State Highway and Transportation Officials (AASHTO) Design Guide.

At the sites evaluated, pavement drainage in Nebraska is rated Poor to Very Poor. Therefore, C_d will range from 0.95 to 0.70 depending on topography of the right-of-way and climate. LS values of 1to1.5 are appropriate for design, unless highly permeable non-erodable subbases are designed so that pavement drainage can be rated Good.

A computer model which incorporates the AASHTO 1993 Design equation for PCC concrete pavement is presented in a spreadsheet format that provides ease of design for evaluation of alternate criteria and material properties is presented. Two design examples representing conditions at one of the test sites are presented. The examples assume poor and good drainage for design assumptions comparison.

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EXECUTIVE SUMMARY

Three objectives were established for the research:

OBJECTIVE 1 - To develop a set of design charts for use with American Association of State Highway and Transportation Officials (AASHTO) rigid pavement design to determine drainage coefficients and loss of support for Nebraska soils and bases.

OBJECTIVE 2 - To develop drainage coefficients for Nebraska soils and bases for modification of the layer coefficients and environmental factors for use in AASHTO flexible pavement design.

OBJECTIVE 3 - To develop a set of standard design examples using Nebraska materials and construction practices.

These objectives were achieved with the following results:

OBJECTIVE 1 and OBJECTIVE 2

A chart of drainage time to achieve 50 percent saturation for bases and subbases with edge drains was developed. Using this chart recommended values for drainage coefficients for portland cement concrete (PCC) and asphalt cement (AC) pavements can be determined from the 1993 AASHTO Design Guide. Without edge drains it was shown that current drainage of Nebraska pavements must be considered as rated Poor to Very Poor according to the AASHTO design criteria. Edge drains and subbase materials having permeabilities exceeding 200 ft/day are required to provide good drainage. Loss of support (LS) values of 1to1.5 are appropriate for design, unless highly permeable non-erodable subbases are designed so that pavement drainage can be rated Good.

Selection of coefficient of drainage, C_d, and m_i for PCC and AC pavements, respectively, depends on drainage and the estimated percentage of the time the pavement materials are in a near saturated state. At the sites evaluated, pavement drainage is rated Poor to Very Poor. Therefore, C_d will range from 0.95 to 0.70 depending on topography of the right-of-way and climate. Locations in western Nebraska may reach 0.95 and those in eastern Nebraska may be as low as 0.70. Natural topographic surface drainage courses that transect the pavement may require special design to improve pavement drainage. This finding is considered to be typical of current designs.

OBJECTIVE 3

A computer model which incorporates the AASHTO 1993 Design equation for PCC concrete pavement is presented in a spreadsheet format that provides ease of design for evaluation of alternate criteria and material properties is presented. Two design examples representing conditions at one of the test sites are presented. The examples assume poor and good drainage for design assumptions comparison.

DISCLAIMER

The contents of this report reflect the views of the authors who are solely responsible for the findings and conclusions of the research. The contents do not necessarily reflect the official views or policies of the Nebraska Department of Roads or the University of Nebraska-Lincoln. This report does not constitute a standard, specification, or regulation.

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Chapter 1

INTRODUCTION

Background

The performance of highway pavement structures is dependent in part on the modulus of each material that makes up the pavement structural section. To account for changes in the modulus over time, the American Association of State Highway Transportation Officials (AASHTO) design procedures utilize modified structural layer coefficients for flexible pavements and measures of relative drainage and loss of support for rigid pavements. The designer is required to input a modified structural number or a drainage parameter based upon permeability, site geometry, as well as a loss of support factor for portland cement concrete (PCC) pavement design.

The fundamental quantities for these inputs were derived from American Association of State Highway Officials (AASHO) Road Test *(10,11)* in Illinois, but to date, there has been little comprehensive work to assess these values. Since AASHTO design procedures are sensitive to drainage and loss of support, inputs from improved values could lead to substantial savings in terms of lower initial costs through less conservative design or improved life cycle costs due to more appropriate initial construction and reduced maintenance.

Research Objectives

Three objectives were established for the research:

1. To develop a set of design charts for use with AASHTO rigid pavement design to determine drainage coefficients and loss of support for Nebraska soils and bases.

2. To develop drainage coefficients for Nebraska soils and bases for modification of the layer coefficients and environmental factors for use in AASHTO flexible pavement design.

2. To develop a set of standard design examples using Nebraska materials and construction practices.

The final status of each research objective is as follows:

OBJECTIVE 1

The development of design charts for the AASHTO design for selecting drainage coefficients C_d, for rigid pavement and layer coefficients, m_i, for flexible pavement design is presented in Chapter 6. An evaluation of the present state of drainage of Nebraska's pavements is also presented. Loss of support guidelines are presented in Chapters 5 and 7.

OBJECTIVE 2

The development of drainage coefficients, m_i, for AASHTO design of flexible pavement used to modify layer coefficients is presented in Chapter 6.

OBJECTIVE 3

A spreadsheet numerical model for 1993 AASHTO rigid pavement design is given in Chapter 7. A design example for Nebraska 103 near Crete, NE, one of the test sections is given. The selection of drainage coefficients, loss of support, and k for the design is illustrated.

Research Tasks

Six research tasks were outlined at the beginning of the project. The final status of each task is given below.

1. *Identify subgrade soils from the Nebraska Department of Roads (NDOR) records where pavement failures by faulting have occurred resulting from loss of support through inadequate drainage and subsequent pumping.*

   Five PCC projects were identified by NDOR personnel which showed loss of support as evidenced by faulting. Falling weight deflectometer (FWD) deflections, shelby tube samples, and bag samples were provided by NDOR for evaluation.

2. *Extend the NDOR's existing resilient modulus database to include a greater range of moisture effects including saturation for Nebraska soils.*

   Examples and a methodology for extending current data base resilient modulus values are given in Chapter 3.

3. *Correlate lab resilient modulus testing with values obtained by back calculation using field falling weight deflectometer data.*

   Backcalculated resilient modulus from FWD deflections is correlated to lab resilient moduli from shelby tube samples recovered from the test sites and is presented Chapter 4.

4. *Use information from distressed pavements (i.e., soil type and drainage conditions) to "back calculate" loss of support and use this data to evaluate the results of laboratory dynamic modulus tests of the identified problem soils.*

   Subgrade modulus, k, was backcalculated at each site from FWD deflections and correlated to lab and FWD resilient modulus. Reliable axle load data in the form of 18k equivalent single-axle loads (ESALs) to date and present serviceability index (PSI) data is not available. Therefore, backcalculation of a reduced k and thence a loss of support from site data was not feasible. An alternate approach is presented in Chapter 4.

5. *Perform laboratory permeability testing of selected problem subgrade soils and base course materials including drainable bases and analyze geometry to develop flow models.*

In Chapter 6 are descriptions of the numerical flow model that was used to evaluate drainage conditions. The permeabilities used in the model are those of base course materials thought to be drainable by NDOR .

6. *Develop design charts for drainage and potential loss of support which reflect Nebraska soils, climate, and construction practices.*

A combination of charts, tables, and a computer model are presented in Chapters 6 and 7 for design of PCC pavement together with a design example.

Chapter 2

FAULTED PCC PAVEMENT SLABS

Location and Natural Drainage

Five PCC pavement sections were selected by the NDOR personnel for study. Each test site showed evidence of faulting. Table 2.1 lists each site location, its relationship to natural drainage courses in the area, and the depth to the water table at the site.

Highway No.	General Location	Reference Post and Lane	Natural Drainage of Topography and Water Table Depth
N-32	Petersburg East to N-45	Mile Marker 10 East bound	Natural drainage area No water table 29 ft.
N-33	Crete to Sprague	Mile marker 23 West bound	Near culvert Water table 29 ft.
N-58	Loup City to Arcadia	Mile Marker 48 North bound	Natural drainage area No water table at 29 ft.
US-81	North of Norfolk	Drainage culvert No. 159.7 South bound	Multiple drainage culverts No water table at 29 ft.
N-103	Wilber to Crete	Mile marker 35 South bound	Box culvert 03514 Major drainage ditch on East Water table 17-18 ft.

Table 2.1 Test Site Descriptions

It should be noted that faulting typically occurred at points where natural drainage courses transect the right-of-way.

Pavement Sections and Subgrade Soil Properties

Table 2.2 shows thicknesses of the pavement section. The subbase is of sand and probably provides little structural support. The pavement sections are all jointed plain concrete (JPC) and the joints have no dowel bars. Load transfer is by aggregate interlock only. None of the sites have tied shoulders.

Highway No.	Pavement Thickness (in)	Sand Subbase Thickness (in)	Liquid Limit	Plasticity Index	AASHTO Classification
N-32	6	4	22	6	A-4
N-33	6	4	41	21	A-7-6
N-58	6	4	30-38	6-13	A-4
US-81	8	4	38	18	A-6
N-103	6	4	40	20	A-6

Table 2.2 Pavement Section and Subgrade Properties

Chapter 3

EXTENDED RESILIENT MODULUS TESTING

Validation of Previous Resilient Data

Resilient modulus data for 14 Nebraska soils representing a wide range of soil types across the state was previously reported (*1*). Investigators have raised concerns about the reliability of M_r testing citing problems with transducer calibration, placement of load cells and linear variable displacement transducers (LVDTs) and test system compliance. These problems were addressed in the initial test series by calibration of the system deflection measurements using a 4.0 in. diameter x 8.0 in. steel blank. The load transducer was internal. However, in order to evaluate the quality of these M_r measurements, the integrated testing system was used to measure the stiffness of 4.0 in. x 8.0 in. cylindrical polyurethane test specimens of known stiffness (*2*, *3*, *4*).

Stiff materials (M_r > 40,000 psi) provide the greater challenge to measure M_r accurately. Test system compliance, if not properly accounted for, makes the sample deformation unreliable since a large percentage of the recorded deformation is actually in the testing system. A synthetic specimen (E=46,700 psi) was used to improve the test setup protocol until system measurements were approximately equal to the specimen's known stiffness. Measured values ranged from 45,700 to 48,200 psi. Soft soils are much less problematic since a much smaller percentage of the measured deformation is in the test system. The original protocol used to set up the testing equipment was reevaluated and found to be adequate. The M_r values reported for the 14 soils tested are reliable measurements, with the following caveat. **M_r values at a deviator stress of 3 psi or less are unreliable** because the load level is too low for reliable measurements. Testing at this deviator stress level produces a bilinear curve which has been reported in the literature, but is considered by us to be test error not material behavior. It should be noted that 2 psi represents a total load of only 12 to 13 pounds. Figure 3.1 shows this "bilinear behavior" for a medium plastic Peorian soil that was retested as described below.

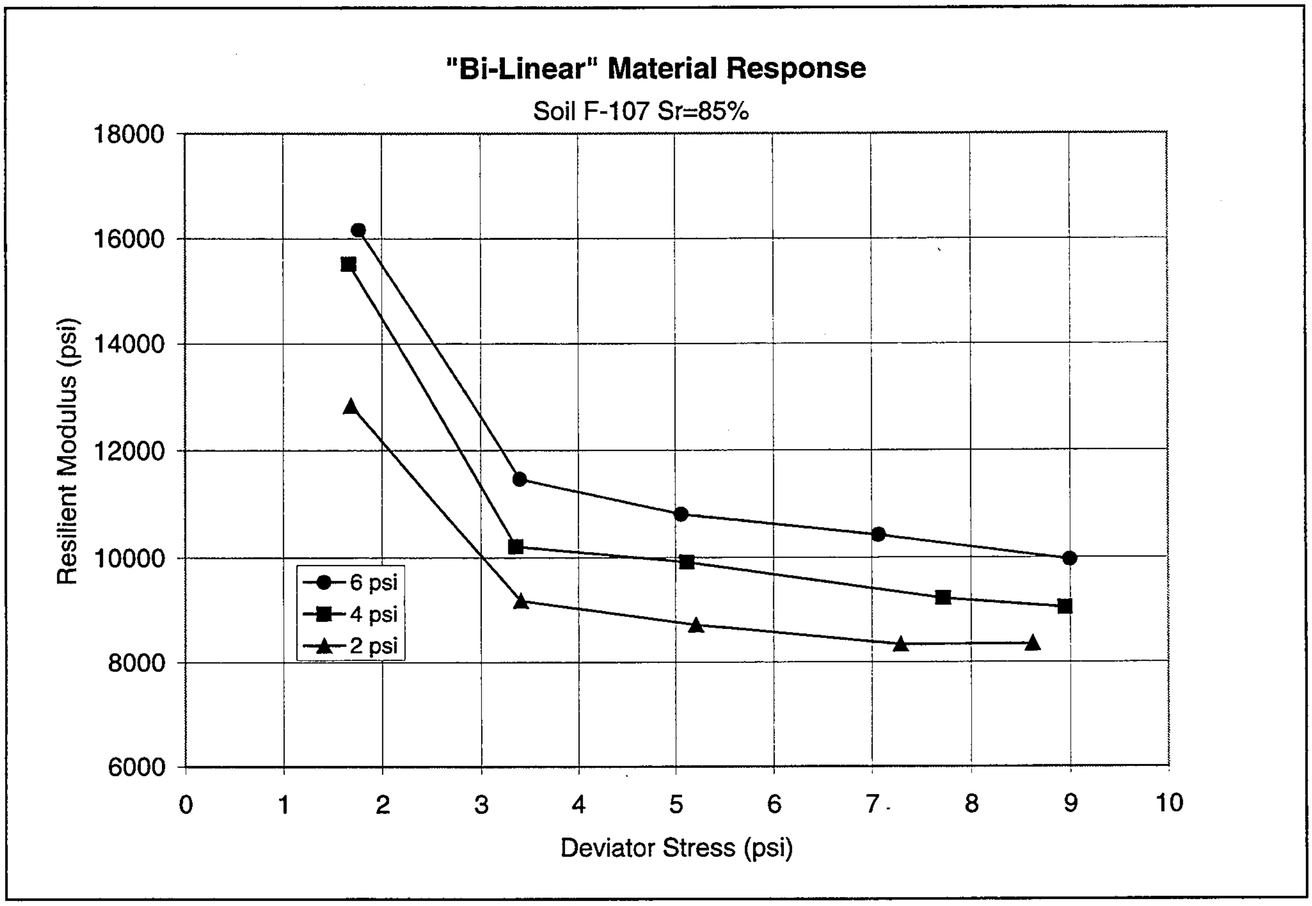

Figure 3.1 "Bi-Linear" Material Response of Soil F-107

The conclusion that the original testing was reliable was validated by testing a new soil sample, F-107. It was located at a site near that of S86-246, a medium-plastic Peorian loess, that was part of the original test series. Table 3.1 compares AASHTO classification and plasticity properties of these two samples. As the table indicates, they are very similar.

Sample No.	AASHTO Classification	Liquid Limit	Plasticity Index	Specific Gravity
S86-246	A-6(10)	36	15	2.63
F-107	A-6(10)	35	14	2.62

Table 3.1 Comparison of AASHTO Classification and Plasticity of Soils F-107 and S86-246

M_r for the new sample was determined according to AASHTO TP46, the provisional Strategic Highway Research Program (SHRP) standard. The test system setup used the improved protocol previously discussed. Two samples were tested with dry unit weights and original moisture states as shown in Table 3.2. Sample No. 2 was back pressure saturated after placement in the triaxial cell. M_r values for soil S86-246, using samples at different initial moisture states (dry of optimum, optimum, and wet of optimum) were combined with the F-107 test results. The S86-246 tests were performed in accordance with AASHTO T 294-86. These test methods have different confining pressures and test deviator stresses specified. Results for five samples, three S86-246 of the original test series (*1*) and two for F-107 were combined at common confining stress and deviator stress levels by interpolation of the data. These data are shown in Table 3.3 for a confining pressure of 3.0 psi.

Test No.	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Initial Saturation (%)	Final Saturation (%)
1	105.65	17.8	85.2	85.2
2	103.54	21.4	96.8	100

Table 3.2 Weight-Phase Final Saturation of F-107 Samples

Sample No.	Dry Unit Weight (pcf)	Final Saturation (%)	M_r @ Deviator Stress 4.0 psi (psi)	M_r @ Deviator Stress 8.0 psi (psi)	M_r @ Deviator Stress 10.0 psi (psi)
S86-246	105.6	80.2	13500	11500	10000
S86-246	107.6	89.6	20000	14000	12500
S86-246	108.1	95.4	7300	5000	4200
F-107	105.7	85.2	10000	9200	7900
F-107	103.5	approx.100	3200	2850	2800

Table 3.3 Extended Saturation Tests

The data trend is linear and shows that the testing in the original series is consistent with the new data. The original data is, therefore, reliable as previously stated. The data for the S86-246 sample at S_r= 89.6 percent falls off the trend line and was suspect in the original report.

New Near Saturation Resilient Modulus Data

Figure 3.2 provides near saturation M_r data for soil S86-246. A linear relationship shown for M_r as a function of degree of saturation was used in the AASHTO classification-M_r against S_r plot shown by Darter, Hall, and Kuo (*6*) in Appendix F. This technique can be utilized to extrapolate test results for other soils in the original test series.

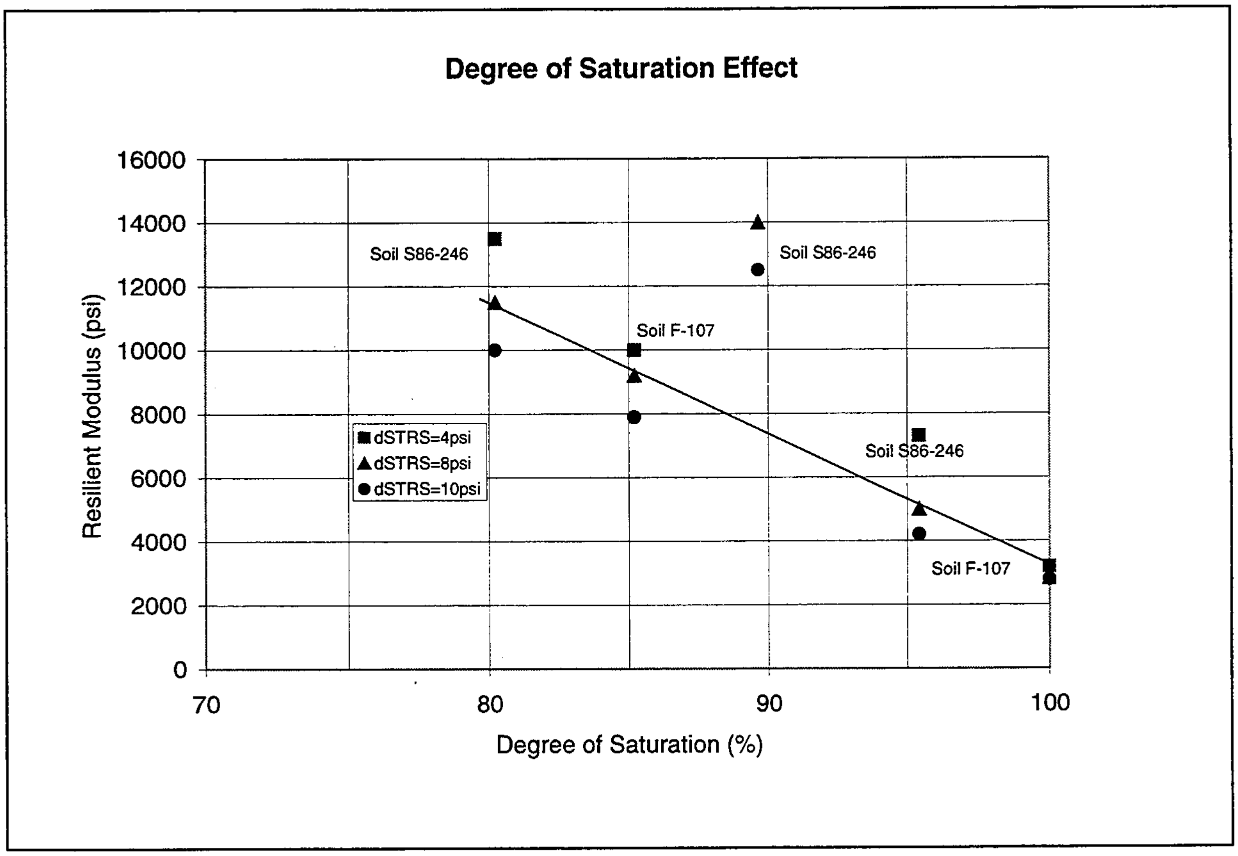

Figure 3.2 Saturation Extrapolation for M_r for Soil S86-246 a Plastic Soil

Using S_r for extrapolation combines both void ratio (dry unit weight) and water content. It is illustrated in Figure 3.3 for soil S86-174 a Tertiary soil which is a non-plastic silt classified A-4 (3.8).

The simple extrapolation correctly infers very low or unknown stiffness when saturated. The actual response may involve negative pore pressures caused by undrained dilation during deformation. This behavior is illustrated by the non-plastic Tertiary soil S86-335 in the original test series for both optimum and wet of optimum compaction. The degree of saturation for these samples was 79.5 and 87.2 percent. Interpretation of near saturation data for plastic soils is thought to be reliable, but its use for non-plastic silts is open to conjecture. If M_r =1500 psi is assigned to soil S86-174 at near saturation, based on judgement, the subgrade modulus, k, would only be 16 psi/in ($k=M_r/91$) as shown in Chapter 4. This is much less than 25, the recommended minimum value (*5*).

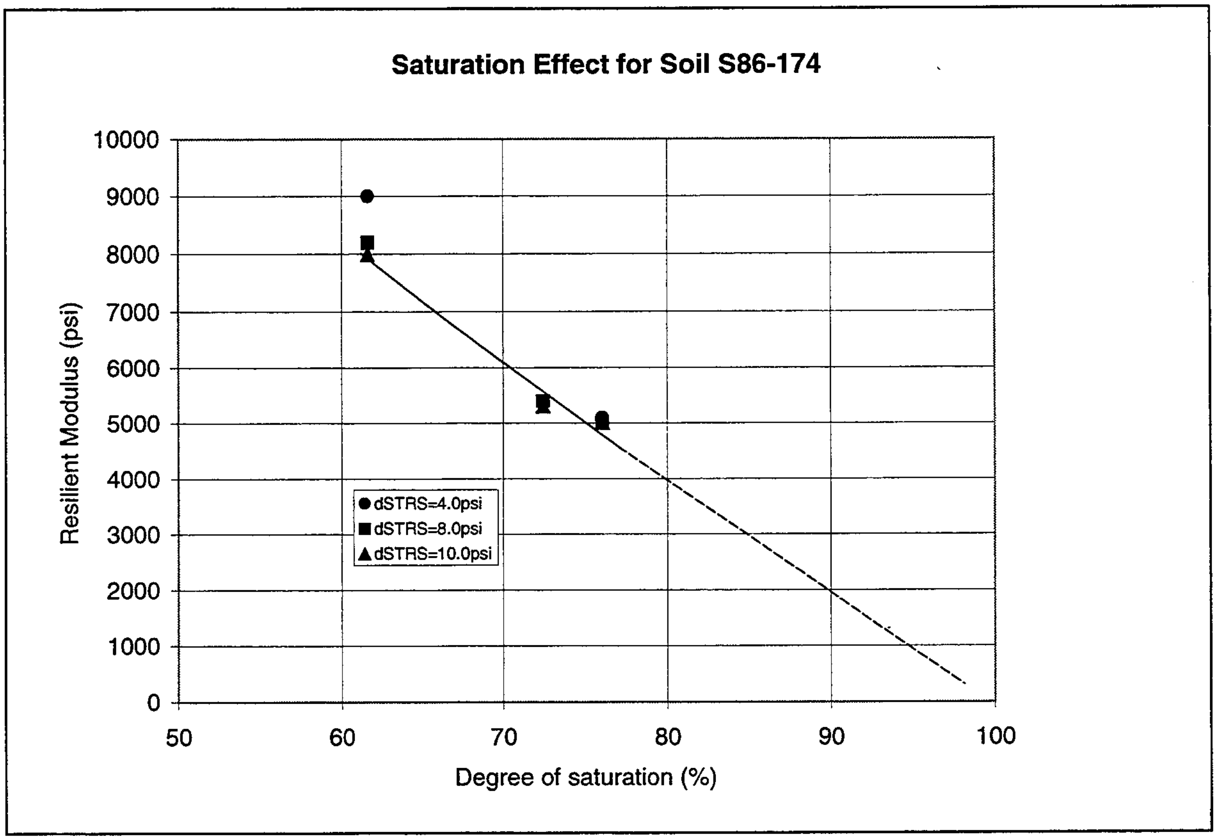

Figure 3.3 Saturation Extrapolation for M_r for Soil S86-174 a Non-plastic Silt

Chapter 4

CORRELATION OF FIELD AND LAB RESILIENT MODULUS AND SUBGRADE MODULUS DATA

Introduction

The design thickness of PCC pavement requires the coefficient of subgrade reaction, k. It is the primary subgrade mechanistic characterization. Darter, Hall, and Kuo (*6*) have shown that this is the static elastic k from a plate loading test carried out directly on the compacted subgrade. Either a repetitive or non-repetitive plate load test in accordance with AASHTO T221 or AASHTO T 222 (ASTM 1195 and 1196) can be used to determine k. This test is very time consuming and expensive and, consequently, is used infrequently. Typically correlations to quantities measured using less expensive testing methods such as resilient modulus, M_r, California Bearing Ratio (CBR), AASHTO classification, and falling weight deflectometer, FWD, are used. The actual in situ value of k is strongly dependent on soil moisture under the pavement. Therefore, k is strongly effected by drainage, climate, and season.

In an effort to extend the usefulness of existent laboratory M_r data for Nebraska soils (*1*), M_r values for subgrade soils beneath five existing PCC pavements were determined. Samples were retrieved using shelby tubes and were tested in the laboratory to determine M_r. FWD deflections were measured at these sites to estimate M_r and to backcalculate k.

M_r Tests of Subgrade Samples

Shelby tube samples were retrieved from five PCC pavement test sites selected by the NDOR. These sites all showed evidence of faulting. Each pavement selected was JPC without dowel bars. The location of the test sections and other site-specific data are shown in Table 4.1 below. As was shown in Table 2.1 the test sites are in areas near drainage courses.

Highway No.	Test Station	Sampling Date	FWD Test Date	Pavement Depth in.	W.T. Depth
N-32	531+42	11/15/94	12/1/94	6	> 29 ft
N-32	534+42	10/28/94	12/1/94	6	> 29 ft
N-33	624+62	12/1/94	11/30/94	8	19.1 ft.
N-33	624+62	11/14/94	11/30/94	8	18.6 ft.
N-58	130+37	11/7/94	12/2/94	6	> 29 ft
US-81	130+37	10/28/94	12/1/94	8	> 29 ft
N-103	113+02	10/28/94	11/30/94	6?	> 29 ft

Table 4.1 Site Specific Test Section Data

Table 4.2 shows the AASHTO classification dry unit weight and degree of saturation of the tube samples taken at the five sites. Table 4.2 indicates that all of the samples tested were fine-grained, plastic materials. Samples taken at the same location but from different bore holes are shown to have the same plasticity although separate tests were not run.

Highway No.	Dry Unit Weight (pcf)	Water Content (%)	S_r (%)	Liquid Limit	Plasticity Index	AASHTO Classification
N-32	-	-	-	22	6	A-4
N-32	104.9	21.0	92	22	6	A-4
N-33	103.0	23.3	97	41	21	A-7-6
N-33	102.8	23.2	98	41	21	A-7-6
N-58	97.0	23.4	86	30-38	6-13	A-4
US-81	90.8	28.7	91	38	18	A-6
N-103	90.4	21.8	85	40	20	A-6
N-103	90.4	25.8	85	40	20	A-6

Table 4.2 Weight-Phase and Plasticity Data for Shelby Tube Samples at Each Site

All of the samples were tested in accordance with the AASHTO TP 46 test protocol. The testing system calibration was verified throughout this testing phase using polyurethane specimens of known elastic modulus and a rigorous test setup protocol as previously described. The average M_r values reported in Table 4.3 are for confining pressures of 2 to 4 psi and a deviator stresses of 5 to 6 psi. This table also shows k based on laboratory M_r and a correlation factor $C_{FWD} = 91$ calculated from FWD data taken at the sites as explained below.

Highway No.	S_r (%)	AASHTO Classification	Laboratory M_r (psi)	$k = M_r/C_{FWD}$ (psi/in)
N-32	-	A-4	3500	38
N-32	92	A-4	5000	55
N-33	97	A-7-6	6700	74
N-33	98	A-7-6	5700	63
N-58	86	A-4	2500	27
US-81	91	A-6	8000	88
N-103	85	A-6	8000	88
N-103	85	A-6	10000	110

Table 4.3 M_r and k Values ($C_{FWD} = 91$) for Shelby Tube Samples at Each Site

Backcalculation of M_r Using FWD Data

A second way to determine the M_r of the subgrade is by backcalculation using FWD deflections. The deflection measurements used should be far enough from the load source that the influence of the pavement and any intermediate layers is eliminated. Deflections at radial distances, r, of 48 and 60 in. were used in this study. The backcalculated M_r (*7*), measured in psi, is computed from the following equation:

$$M_r = \frac{0.24P}{d_r r}$$

Equation 4.1

Where:

P = applied load in pounds
d_r = measured deflection at the distance r
r = radial distance at which the deflection is measured in inches

Comparisons of M_r values which were backcalculated at the AASHO Road Test site to laboratory measured values indicate that backcalculated values should be adjusted by a multiplier, C. Recent comparisons (*8*) vary the value of C depending on pavement material type. The FWD M_r adjusted for comparison to laboratory measured values is given by:

$$M_r = C\frac{0.24P}{d_r r}$$

Equation 4.2

Where:

C = 1.32 for subgrade soils below a stabilized subgrade
C = 0.52 for subgrade soils without an unbound granular base
C = 0.35 for subgrade soils below an unbound granular base.

The pavement sections at the test sites had a 4-in thick course of sand on the top of the subgrade, and therefore, do not fit any of the above conditions exactly. Figure 4.1 compares adjusted backcalculated M_r values at the five test sites with laboratory measured values from the shelby tube samples recovered at the sites. A value of C = 0.49 gives the best equivalency for FWD and laboratory measured values. This is very close to the C = 0.52 for a pavement directly on the subgrade. This is quite consistent with the 4-in sand layer used beneath the pavement sections at the sites. The C value for an unbound granular base, C = 0.35, fits very poorly.

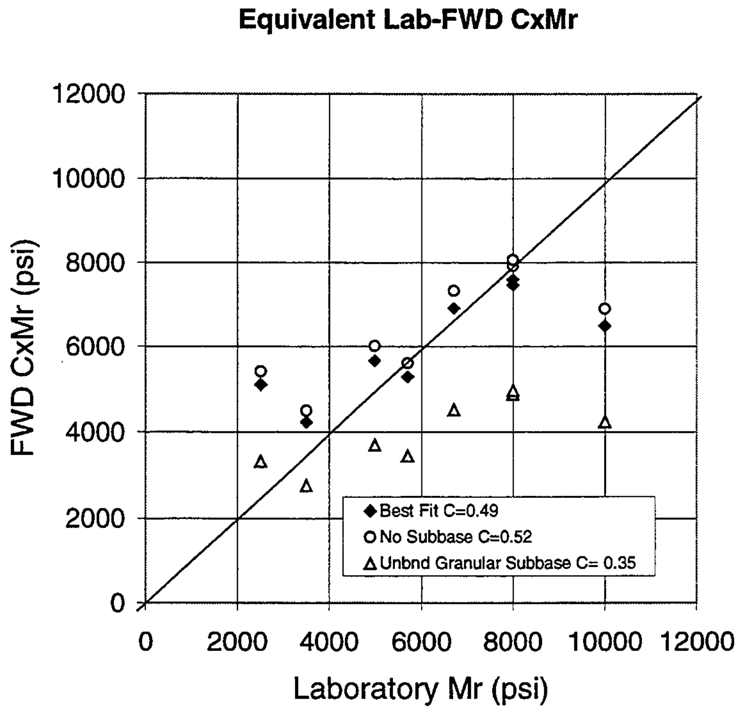

Figure 4.1 Comparison of Laboratory and Backcalculated M_r Values

Hall et.al. (*5*) provide several equations for calculating a parameter of the deflection basin called AREA for interpreting deflection basins. It is not the area of the deflection basin, but is a normalized length. Several AREA equations for backcalculating k from the deflection basin using FWD for various placements of the sensors are presented. The NDOR sensor placements were at –12 in., 0 in., 12 in., 18 in., 24 in., 36 in., 60 in. from the load point which permits the use of the AREA equation designated A5 by Hall et.al. (*5*). The A5 equation is for sensor placements at
18 in., 24 in. , 36 in. , 60 in. AREA for this placement is given by :

$$AREA = 3 + 6\left(\frac{d_{18}}{d_{12}}\right) + 9\left(\frac{d_{24}}{d_{12}}\right) + 18\left(\frac{d_{36}}{d_{12}}\right) + 12\left(\frac{d_{60}}{d_{12}}\right)$$

Equation 4.3

ℓ , the radius of relative stiffness, can be calculated by substituting AREA calculated from Equation 4.3 using the following equation

$$\ell = \left[\frac{\ln\left(\frac{48 - AREA}{158.4}\right)}{-0.476} \right]^{2.220}$$

Equation 4.4

The subgrade modulus can then be calculated from the following equation:

$$k = \frac{P d_r^*}{d_r \ell^2}$$

Equation 4.5

Where:

d_r = sensor deflection at 12 in
d_r* is given by the following equation
P = applied load in pounds

$$d_r^* = 0.12188\, e^{\left[-0.79432\, e^{(0.07074\ell)}\right]}$$

Equation 4.6

The value of k calculated using Equation 4.5 is based on a slab of infinite size; therefore, this k should be corrected for a slab of finite size (5) consistent with the slabs tested. The equation for doing this is given below.

$$k = \frac{k_{est}}{\left(AF_{\ell_{est}}\right)^2 AF_{d_0}}$$

Equation 4.7

Where:

$$AF_{d_0} = 1 - 1.15085\, e^{-0.71878\left(\frac{L}{\ell_{est}}\right)^{0.80151}}$$

$$AF_{\ell} = 1 - 0.89434\, e^{-0.61662\left(\frac{L}{\ell_{est}}\right)^{1.04831}}$$

$$L = \sqrt{L_1 L_2}$$

Highway No.	AASHTO Classification	Laboratory M_r (psi)	$k = M_r/C_{FWD}$ (psi/in)	$k = k_{FWD}/2$ (psi/in)	$k = k_{class}$ (psi/in)
N-32	A-4	3500	38	47	58
N-32	A-4	5000	55	62	58
N-33	A-7-6	6700	74	76	85
N-33	A-7-6	5700	63	64	85
N-58	A-4	2500	27	58	66
US-81	A-6	8000	88	83	64
N-103	A-6	8000	88	83	83
N-103	A-6	10000	110	72	83

Table 4.4 Comparison of k Values: Lab M_r, AREA, and AASHTO Soil Classification

It has been found (*6*) that backcalculated values of k using FWD deflections exceed the values of static, elastic k values from a plate loading test carried out directly on the subgrade by a factor of 2.

The average ratio of M_r, calculated from Equation 4.1, to k_{FWD} from FWD deflections at each site is 91. The range is 85 to 93. The AASHTO Guide (*7 and 9*) suggests a value of 19.4 . Darter et.al. (*6*) show in their Figure A.22 a comparison between backcalculated k and k determined from Equation 4.1. Their k values differ by a ratio of 4.32 to 4.84 or in terms of M_r , 84 to 94 which compares favorably with the value of 91 found at the sites studied.

Table 4.4 compares k values estimated from laboratory M_r , to k from FWD using Equations 4.3 and 4.5, and to correlations based on AASHTO soil classification. Generally, the agreement is reasonable. Values of backcalculated k using AREA5 from FWD measurements compare well to k calculated based on a k/M_r of 91for M_r measured on shelby tube samples tested in the laboratory. This conclusion suggests that laboratory M_r values from previous testing (*1*) can be used to find k if divided by 91 and not 19.4 as presently suggested by the 1993 AASHTO Guide (*7*). Correlations of k with AASHTO soil classification and estimated S_r of the subgrade (*6*) may be useful for preliminary design work for subgrades composed of fine-grained soil similar to those of the study area.

Figure 4.2 shows the equivalency of k values determined from laboratory M_r tests with those estimated from AREA using the FWD. Generally, k's from laboratory M_r are larger than those backcalculated from FWD.

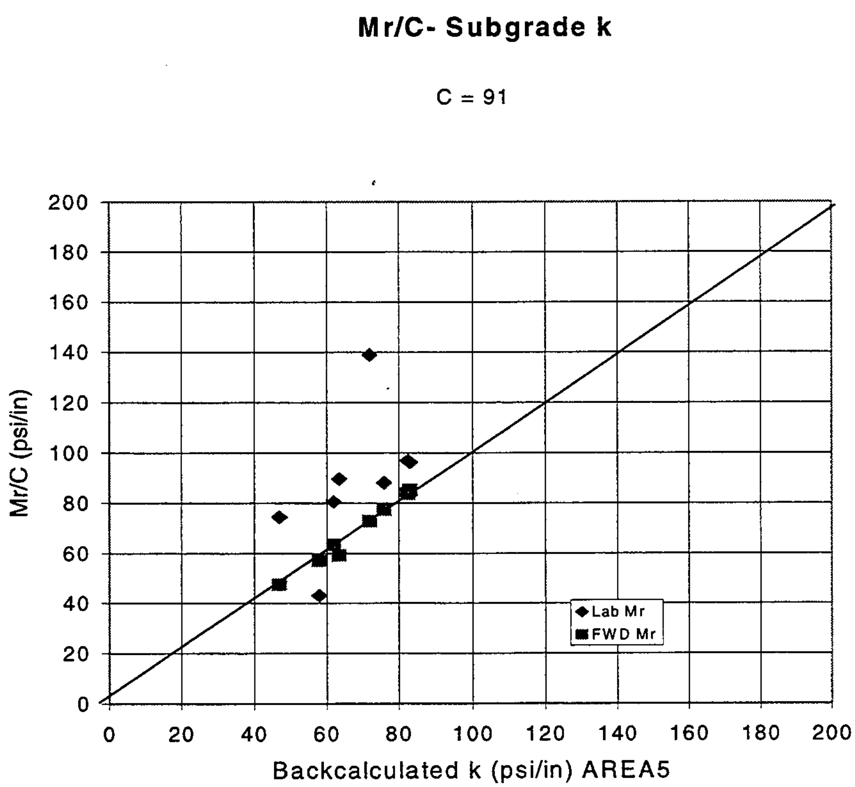

Figure 4.2 Equivalency of Backcalculated k from AREA, k from Shelby Tube Samples and from FWD Using $M_r/91$

Chapter 5

BACK CALCULATED LOSS OF SUPPORT AND DRAINAGE COEFFICIENT VALUES

Pavement Sections for Modeling Loss of Support

The PCC sections selected by the NDOR for this study had traffic volumes, axle load histories, pavement stiffness, and coefficients of subgrade reaction which are not known with any degree of confidence. Back calculating loss of support values (LS) and drainage coefficients, C_d, based upon the AASHTO 1993 Design Guide Volume 2 (*9*) without these data is at very best, problematic. The 1993 AASHTO rigid pavement design equation (*7*) is given here for reference in the discussion which follows.

$$\log_{10}(W_{18}) = Z_R + 7.35\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{(4.5-1.5)}\right]}{1+\frac{1.624\times10^{7}}{(D+1)^{8.46}}} + (4.22-0.32p_t)\log_{10}(\beta)$$

Equation 5.1

$$\beta = \frac{S_c' \times C_d \times \left(D^{0.75} - 1.132\right)}{215.63 \times J \left[D^{0.75} - \frac{18.42}{\left(E_c / k\right)^{0.25}} \right]}$$

Equation 5.2

WHERE:

W_{18} = predicted number of 18-kip equivalent single axle loads (ESALS)
Z_R = standard normal deviate
S_o = combined standard error of traffic prediction, performance prediction
ΔPSI = difference between initial design serviceability, p_0, and the design terminal serviceability, p_t
D = pavement thickness in in.
S_c' = modulus of rupture of the concrete in psi
J = load transfer coefficient (effect of dowels and integral curbs and shoulders)
C_d = drainage coefficient
E_c = modulus of elasticity of the portland cement concrete
k = effective modulus of subgrade reaction adjusted for subbase stiffness (see Figure 3.4 1993 AASHTO Guide); seasonal variation Figure 3.3 1993 AASHTO Guide); depth to rigid layer (see Figure 3.5 1993 AASHTO Guide); and the effect of loss of support, LS, from subgrade pumping (see Figure 3.6 1993 AASHTO Guide)

Equation 5.1 predicts the number of 18 kip ESALS that can be carried with a stated reliability for a given design pavement section thickness, D, modulus of subgrade reaction, *k*, (related to M_r), concrete stiffness, seasonally adjusted modulus of subgrade reaction modified for loss of support and pavement section drainage. Therefore, in order to back calculate LS and C_d values, the number of 18 kip ESALS, predicted by equation, must be made equal to the actual ESALS sustained by the pavement for assumed values of LS and C_d which are selected by trial. Since the actual number of ESALS sustained was not known for the Nebraska sections in this study this approach was not used and an alternate procedure was developed.

Interim design values of LS and C_d for Nebraska PCC pavement design can be estimated by using AASHO Road Test performance data which has well documented load histories, pavement sections, pavement material properties, and *k* values (*10*, *11*). Pavements in the AASHO Road Test study underwent substantial loss of support manifest by pumping but not by faulting at the transverse joints. This loss of support led to slab cracking and loss of serviceability. It should be noted that loss of support to reduce *k* values used in design, drainage coefficients, and *k* values seasonally adjusted for relative damage, first appeared in the design equation in the 1986 Guide. The AASHO Road Test data is not ideal.

Four points should be noted. The AASHO Road Test used a fixed 15 ft joint spacing, and all joints were doweled. The design equation does not consider slab curling caused by temperature differentials between the slab surface and the slab-base interface, and a composite k value which includes base effects is used. In the AASHO Road Test, the use of dowels prevented faulting, despite extensive pumping at many joints (loss of support). Additionally, the single subgrade soil at the AASHO site, the seasonal factors, and a two-year test length restrict the usefulness of these test performance data when extrapolated to Nebraska conditions. This restriction applies since the design equation is empirically based. The Nebraska pavements evaluated in the study are not doweled, the subgrade soils are different, and the climate and drainage conditions are different than those of the AASHO Test site. New design procedures, described in NCHRP Report 372, address some of the short comings of the present 1993 AASHTO Guide, namely, faulting and curling are considered explicitly and subgrade *k* and base stiffness are handled in a more theoretically consistent manner (*12*). However, incorporation of these procedures into practical pavement designs at NDOR requires substantial change in the AASHTO Guide which may take years. Therefore, even with these variances from an ideal set of performance data, the AASHO Test data is superior to data available for the Nebraska sections that were evaluated. It was used in this study to evaluate loss of support and drainage factors so that reasonable values can be estimated as guidelines for Nebraska rigid pavements.

Back Calculated LS and C_d Values

Modulus of Subgrade Reaction

The most recent data presently available for the seasonal modulus of subgrade reaction and subbase resilient modulus properties at the AASHO Test site (*6*) were selected. The k values, elastic measurements for the subgrade only for winter, spring, summer, and fall were 168, 77, 98, and 111 psi/in, respectively. The single design value of k for use in Equation 5.1, according to the 1993 AASHTO Design Guide, is based upon seasonally adjusted values of M_r (*7,9*). The 1993 AASHTO Design Guide relationship between k and M_r is :

$$k = \frac{M_r}{19.4}$$

Equation 5.3

Season	Subgrade M_r (psi)	Subbase M_r (psi)	Composite k (pci)	Season Length (months)
Winter	3259	25000	196	3
Spring thaw	1494	25000	103	1
Dry	2153	25000	139	6
Wet	1900	25000	125	2

Table 5.1 M_r , Composite k, and Season Length Utilized in Design Equation

As discussed with Figure 4.2 the ratio M_r / k, was found to be approximately 91 rather than 19.4 as given by Equation 5.3. This suggests that the denominator of Equation 5.3 is much too small and k values determined by it are seriously overestimated and should not be used.

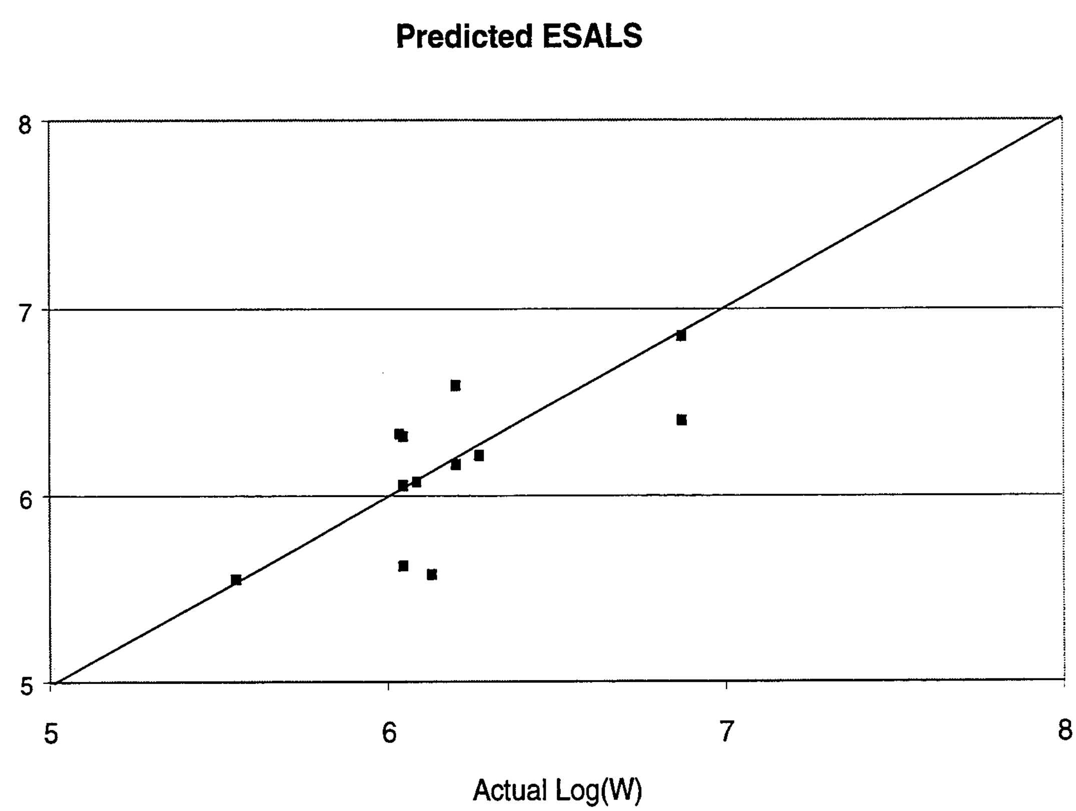

Figure 5.1 Road Test Measured and Calculated Log_{10} (W_{18}) at Final P_t . Calculated values of W_{18} are based Equation 5.1 and elastic k values measured at the test site which produce the M_r values shown in Table 5.1.

The composite k, for all computations at the AASHO site, was based upon the M_r values back calculated from Equation 5.3 to obtain site and season top of the subgrade k values. These M_r values and season lengths are given in Table 5.1. The effective modulus of subgrade reaction k, used in the design equation, was 145 pci using these assumed values for seasonal M_r. The reliability assumed was 50 percent. Table 5.2 shows the calculated and measured final $Log_{10}(W_{18})$ values using the pavement section depths and assumed values of LS = 1.1 and $C_d = 1.0$. The drainage at the AASHO Test site was assumed to be fair as stated in the Guide, so $C_d \sim 1.0$. No LS value was assigned to the AASHO Test but as previously noted, significant loss of support was reported. Values of LS and C_d were chosen which produced the best fit of the present serviceability index (PSI) trends as well as final values of calculated and measured $Log_{10}(W_{18})$ during the AASHO Road Test. Table 5.2 shows only the final values at the end of each test sequence using values of LS equal to 1.1 and a C_d equal to 1.0 for each loop. These values were found to fit the road test data quite well as shown in Table 5.2 and Figure 5.1.

Although Darter et. al. (*12*) illustrate several design deficiencies in the 1993 Guide, the predictions of the actual AASHO ESAL data are quite good using field measured elastic k values. They are a sound basis for establishing guidelines for selecting LS and C_d values for use in NDOR rigid pavement designs based upon the 1993 Guide as further discussed in the next section. The AASHO tests had significant loss of support but no faulting. Since LS = 1.1 fit the AASHO test sections, and the Nebraska PCC pavement test sections had little evidence of pumping, a value for LS of 0.9 to 1.1is suggested for such pavements (without large numbers of high axle loads). Larger values would be required for pavements having large numbers of high axle loads or high quality drainable subbases should be designed.

AASHO Test Loop Section	SA/DT Axle Load (kips)	Pavement D (in)	Subbase D_b (in)	P_t	Calculated $Log_{10}(W_{18})$	Measured $Log_{10}(W_{18})$
4/647 PC	18 SA	6	6.0	1.5	5.548301	5.546543
4/697 PC	18 SA	6.5	6.0	4.4	5.620988	6.046885
4/683 PC	18 SA	8.0	6.0	4.4	6.046409	6.046885
4/701 PC	18 SA	9.5	6.0	4.5	6.308807	6.046885
5/517 PC	22.4 SA	6.5	6.0	2.5	6.208964	6.274184
6/393 PC	30 SA	8.0	6.0	4.4	6.394679	6.874366
4/648 PC	32 DT	5.0	6.0	2.5	5.574605	6.131939
4/656 PC	32 DT	6.5	6.0	3.5	6.069535	6.087781
4/684 PC	32 DT	8.0	6.0	4.4	6.160338	6.205204
4/702 PC	32 DT	9.5	6.0	4.2	6.582982	6.205204
4/689 RC	18 SA	8.0	6.0	4.0	6.052689	6.046885
4/653 RC	18 SA	6.5	9.0	2.0	6.323822	6.037028
6/385 PC	30 SA	8.0	6.0	3.9	6.84837	6.873495

Table 5.2 AASHO Road Test Calculated and Measured $Log_{10}(W_{18})$

Chapter 6

DRAINAGE COEFFICIENTS FOR NEBRASKA SUBGRADE SOILS

Introduction

The estimation of drainage quality of a pavement section at different points along the right-of-way relative to local topography, drainage patterns, and subsurface conditions is a critical pavement design step. The selection of the coefficient of drainage, C_d, to be used in the AASHTO design equation, has a strong effect on the predicted ESALS. The 1993 AASHTO Guide provides a qualitative statement about drainage as shown in Table 6.1. The selection of C_d is also related to the percentage of the time the pavement section is in a state near saturation. This, in turn, encompasses climate, localized topographic features, and drainage patterns. Values of C_d from the 1993 AASHTO Guide are shown in Table 6.2. Drainage at the AASHO Test site was rated evaluated as Fair.

Quality of Drainage	Time Required for Water Removal (hrs)
Excellent	2 hours (2)
Good	1 day (24)
Fair	1 week (168)
Poor	1 month(720)
Very poor	Does not drain

Table 6.1 Quality of Drainage

Drainage Quality	Percent of Time ~Saturated Less Than 1%	Percent of Time ~Saturated 1 -5 %	Percent of Time ~Saturated 5 -25 %	Percent of Time ~Saturated Greater Than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.0
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
V. Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

Table 6.2 Recommended Values of Drainage Coefficients C_d for PCC Pavements

The AASHO section was estimated to drain to 50 percent saturation in one week. The1993 AASHTO Guide defines the time required to achieve 50 percent average saturation in the subbase as T_{50}. It is an average based upon the spatial variation of saturation vertically and horizontally through the subbase. The time, T_{50}, depends upon spatial distances and material properties. The spatial distances are length of the drainage path beneath the pavement section, L, the slope of the drainage path, s, the thickness of the base or subbase which is draining, d, the depth of the edge drain, w. The material properties are the permeability of the base or subbase that is draining, k, and Ψ, the soil suction.

The selection of drainage coefficients, m, for flexible pavement design is a function of drainage quality determined from Table 6.1 and the percentage of the time the pavement section is in a state near saturation. This encompasses climate, localized topographic features, and drainage patterns. Values of m_i from the 1993 AASHTO Guide are shown in Table 6.3 .

Drainage Quality	Percent of Time ~Saturated Less Than 1%	Percent of Time ~Saturated 1 -5 %	Percent of Time ~Saturated 5 -25 %	Percent of Time ~Saturated Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.0
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
V. Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 6.3 Recommended Values of Drainage Coefficients m_i for Flexible Pavements

Numerical Modeling

McEnroe (*13*) developed a numerical procedure for solving the non-linear partial differential equation to estimate drainage times to achieve T_{50} . The procedure is based upon Brooks and Corey's formula (*14*) for water retention in a partially saturated granular base held by soil suction against gravity. McEnroe demonstrated that the method used in the Federal Highway Administration (FHWA) sub-drainage design manual tends to underestimate drainage times because it neglects the spatial variability of suction in the base.

Using McEnroe's procedure, a program was written to determine drainage times required to achieve T_{50} for a drainable base with an edge drain depth, w, below the base located at the edge of the pavement. Figure 6.1 shows a section of the drainage system. Figure 6.2 shows T_{50} times for the assumptions of w = 1.6 ft., drainage lengths of 12 and 24 ft., d = 6 in., and s between 0.02 and 0.03 for a range of permeabilities.

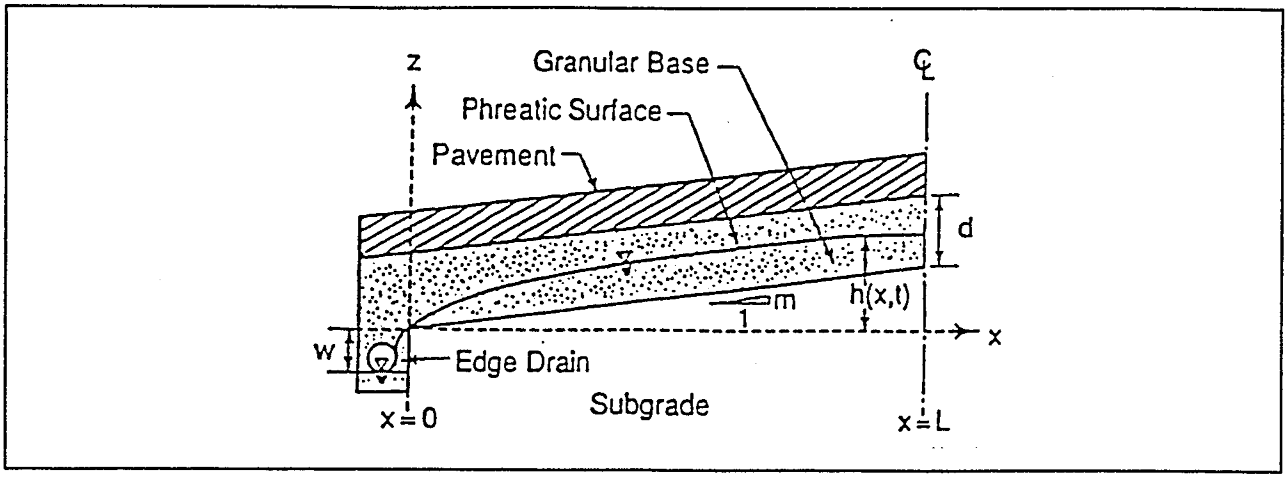

Figure 6.1 Drainable Pavement Section

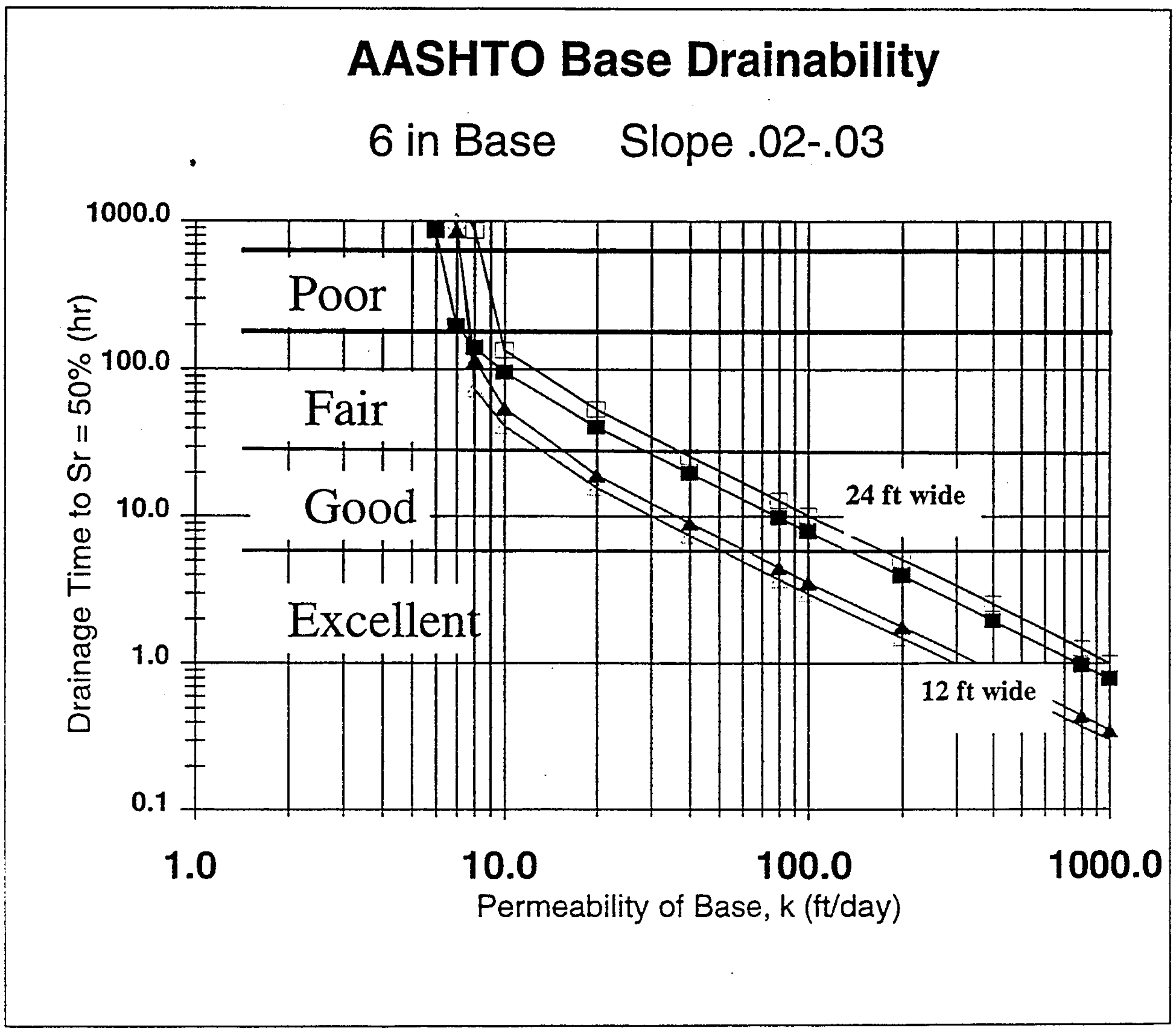

Figure 6.2 Drainage Time T_{50} for Two Typical Drained Pavement Sections

Figure 6.2 illustrates that field permeability exceeding 8 to 10 ft/day in the subbase is required for the drainage of the pavement section to be rated Fair if edge drains are present. The laboratory permeability of crushed Portland cement concrete and asphalt cement concrete samples taken from the NDOR stockpiles is approximately 10 to 20 ft/day as currently graded. Since laboratory permeabilities should be three times the field permeability, values of 3 to 7 ft/day should be used with Figure 6.2. None of the sections evaluated in this study had permeability even approaching this since the subbases were sands stabilized by fines. No edge drains were present at any site. The drainage at the test sites is rated Poor to Very Poor based on Figure 6.2.

Assuming a saturated state from 5 to 25 percent of the time for right-of-way on an upland site $C_d \leq 0.80$ ($m_I \leq 0.8$) seems to be a reasonable guide. For a saturated state ≥ 25 percent of the time, such as for right-of-way on a site located at a natural drainage course, $0.80 \leq C_d \leq 0.70$ ($0.6 \leq m_I \leq 0.4$) seems to be a reasonable guide. Faulting was observed at all sites located at a natural drainage course. It is completely unreasonable to assume optimistically higher values of C_d at these locations without having drainage rated Good as evidenced by edge drains and a subbase permeability in the range of 200-500 ft/day (see Figure 6.3). Figure 6.3 shows the phreatic line drawdown in a pavement section rated Good drainage.

It must be noted that pavement drainage that is rated Fair **cannot** be achieved with the current gradation of crushed pavement materials yielding the laboratory permeabilities measured even with edge drains as shown by Figure 6.2. Note, the permeabilities in Figure 6.2 are field permeabilities. The gradation must have no fines to achieve the field permeability range rated Good. Stated differently, subbases stabilized with fines will not provide sufficient permeability even with edge drains. Removing fines may require stabilizing the material with portland cement, asphalt, or self-cementing fly ash to produce a stable construction platform with high permeability (greater than 200 ft/day). The additional costs during construction must be balanced against the cost of a thickened pavement section and increased maintenance over time. Use of the NDOR crushed asphalt concrete subbase (k=12.5 ft/day) with edge drains and the resultant drainage is shown in Figure 6.4. Figure 6.4 shows the average vertical S_r as well as the horizontal spatial variation of S_r after 864 hours indicating drainage rated Poor.

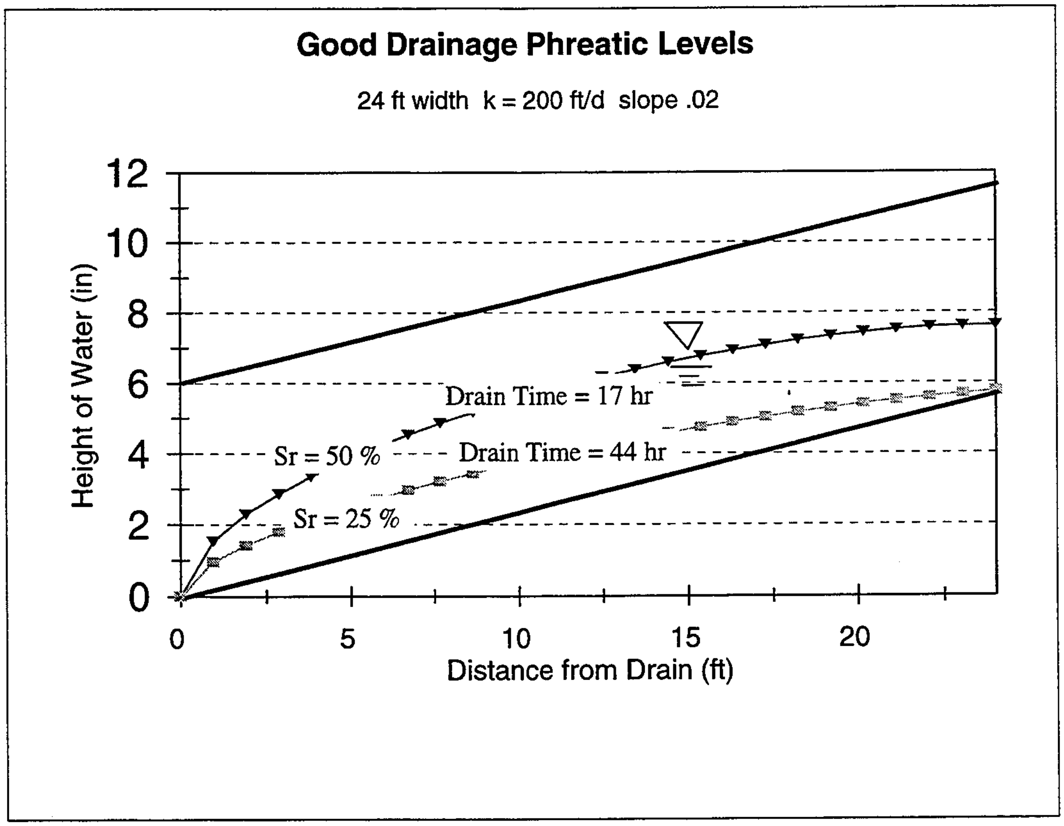

Figure 6.3 Phreatic Line Drawdown in Pavement Section with Good Drainage

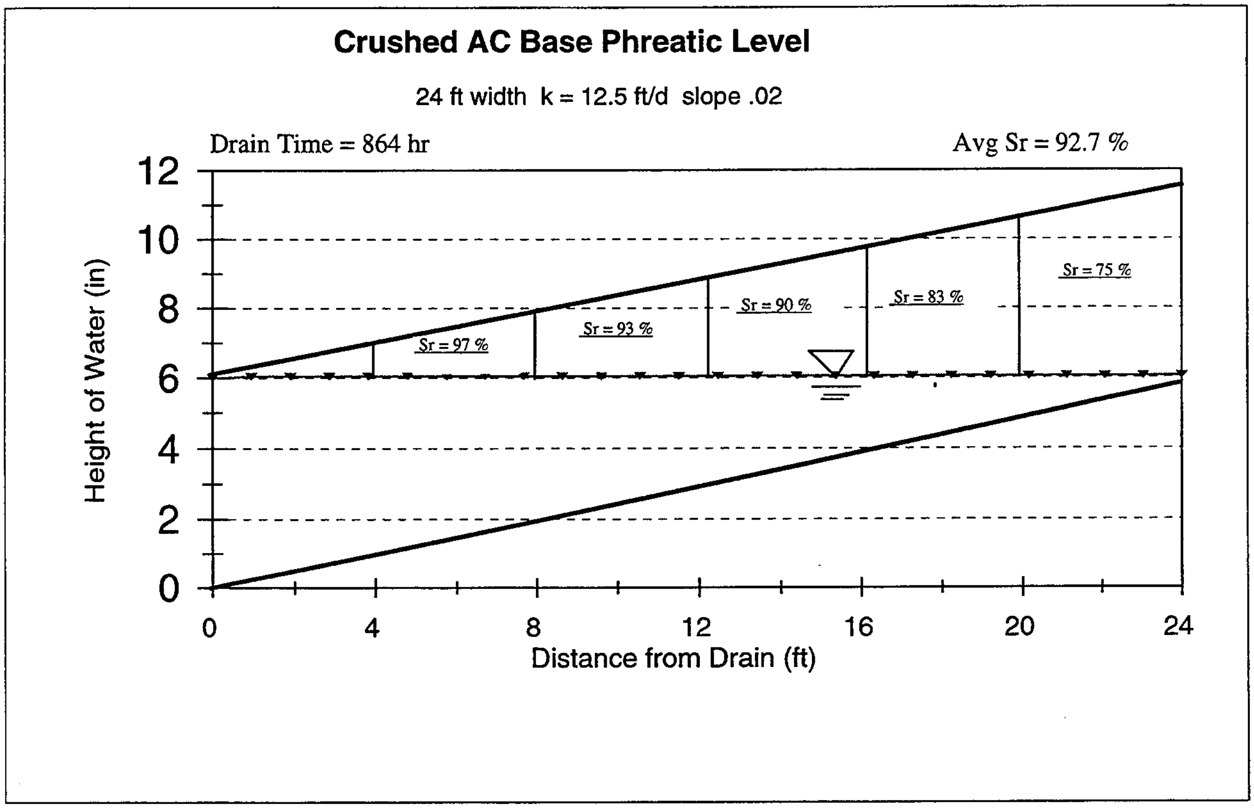

Figure 6.4 Poor Drainage of a Crushed AC Subbase

Chapter 7

NEBRASKA IMPLEMENTATION OF AASHTO 1993 DESIGN

LS and C_d Values

Figure 5.1 shows that the AASHTO 1993 design equation for rigid pavement design (Equations 5.1 and 5.2) predicted the actual W_{18} applied at the AASHO site well for a drainage coefficient, C_d, of 1.0 (Fair drainage) and an assumed loss of support LS =1.1. **NOTE: Seasonal, site-specific, static elastic k values were used in the computations.** The AASHO pavement suffered significant loss of support but did not fault because of dowels at the joints. Loss of support relates to erodability of the subbase and subgrade. Erodability depends on pavement deflection (wheel load), drainage, and susceptibility of the pavement materials to erosion. Keeping these concepts in mind, the backcalculation results provide guidance to selection of LS and C_d values for Nebraska.

Drainage of Nebraska pavements must be considered as rated Poor to Very Poor according to the AASHTO Design criteria unless edge drains and subbase materials having permeabilities exceeding 200 ft/day are present. LS values of 1to1.5 are appropriate for design, unless highly permeable non-erodable subbases are designed so that pavement drainage can be rated Good. C_d selection depends on drainage and the percentage of the time the pavement materials are in a near saturated state. Nebraska pavement drainage is rated Poor to Very Poor. Therefore, C_d will range from 0.95 to 0.70 depending on topography of the right-of-way and climate. Locations in western Nebraska may reach 0.95 and those in eastern Nebraska may be as low as 0.70. Natural topographic surface drainage courses that transect the pavement may require special design to improve pavement drainage.

Spreadsheet Model for AASHTO 1993 Design

The design of a section N-103 at the site of the study illustrates the selection of seasonal M_r values from a chart extrapolated from the existing database as shown in Chapter 3. Seasonal values of degree of saturation beneath Nebraska pavements are not available and are selected by judgement. Both the present drainage rated Poor, and a design with drainage rated Good are shown. As an aid to the designer, seasons will be considered as frozen, spring thaw, dry, and wet placing emphasis on the moisture state in the pavement section rather than using winter, spring, summer, and fall. An Excel spreadsheet implementing the AASHTO 1993 rigid pavement design guidelines will be used rather than design charts. Relevant blocks of the spreadsheet will be shown to illustrate data selection and entry. **Only** shaded blocks can be changed by the user.

Example 1: Poor drainage, the present state.

STEP 1: *Select seasonal M_r values*

The soil at the site is very similar to soil S86-246 which is an A-6(10). The N-103 site is A-6(8). Therefore, the extended M_r data given by Figure 3.2 can be utilized. For other soils extended Degree of Saturation -M_r plots can be constructed as shown in Chapter 3. Using estimated S_r, seasonal M_r values from Figure 3.2 are shown in Table 7.1. The spreadsheet uses Equation 5.3 to compute k as given in the AASHTO Guide. The denominator of 19.4 should be 91 as shown in Chapter 4 (a factor of 4.7). Use of adjusted M_r in Table 7.1 produces k values computed by the spreadsheet that are consistent with plate loading static elastic k values for the subgrade.

Moisture Season	Length (months)	S_r (%)	Subgrade M_r (psi)	Adjusted M_r (psi)	Subbase M_r (psi)
Frozen	2.5	-	12000	2560	15000
Spring thaw	1.0	98	4000	850	15000
Dry	5.5	90	7500	2200	15000
Wet	3.0	85	9500	1600	15000

Table 7.1 Seasonal Resilient Moduli for Soil S86-246

These data are shown after entering them into a portion of the spreadsheet.

Seasons	Subgrade Mr	Subbase Esb	Composite Inf K & Esb k	Composite Rgd-K & Esb k	Subdivision months m	Damage ur
frozen	2560	15000	136	136	2.50	24
sprg thw	850	15000	52	52	1.00	35
dry	2020	15000	111	111	5.50	27
wet	1600	15000	90	90	3.00	29
				Sum m =	12.00	
					avg ur	27

Figure 7.1 M_r Spreadsheet

STEP 2: *Select other pavement data*

Other miscellaneous inputs are shown in Figure 7.2. It should be noted that since this pavement has no dowel bars, J = 3.8 to 4.4 for plain jointed PCC. The shoulders are not tied to the pavement. The reliability is 85 percent (Zr=-1.037). A subbase of 4 in. was used.

Design Assumptions		
STD Dev	Zr =	-1.037
STD Err	So =	0.35
Eff Mod Sub R	k =	103
Rigid Layer Depth	Dsg =	40
Mod of Rupture	S'c =	560
E Mod of Concrete	Ec =	4200000
Depth Subbase	Dsb =	4
Initial PSI	po =	4.5
Terminal PSI	pt =	2.5
Envrnmtl PSI Loss	dPSI =	0.25
Load Transfer Coef	J =	3.8

Figure 7.2 Other Pavement Data Spreadsheet

STEP 3: *Drainage and loss of support*

As described above, an LS of 1.1 is reasonable. This input does not alter k. The user must use Figure II-3.6 of the Design Guide (*7*) for an effective modulus of subgrade reaction of 99 and an LS = 1.1 giving a loss of support effective k=37.

Estimated Pav D =	7.63	in
Loss of Support =	1.1	
Drn Coef Cd =	0.75	
Dsgn P (years)	20	
Daily ESALS 2-way	40	
Trffc Grwth i %	1	
% Trffc Dsgn Ln	100	
Loss of Support Effective k =	37	**Use Fig.II-3.6 - 93 Guic**

Figure 7.3 LS, C_d and Loss of Support Effective k Spreadsheet

Based on poor drainage, coupled with a location at a natural drainage course suggests a drainage coefficient of 0.75.

STEP 4: *Predicted ESALS*

Using load tables and traffic studies daily two-way ESALS, ESALS and rate of traffic growth are input (see Figure 7.3).

Predicted ESALS =	1.615E+05
D	Calc W18's
7.63	1.637E+05

Figure 7.4 Calculated = Predicted

STEP 5: *Change pavement depth until calculated W_{18} equals predicted*

These design assumptions require 8.0 in. (7.63, see Figure 7.4) of pavement rounding to the half in.

Example 1: Good drainage, reconstruction.

STEP 1: *Select seasonal M_r values*

The values for M_r are the same. Experience may allow lower degrees of S_r.

Estimated Pav D =	6.20	in
Loss of Support =	0.9	
Drn Coef Cd =	1.05	
Dsgn P (years)	20	
Daily ESALS 2-way	40	
Trffc Grwth i %	1	
% Trffc Dsgn Ln	100	
Loss of Support Effective k =	45	**Use Fig.II-3.6 - 93 Guide**

Figure 7.5 LS, C_d and Loss of Support Effective k Spreadsheet – Good Drainage

STEP 2: *Select other pavement data*

The same values will be used.

STEP 3: *Drainage and Loss of Support*

The loss of support was reduced to 0.9, and the new effective k becomes 45. The drainage coefficient is increased to 1.05 because of good drainage.

STEP 4: *Predicted ESALS*

This step remains the same.

STEP 5: *Change pavement depth until calculated W_{18} exceeds predicted*

Predicted ESALS =	1.615E+05
D	Calc W18's
6.20	1.667E+05

Figure 7.6 Calculated = Predicted ESALS

The required pavement depth has been reduced to 6.5 in. (6.20, see Figure 7.6). This is a reduced pavement thickness of 1.5 in of concrete or 19 percent through improved drainage. Additional maintenance cost reductions would be expected.

Complete Excel spreadsheets for each example are included for reference on pages 35 and 36.

AASHTO 1993 Rigid Pavement Design

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(Rev. 1.2 6/5/98)

3/18/99 14:07 Project Description: **N-103 Wilber to Crete at Test Site - Poor Drainage**

Design Assumptions

STD Dev	Zr =	-1.037	
STD Err	So =	0.35	
Eff Mod Sub R	k =	103	pci
Rigid Layer Depth	Dsg =	40	ft
Mod of Rupture	S'c =	560	psi
E Mod of Concrete	Ec =	4200000	psi
Depth Subbase	Dsb =	4	in
Initial PSI	po =	4.5	
Terminal PSI	pt =	2.5	
Envrnmtl PSI Loss	dPSI =	0.25	
Load Transfer Coef	J =	3.8	
	Estimated Pav D =	7.63	in
	Loss of Support =	1.1	
	Drn Coef Cd =	0.75	
	Dsgn P (years)	20	
	Daily ESALS 2-way	40	
	Trffc Grwth i %	1	
	% Trffc Dsgn Ln	100	
Loss of Support Effective k =		37	**Use Fig.II-3.6 - 93 Guide**
	Predicted ESALS =	1.615E+05	

D	Calc W18's	Log (W18)	Term 1	Term 2	Term 3
7.63	1.637E+05	5.21418	6.51673	-0.25575	-1.0468

Seasons	Subgrade Mr	Subbase Esb	Composite Inf K & Esb k	Composite Rgd-K & Esb k	Subdivision months m	Damage ur
frozen	2560	15000	136	136	2.50	53
sprg thw	850	15000	52	52	1.00	73
dry	2020	15000	111	111	5.50	58
wet	1600	15000	90	90	3.00	62
				Sum m =	12.00	
					avg ur	59

NOTE: uri is based upon exact equation HH.17
not HH.19 used for Fig. 3.5

Figure 7.7 Design Example N-103: Poor Drainage

AASHTO 1993 Rigid Pavement Design

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(Rev. 1.2 6/5/98)

3/18/99 14:04 Project Description: **N-103 Wilber to Crete at Test Site - Good Drainage**

Design Assumptions

STD Dev	Zr =	-1.037	
STD Err	So =	0.35	
Eff Mod Sub R	k =	103	pci
Rigid Layer Depth	Dsg =	40	ft
Mod of Rupture	S'c =	560	psi
E Mod of Concrete	Ec =	4200000	psi
Depth Subbase	Dsb =	4	in
Initial PSI	po =	4.5	
Terminal PSI	pt =	2.5	
Envrnmtl PSI Loss	dPSI =	0.25	
Load Transfer Coef	J =	3.8	
	Estimated Pav D =	6.20	in
	Loss of Support =	0.9	
	Drn Coef Cd =	1.05	
	Dsgn P (years)	20	
	Daily ESALS 2-way	40	
	Trffc Grwth i %	1	
	% Trffc Dsgn Ln	100	
Loss of Support Effective k =		45	**Use Fig.II-3.6 - 93 Guide**
	Predicted ESALS =	1.615E+05	

Seasons	Subgrade Mr	Subbase Esb	Composite Inf K & Esb k	Composite Rgd-K & Esb k	Subdivision months m	Damage ur
frozen	2560	15000	136	136	2.50	24
sprg thw	850	15000	52	52	1.00	35
dry	2020	15000	111	111	5.50	27
wet	1600	15000	90	90	3.00	29
				Sum m =	12.00	
					avg ur	27

NOTE: uri is based upon exact equation HH.17 not HH.19 used for Fig. 3.5

D	Calc W18's	Log (W18)	Term 1	Term 2	Term 3
6.20	1.667E+05	5.22189	5.93844	-0.18276	-0.5338

Figure 7.8 Design Example N-103: Good Drainage

Chapter 8

CONCLUSIONS

Nebraska pavement sections with bases and subbases stabilized by fines without edge drains are rated by the 1993 AASHTO Design Guide as having Poor or Very poor drainage. Field permeability values greater than 200 ft/day with edge drains are required to provide sufficient suction to drain a pavement section fast enough to be rated Fair to Good. The use of recycled crushed PCC and AC pavement by NDOR for a drainable base or subbase using the present gradation provides a laboratory permeability of 10 to 20 ft/day. This laboratory permeability is more than an order of magnitude below that required to produce drainage rated Fair to Good. It must be noted that field permeabilities are usually taken to be 1/3 of the laboratory measured values. Using this criterion, if edge drains are used, the laboratory permeability must be 300 to 600 ft/day.

A chart of drainage time to achieve 50 percent saturation for bases and subbases with edge drains was developed. Using this chart recommended values for drainage coefficients for PCC and AC pavements can be determined from the 1993 AASHTO Design Guide.

The denominator of Equation 5.3, relating k to M_r in the Design Guide, should be 91 not 19.4 for the Nebraska sites tested. Using 91, good correlations between lab M_r and k estimated from AREA calculations using FWD deflections were found. AASHTO classification – k correlations were consistent with site data when S_r was considered.

Extension of M_r into the near saturation range can be accomplished by simple linear extrapolations of the existing M_r data base utilizing S_r values from existing dry of optimum, optimum and wet of optimum test data. These data, when divided by 91, provide static elastic top of the subgrade estimates of k for design consistent with published data.

The existing M_r database is reliable and can be used for design to include seasonal variation of S_r by a simple process of extrapolation.

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