Design Methods

- Highway Pavements
  - AASHTO
  - The Asphalt Institute
  - Portland Cement Association

- Airfield Pavements
  - FAA
  - The Asphalt Institute
  - Portland Cement Association
  - U.S. Army Corps of Engineers

Objectives of Pavement Design

To provide a surface that is:

- Strong
  - Surface strength
  - Moisture control

- Smooth

- Safe
  - Friction
  - Drainage

- Economical
  - Initial construction cost
  - Recurring maintenance cost
Pavements are Designed to Fail!!

Pavement Design Methodologies

• Experience
• Empirical
  ■ Statistical models from road tests
• Mechanistic-Empirical
  ■ Calculation of pavement stresses/strains/deformations
  ■ Empirical pavement performance models
• Mechanistic
  ■ Calculation of pavement stresses/strains/deformations
  ■ Mechanics-based pavement performance models
Empirical vs. Mechanistic Design

Wood Floor Joist

Empirical “Rule of 2”:

\[ d \text{ in inches} = \frac{L \text{ in feet}}{2} + 2 \]

Mechanistic:

\[ \sigma_{\text{bending}} = \frac{PL}{4S} \leq \sigma_{\text{allowable}} \]
AASHTO Pavement Design Guide

- Empirical design methodology
- Several versions:
  - 1961 (Interim Guide)
  - 1972
  - 1986
    - Refined material characterization
    - Version included in Huang (1993)
  - 1993
    - More on rehabilitation
    - More consistency between flexible, rigid designs
    - Current version
  - 2002
    - Under development
    - Will be based on mechanistic-empirical approach

AASHO Road Test (late 1950’s)

(AASHO, 1961)
One Rainfall Zone...

(AASHO, 1961)

One Temperature Zone...

(AASHO, 1961)
One Subgrade...

A-6 / A-7-6 (Clay)
Poor Drainage

Figure 16. Embankment construction, loop 1, using rotary speed mixers to process and adjust moisture content of soil.
(AASHO, 1961)

Limited Set of Materials...

- One asphalt concrete
  - 3/4” surface course
  - 1” binder course
- One Portland cement concrete (3500 psi @ 14 days)
- Four base materials
  - Well-graded crushed limestone (main experiment)
  - Well-graded uncrushed gravel (special studies)
  - Bituminous-treated base (special studies)
  - Cement-treated base (special studies)
- One uniform sand/gravel subbase
1950’s Construction Methods...

Figure 17. Compacting ribbons.

Figure 29. Bituminous concrete construction.

(AASHO, 1961)

1950’s Vehicle Loads...

Figure 23. Test vehicles, showing typical axle arrangement and loadings.

(AASHO, 1961)
Limited Traffic Volumes...

1.1M Axles

2 Years

(AASHO, 1961)

1950’s Data Analysis...

(AASHO, 1961)
Some Failures...

(Some pavements too!)

AASHTO Design Based on Serviceability Decrease

\[ \Delta \text{PSI} = \Delta \text{PSI}_{\text{Traffic}} + \Delta \text{PSI}_{\text{Swell/Frost Heave}} \quad (1.3.1) \]

where

\[ \Delta \text{PSI} = \text{total loss of serviceability}, \]
\[ \Delta \text{PSI}_{\text{Traffic}} = \text{serviceability loss due to traffic (ESAL's)}, \] and
\[ \Delta \text{PSI}_{\text{Swell/Frost Heave}} = \text{serviceability loss due to swelling and/or frost heave of roadbed soil}. \]
What is Serviceability?

- Based upon Present Serviceability Rating (PSR)
- Subjective rating by individual/panel
  - Initial/post-construction
  - Various times after construction
- $0 < \text{PSR} < 5$
- $\text{PSR} < -2.5$: Unacceptable

Present Serviceability Index (PSI)

- PSR correlated to physical pavement measures via Present Serviceability Index (PSI):

$$\text{PSI} = 5.03 - 1.91 \log(1 + SV) - 1.38 \overline{RD}^2 - 0.01(C + P)^{1/2}$$

$SV = \text{slope variance (measure of roughness)}$
$\overline{RD} = \text{average rut depth (inches)}$
$C + P = \text{area of cracking and patching per 1000 ft}^2$

$\text{PSI} \approx \text{PSR}$

Empirical!

Part I: Pavement Design and Management Principles

- Introduction and Background
- Design Related to Project Level Pavement Management
- Economic Evaluation of Alternative Design Strategies
- Reliability


Part II: Pavement Design Procedures for New Construction or Reconstruction

- Design Requirements
- Highway Pavement Structural Design
- Low-Volume Road Design

Part III: Pavement Design Procedures for Rehabilitation of Existing Pavements

- Rehabilitation Concepts
- Guides for Field Data Collection
- Rehabilitation Methods Other Than Overlay
- Rehabilitation Methods With Overlays

Design Scenarios Included in AASHTO Guide

<table>
<thead>
<tr>
<th>Description</th>
<th>Flexible</th>
<th>Rigid</th>
<th>CRCP</th>
<th>PCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 DESIGN VARIABLES</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>2.1.1 Type Confinement</td>
<td></td>
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<tr>
<td>Performance Period</td>
<td>1 1 1 1 1</td>
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<td>Analysis Period</td>
<td>1 1 1 1 1</td>
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<tr>
<td>2.1.2 Traffic</td>
<td></td>
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<td>2.1.3 Rehabilitation</td>
<td></td>
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<td>Roadway Widening</td>
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<td>2.2 PERFORMANCE CRITERIA</td>
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<td>2.2.1 Serviceability</td>
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<td>2.2.2 Allowable Traffic</td>
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<td>2.2.3 Aggregate Loss</td>
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<td>2.3 MATERIAL PROPERTIES FOR STRUCTURAL DESIGN</td>
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<td>2.3.1 Effective Load Factor Modulus</td>
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<td></td>
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<tr>
<td>2.3.2 Effective Modulus of Subgrade Resistance</td>
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<td></td>
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<td>2.3.3 Percent Layer Material Characterization</td>
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<tr>
<td>2.3.4 PCC Modulus of Repairs</td>
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<td>2.3.5 Layer Coefficients</td>
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<td>2.4 BASEMENT STRUCTURAL CHARACTERISTICS</td>
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<td>2.4.1 Design</td>
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<td>Rigid Pavement</td>
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<td></td>
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<td>2.4.2 Load Impacts</td>
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<td>Axial Loads</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Loads</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>2.4.3 Loss of Support</td>
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</tr>
<tr>
<td>2.5 REINFORCEMENT VARIABLES</td>
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<tr>
<td>2.5.1 Joints</td>
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<td></td>
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</tr>
<tr>
<td>Subgrade</td>
<td>1 1</td>
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<td></td>
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<tr>
<td>Webbing</td>
<td>1 1</td>
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</tr>
<tr>
<td>Prestressed</td>
<td>1 1</td>
<td></td>
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<td>2.5.2 Prestressed</td>
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<tr>
<td>Concrete Joints</td>
<td>1 1</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Stiffness</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Thermal Conductivity</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Bitumen</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Thermal Conductivity</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Temperature Drop</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestressed</td>
<td>1 1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(AASHTO, 1993)
AASHTO Design Based on Serviceability Decrease

\[
\Delta PSI = \Delta PSI_{\text{Traffic}} + \Delta PSI_{\text{Settle/Frost Heave}} \tag{1.3.1}
\]

where

\(\Delta PSI\) = total loss of serviceability,
\(\Delta PSI_{\text{Traffic}}\) = serviceability loss due to traffic (ESAL's), and
\(\Delta PSI_{\text{Settle/Frost Heave}}\) = serviceability loss due to swelling and/or frost heave of roadbed soil.

(AASHTO, 1993)

Flexible Pavements
Design Equation

\[
\log_{10}(W_{18}) = Z_R S_o + 9.36 \log_{10}(SN + 1) - 0.20 \\
+ \log_{10}\left(\frac{\Delta PSI}{4.2 - 1.5}\right) + 2.32 \log_{10}(M_R) - 8.07 \\
0.40 + \frac{1}{(SN + 1)^{0.19}}
\]

- \(W_{18}\) = design traffic (18-kip ESALs)
- \(Z_R\) = standard normal deviate
- \(S_o\) = combined standard error of traffic and performance prediction
- \(\Delta PSI\) = difference between initial and terminal serviceability index
- \(M_R\) = resilient modulus (psi)
- \(SN\) = structural number

Figure 3.1. Design Chart for Flexible Pavements Based on Using Mean Values for Each Input

(AASHTO, 1993)
Traffic vs. Analysis Period

Figure 3.1. Example Plot of Cumulative 18-kip ESAL Traffic Versus Time
(AASHTO, 1993)

Analysis Period

<table>
<thead>
<tr>
<th>Highway Conditions</th>
<th>Analysis Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-volume urban</td>
<td>30–50</td>
</tr>
<tr>
<td>High-volume rural</td>
<td>20–50</td>
</tr>
<tr>
<td>Low-volume paved</td>
<td>15–25</td>
</tr>
<tr>
<td>Low-volume aggregate surface</td>
<td>10–20</td>
</tr>
</tbody>
</table>

(Also basis for life-cycle cost analysis)

(AASHTO, 1993)
Design Traffic (18K ESALs)

\[ w_{18} = D_D \times D_L \times \hat{w}_{18} \]

where

- \( D_D \) = a directional distribution factor, expressed as a ratio, that accounts for the distribution of ESAL units by direction, e.g., east-west, north-south, etc.,
- \( D_L \) = a lane distribution factor, expressed as a ratio, that accounts for distribution of traffic when two or more lanes are available in one direction, and
- \( \hat{w}_{18} \) = the cumulative two-directional 18-kip ESAL units predicted for a specific section of highway during the analysis period (from the planning group).

(AASHTO, 1993)

Design Traffic (18K ESALs)

- \( D_D = 0.5 \) typically
- \( D_L \):

<table>
<thead>
<tr>
<th>Number of Lanes in Each Direction</th>
<th>Percent of 18-kip ESAL in Design Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>80–100</td>
</tr>
<tr>
<td>3</td>
<td>60–80</td>
</tr>
<tr>
<td>4</td>
<td>50–75</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)
Reliability

Recommended Values for Standard Error $S_0$

- Rigid Pavements: 0.30 - 0.40
- Flexible Pavements: 0.40 - 0.50
Standard Normal Deviate $Z_R$

Table 4.1. Standard Normal Deviate ($Z_R$) Values Corresponding to Selected Levels of Reliability

<table>
<thead>
<tr>
<th>Reliability, R (percent)</th>
<th>Standard Normal Deviate, $Z_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>$-0.000$</td>
</tr>
<tr>
<td>60</td>
<td>$-0.253$</td>
</tr>
<tr>
<td>70</td>
<td>$-0.524$</td>
</tr>
<tr>
<td>75</td>
<td>$-0.674$</td>
</tr>
<tr>
<td>80</td>
<td>$-0.841$</td>
</tr>
<tr>
<td>85</td>
<td>$-1.037$</td>
</tr>
<tr>
<td>90</td>
<td>$-1.282$</td>
</tr>
<tr>
<td>91</td>
<td>$-1.340$</td>
</tr>
<tr>
<td>92</td>
<td>$-1.405$</td>
</tr>
<tr>
<td>93</td>
<td>$-1.476$</td>
</tr>
<tr>
<td>94</td>
<td>$-1.555$</td>
</tr>
<tr>
<td>95</td>
<td>$-1.645$</td>
</tr>
<tr>
<td>96</td>
<td>$-1.751$</td>
</tr>
<tr>
<td>97</td>
<td>$-1.881$</td>
</tr>
<tr>
<td>98</td>
<td>$-2.054$</td>
</tr>
<tr>
<td>99</td>
<td>$-2.327$</td>
</tr>
<tr>
<td>99.9</td>
<td>$-3.090$</td>
</tr>
<tr>
<td>99.99</td>
<td>$-3.750$</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)

Recommended Reliability Levels

Table 2.2. Suggested Levels of Reliability for Various Functional Classifications

<table>
<thead>
<tr>
<th>Functional Classification</th>
<th>Recommended Level of Reliability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Urban</td>
</tr>
<tr>
<td>Interstate and Other Freeways</td>
<td>85–99.9</td>
</tr>
<tr>
<td>Principal Arterials</td>
<td>80–99</td>
</tr>
<tr>
<td>Collectors</td>
<td>80–95</td>
</tr>
<tr>
<td>Local</td>
<td>50–80</td>
</tr>
</tbody>
</table>

Note: Results based on a survey of the AASHTO Pavement Design Task Force.

(AASHTO, 1993)
Serviceability

$$\Delta PSI = p_o - p_t$$

- $PSI$ = Pavement Serviceability Index, $1 < PSI < 5$
- $p_o$ = Initial Serviceability Index
  - Rigid pavements: 4.5
  - Flexible pavements: 4.2
- $p_t$ = Terminal Serviceability Index

<table>
<thead>
<tr>
<th>Terminal Serviceability Level</th>
<th>Percent of People Stating Unacceptable</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>12</td>
</tr>
<tr>
<td>2.5</td>
<td>55</td>
</tr>
<tr>
<td>2.0</td>
<td>85</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)

Adjustment of Roadbed (Subgrade) $M_R$ for Seasonal Variations

(AASHTO, 1993)
Structural Number

\[ SN = a_i D_i + \sum_{i=2}^{n} a_i D_i m_i \]

- \( SN \) = structural number = \( f \) (structural capacity)
- \( a_i \) = \( i \)th layer coefficient
- \( D_i \) = \( i \)th layer thickness (inches)
- \( m_i \) = \( i \)th layer drainage coefficient
- \( n \) = number of layers (3, typically)

No Unique Solution!

![Diagram of pavement layers with formulas and requirements](image)

1. \( a, D, m \) and \( SN \) are as defined in the text and are minimum required values.
2. An asterisk (*) with \( D \) or \( SN \) indicates that it represents the value actually used, which must be equal to or greater than the required value.

Figure 3.2: Procedure for Determining Thicknesses of Layers Using a Layered Analysis Approach (AASHTO, 1993)
Layer Coefficient $a_1$: Asphalt Concrete

![Graph showing the relationship between Structural Layer Coefficient, $a_1$, for Asphalt Concrete and Elastic Modulus, $E_{base}$ (psi).](image)

Figure 2.5. Chart for Estimating Structural Layer Coefficient of Dense-Graded Asphalt Concrete Based on the Elastic (Resilient) Modulus ($J$) (AASHTO, 1993)

Layer Coefficient $a_2$: Granular Base

$$a_2 \approx 0.249 \left( \log_{10} E_{base} \right) - 0.977$$

$E_{base}$ in psi

![Graph showing the variation in Granular Base Layer Coefficient ($a_2$) with Various Base Strength Parameters ($J$).](image)

Figure 3.6. Variation in Granular Base Layer Coefficient ($a_2$) with Various Base Strength Parameters ($J$) (AASHTO, 1993)
Layer Coefficient $a_2$: Cement Treated Base

Figure 2.8. Variation in $a_2$ for Cement-Treated Base with Base Strength Parameter ($f$)

(AASHTO, 1993)

Layer Coefficient $a_2$: Bituminous Treated Base

Figure 2.9. Variation in $a_2$ for Bituminous-Treated Base with Base Strength Parameter ($f$)

(AASHTO, 1993)
Layer Coefficient $a_3$: Granular Subbase

$$a_3 = 0.227 \log_{10} E_{subbase} - 0.839$$

$E_{subbase}$ in psi

Quality of Drainage

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Water Removed Within</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>2 hours</td>
</tr>
<tr>
<td>Good</td>
<td>1 day</td>
</tr>
<tr>
<td>Fair</td>
<td>1 week</td>
</tr>
<tr>
<td>Poor</td>
<td>1 month</td>
</tr>
<tr>
<td>Very poor</td>
<td>(water will not drain)</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)
Drainage Coefficient $m_i$

$m_i$ increases/decreases the effective value for $a_i$

Table 2.4. Recommended $m_i$ Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Less Than 1%</th>
<th>1-5%</th>
<th>5-25%</th>
<th>Greater Than 25%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>1.40-1.35</td>
<td>1.35-1.30</td>
<td>1.30-1.20</td>
<td>1.20</td>
</tr>
<tr>
<td>Good</td>
<td>1.35-1.25</td>
<td>1.25-1.15</td>
<td>1.15-1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Fair</td>
<td>1.25-1.15</td>
<td>1.15-1.05</td>
<td>1.00-0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>Poor</td>
<td>1.15-1.05</td>
<td>1.05-0.80</td>
<td>0.80-0.60</td>
<td>0.60</td>
</tr>
<tr>
<td>Very poor</td>
<td>1.05-0.95</td>
<td>0.95-0.75</td>
<td>0.75-0.40</td>
<td>0.40</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)

Table 3.1. Example of Process Used to Predict the Performance Period of an Initial Pavement Structure Considering Swelling and/or Frost Heave

<table>
<thead>
<tr>
<th>(2) Trial Performance Period (years)</th>
<th>(3) Total Serviceability Loss Due to Swelling and Frost Heave $\Delta PSI_{sw, fh}$</th>
<th>(4) Corresponding Serviceability Loss Due to Traffic $\Delta PSI_{tr}$</th>
<th>(5) Allowable Cumulative Traffic (10-kip ESAL)</th>
<th>(6) Corresponding Performance Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13.0</td>
<td>0.73</td>
<td>1.17</td>
<td>$2.0 \times 10^6$</td>
</tr>
<tr>
<td>2</td>
<td>9.7</td>
<td>0.63</td>
<td>1.27</td>
<td>$2.3 \times 10^6$</td>
</tr>
<tr>
<td>3</td>
<td>8.5</td>
<td>0.56</td>
<td>1.34</td>
<td>$2.6 \times 10^6$</td>
</tr>
</tbody>
</table>

Column No. Description of Procedures
2 Estimated by the designer (Step 2).
3 Using estimated value from Column 2 with Figure 2.2, the total serviceability loss due to swelling and frost heave is determined (Step 3).
4 Subtract environmental serviceability loss (Column 3) from design total serviceability loss to determine corresponding serviceability loss due to traffic.
5 Determined from Figure 3.1 keeping all input constant (except for use of traffic serviceability loss from Column 4) and applying the chart in reverse (Step 5).
6 Using the traffic from Column 5, estimate net performance period from Figure 2.1 (Step 6).

(AASHTO, 1993)
Traffic vs. Analysis Period

(AASHTO, 1993)

Figure 2.1. Example Plot of Cumulative 8-Kip ESAL Traffic Versus Time

(AASHTO, 1993)

Figure 2.2. A Conceptual Example of the Environmental Suitability Loss Versus Time Graph that may be Developed for a Specific Location
Effect of Frost on Performance

PSI = Pavement Servicability Index

1 < PSI < 5

"Failure": PSI < 2+

Figure G6. Chart for Estimating Servicability Loss Due to Frost Heave (AASHTO, 1993)

Frost Heave Rate $\phi$

$\phi = f (-0.02\text{mm})$

Figure G6. Chart for Estimating Frost Heave Rate for a Rutted Soil, Part II (AASHTO, 1993)
Maximum Serviceability Loss

\[ \Delta \text{PSI}_{\text{max}} = f(\text{frost depth, drainage}) \]

Effect of Swelling on Performance

PSI = Pavement Servicability Index

1 < PSI < 5

"Failure": PSI < 2+

Figure G.7. Graph for Estimating Maximum Serviceability Loss Due to Frost Heave  
(AASHTO, 1993)

Figure G.4. Chart for Estimating Serviceability Loss Due to Ravelled Swelling  
(AASHTO, 1993)
Swell Rate Constant $\theta$

$\theta = f \text{ (moisture supply, soil fabric) }$

---

Maximum Potential Heave $V_R$

$V_R = f \text{ (PI, compaction, thickness) }$

---

(AASHTO, 1993)
Rigid Pavements

Design Equation

\[
\log_{10}(W_{18}) = Z_e S_e + 7.35 \log_{10}(D+1) - 0.06 + \log_{10} \left[ \frac{\Delta PSI}{4.5 - 1.5} \right] \\
+ \log_{10} \left[ \frac{1}{1 + 1.64 \times 10^3 \left( \frac{D + 1}{2} \right)^{0.5}} \right] \\
\left[ \frac{S'_c C_d (D^{0.75} - 1.132)}{215.63 J \left( D^{0.75} - \frac{18.42}{(E_c / k)^{0.325}} \right)} \right] \\
\]

- \( W_{18} \) = design traffic (18-kip ESALs)
- \( Z_e \) = standard normal deviate
- \( S_c \) = combined standard error of traffic and performance prediction
- \( D \) = thickness (inches) of pavement slab
- \( \Delta PSI \) = difference between initial and terminal serviceability indices
- \( p_i \) = terminal serviceability value
- \( S'_c \) = modulus of rupture (psi) for Portland cement concrete
- \( J \) = load transfer coefficient
- \( C_d \) = drainage coefficient
- \( E_c \) = modulus of elasticity (psi) for Portland cement concrete
- \( k \) = modulus of subgrade reaction (pci)
Figure 3.7. Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 1)

Figure 3.7. Continued—Design Chart for Rigid Pavements Based on Using Mean Values for Each Input Variable (Segment 3)
Design Inputs

$W_{18} =$ design traffic (18-kip ESALs)
$Z_R =$ standard normal deviate
$S_o =$ combined standard error of traffic and performance prediction
$\Delta PSI =$ difference between initial and terminal serviceability indices
$p_t =$ terminal serviceability index (implicit in flexible design)

*All consistent with flexible pavements!* 

Additional Design Inputs

- $S'_c =$ modulus of rupture for concrete
- $J =$ joint load transfer coefficient
- $C_d =$ drainage coefficient (similar in concept to flexible pavement terms)
- $E_c =$ modulus of elasticity for concrete
- $k =$ modulus of subgrade reaction

*Additional inputs reflect differences in materials and structural behavior.*
Modulus of Rupture $S_c'$

Because of the treatment of reliability in this Guide, it is strongly recommended that the normal construction specification for modulus of rupture (flexural strength) not be used as input, since it represents a value below which only a small percent of the distribution may lie. If it is desirable to use the construction specification, then some adjustment should be applied, based on the standard deviation of modulus of rupture and the percent (PS) of the strength distribution that normally falls below the specification:

$$S_c'(\text{mean}) = S_c + z(\text{SD}_c)$$

where

- $S_c'$ = estimated mean value for PCC modulus of rupture (psi),
- $S_c$ = construction specification on concrete modulus of rupture (psi),
- $\text{SD}_c$ = estimated standard deviation of concrete modulus of rupture (psi), and
- $z$ = standard normal variate:
  - $0.841$, for $PS = 20$ percent,*
  - $1.037$, for $PS = 15$ percent,
  - $1.282$, for $PS = 10$ percent,
  - $1.645$, for $PS = 5$ percent, and
  - $2.327$, for $PS = 1$ percent.

*NOTE: Permissible number of specimens, expressed as a percentage, that may have strengths less than the specification value.

---

Joint Load Transfer Coefficient $J$

<table>
<thead>
<tr>
<th>Pavement Type</th>
<th>$J$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(no tied shoulders)</td>
<td></td>
</tr>
<tr>
<td>JCP/JRCP w/ load transfer devices</td>
<td>3.2</td>
</tr>
<tr>
<td>JCP/JRCP w/out load transfer devices</td>
<td>3.8-4.4</td>
</tr>
<tr>
<td>CRCP</td>
<td>2.9</td>
</tr>
</tbody>
</table>
Joint Load Transfer Coefficient $J$

Additional benefits of tied shoulders:

Table 2.6. Recommended Load Transfer Coefficient for Various Pavement Types and Design Conditions

<table>
<thead>
<tr>
<th>Shoulder</th>
<th>Asphalt</th>
<th>Tied P.C.C.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Transfer Devices</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Pavement Type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Plain jointed and</td>
<td>3.2</td>
<td>3.8-4.4</td>
</tr>
<tr>
<td>jointed reinforced</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. CRCP</td>
<td>2.9-3.2</td>
<td>N/A</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)

Drainage Coefficient $C_d$

- Two effects:
  - Subbase and subgrade strength/stiffness
  - Joint load transfer effectiveness

Table 2.5. Recommended Values of Drainage Coefficient, $C_d$, for Rigid Pavement Design

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Less Than 1%</th>
<th>1-5%</th>
<th>5-25%</th>
<th>Greater Than 25%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>1.25-1.20</td>
<td>1.20-1.15</td>
<td>1.15-1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>Good</td>
<td>1.20-1.15</td>
<td>1.15-1.10</td>
<td>1.10-1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Fair</td>
<td>1.15-1.10</td>
<td>1.10-1.00</td>
<td>1.00-0.90</td>
<td>0.90</td>
</tr>
<tr>
<td>Poor</td>
<td>1.10-1.00</td>
<td>1.00-0.90</td>
<td>0.90-0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>Very poor</td>
<td>1.00-0.90</td>
<td>0.90-0.80</td>
<td>0.80-0.70</td>
<td>0.70</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)
**PCC Modulus of Elasticity $E_c$**

- Measure directly per ASTM C469
- Correlation w/ compressive strength:

  $$E_c = 57,000 \left(f'_c \right)^{0.5}$$

  $E_c = \text{elastic modulus (psi)}$
  $f'_c = \text{compressive strength (psi) per AASHTO T22, T140, or ASTM C39}$

**Effective Subgrade Modulus $k$**

- Depends on:
  - Roadbed (subgrade) resilient modulus, $M_R$
  - Subbase resilient modulus, $E_{SB}$
- Both vary by season
Determining Effective \( k \) (See Table 3.2)

- Identify:
  - Subbase types
  - Subbase thicknesses
  - Loss of support, \( LS \) (erosion potential of subbase)
  - Depth to rigid foundation (feet)
- Assign roadbed soil resilient modulus \( (M_R) \) for each season
- Assign subbase resilient modulus \( (E_{SB}) \) for each season
  - \( 15,000 \text{ psi (spring thaw)} < E_{SB} < 50,000 \text{ psi (winter freeze)} \)
  - \( E_{SB} < 4(M_R) \)

### Table 3.2: Table for Estimating Effective Modulus of Subgrade Reaction

<table>
<thead>
<tr>
<th>Month</th>
<th>Roadbed Modulus, ( M_a ) (psi)</th>
<th>Subbase Modulus, ( E_m ) (psi)</th>
<th>Composite k Value (psi) (Fig. 3.5)</th>
<th>k Value (psi) on Rigid Foundation (Fig. 3.6)</th>
<th>Relative Damage, ( n_a ) (Fig. 3.7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mar.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apr.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>May</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>June</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>July</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aug.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sept.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oct.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nov.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dec.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average: \( n_a = \frac{\text{Summation of } n_a}{12} \)

Effective Modulus of Subgrade Reaction, \( k \) (psi) = 

Corrected for Loss of Support, \( k \) (psi) = 

\( (AASHTO, 1993) \)
Determining Effective k (cont’d)

• Determine composite $k$ for each season
  - For $DSB = 0$: $k = \frac{M_R}{19.4}$
  - For $DSB > 0$: Use Figure 3.3

• If depth to rigid foundation < 10 feet, correct $k$ for effect of rigid foundation near the surface (Figure 3.4)

• Estimate required thickness of slab (Figure 3.5) and determine relative damage $u_r$ for each season

• Use average $u_r$ to determine effective $k$ (Figure 3.5)

• Correct $k$ for potential loss of support $LS$ (Figure 3.6)

\[
k = f (M_R, E_{SB}, D_{SB})
\]

**Composite Modulus of Subgrade Reaction**

(AASHTO, 1993)

Figure 3.3. Chart for Estimating Composite Modulus of Subgrade Reaction, $k_{cm}$. Assuming a semi-infinite Subgrade Depth. (For practical purposes, a semi-infinite depth is considered to be greater than 10 feet below the surface of the subgrade.)
Rigid Foundation Correction

Relative Damage
\[ u_r = f(k, D) \]
Subbase/subgrade erosion at joints causes Loss of Support, impairs load transfer.

(AASHTO, 1993)
Loss of Support

Figure 3.6. Correction of Effective Modulus of Subgrade Reaction for Potential Loss of Subbase Support (6)

Table 3.3. Example Application of Method for Estimating Effective Modulus of Subgrade Reaction

<table>
<thead>
<tr>
<th>Month</th>
<th>Bedded Modulus, ( M_a ) (pci)</th>
<th>Subbase Modulus, ( E_b ) (pci)</th>
<th>Composite K Value (pci) (Fig. 3.5)</th>
<th>S-N value (pci) on Rigid Foundation (Fig. 3.4)</th>
<th>Relative Thinner, ( \epsilon_r ) (Fig. 3.4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan.</td>
<td>20,000</td>
<td>50,000</td>
<td>1,100</td>
<td>1,150</td>
<td>0.35</td>
</tr>
<tr>
<td>Feb.</td>
<td>20,000</td>
<td>50,000</td>
<td>1,100</td>
<td>1,150</td>
<td>0.35</td>
</tr>
<tr>
<td>Mar.</td>
<td>2,500</td>
<td>15,000</td>
<td>140</td>
<td>170</td>
<td>0.86</td>
</tr>
<tr>
<td>Apr.</td>
<td>4,000</td>
<td>15,000</td>
<td>230</td>
<td>300</td>
<td>0.72</td>
</tr>
<tr>
<td>May</td>
<td>4,000</td>
<td>15,000</td>
<td>230</td>
<td>300</td>
<td>0.78</td>
</tr>
<tr>
<td>Jun.</td>
<td>7,000</td>
<td>20,000</td>
<td>400</td>
<td>540</td>
<td>0.88</td>
</tr>
<tr>
<td>July</td>
<td>7,000</td>
<td>20,000</td>
<td>400</td>
<td>540</td>
<td>0.88</td>
</tr>
<tr>
<td>Aug.</td>
<td>7,000</td>
<td>20,000</td>
<td>400</td>
<td>540</td>
<td>0.88</td>
</tr>
<tr>
<td>Sept.</td>
<td>7,000</td>
<td>20,000</td>
<td>400</td>
<td>540</td>
<td>0.88</td>
</tr>
<tr>
<td>Oct.</td>
<td>7,000</td>
<td>20,000</td>
<td>400</td>
<td>540</td>
<td>0.88</td>
</tr>
<tr>
<td>Nov.</td>
<td>4,000</td>
<td>15,000</td>
<td>230</td>
<td>300</td>
<td>0.78</td>
</tr>
<tr>
<td>Dec.</td>
<td>20,000</td>
<td>50,000</td>
<td>1,100</td>
<td>1,150</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Average, \( \epsilon_r = \frac{4.3}{10} = 0.43 \)  
Effective Modulus of Subgrade Reaction, \( k \) (pci) = \( \frac{540}{1.75} \)  
Corrected for Loss of Support, \( k \) (pci) = \( \frac{540}{1.35} \)  

(AASHTO, 1993)
Table 3.4. Example of Process Used to Predict the Performance Period of an Initial Rigid Pavement Structure Considering Swelling and/or Frost Heave

<table>
<thead>
<tr>
<th>Slab Thickness (inches)</th>
<th>9.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Possible Performance Period (years)</td>
<td>20</td>
</tr>
<tr>
<td>Design Serviceability Loss, ΔPSI = p_t - p_o =</td>
<td>4.2 - 2.5 = 1.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(1) Iteration No.</th>
<th>(2) Performance Period (years)</th>
<th>(3) Total Serviceability Loss Due to Swelling and Frost Heave ΔFSHE+FHE</th>
<th>(4) Corresponding Serviceability Loss Due to Traffic ΔPSI</th>
<th>(5) Allowable Traffic (18-kip ESAL) Period (years)</th>
<th>(6) Corresponding Performance Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.0</td>
<td>0.75</td>
<td>0.95</td>
<td>3.1 x 10^6</td>
<td>9.6</td>
</tr>
<tr>
<td>2</td>
<td>11.8</td>
<td>0.69</td>
<td>1.01</td>
<td>3.3 x 10^6</td>
<td>10.2</td>
</tr>
<tr>
<td>3</td>
<td>11.0</td>
<td>0.67</td>
<td>1.03</td>
<td>3.4 x 10^6</td>
<td>10.4</td>
</tr>
</tbody>
</table>

Column No. Description of Procedures

2 Estimated by the designer (Step 2).
3 Using estimated value from Column 2 with Figure 2.2, the total serviceability loss due to swelling and frost heave is determined (Step 3).
4 Subtract environmental serviceability loss (Column 3) from design total serviceability loss to determine corresponding serviceability loss due to traffic.
5 Determined from Figure 3.5 keeping all inputs constant (except for use of traffic serviceability loss from Column 4) and applying the chart in reverse (Step 5).
6 Using the traffic from Column 5, estimate net performance period from Figure 2.1 (Step 6).

Consistent with flexible pavement approach!

(AASHTO, 1993)

Traffic vs. Analysis Period

(AASHTO, 1993)
Joint Design

- Joint Types
  - Contraction
  - Expansion
  - Construction
  - Longitudinal
- Joint Geometry
  - Spacing
  - Layout (e.g., regular, skewed, randomized)
  - Dimensions
- Joint Sealant Dimensions

Types of Joints

- Contraction
  - Transverse
  - For relief of tensile stresses
- Expansion
  - Transverse
  - For relief of compressive stresses
  - Used primarily between pavement and structures (e.g., bridge)
- Construction
- Longitudinal
  - For relief of curling and warping stresses
Typical Contraction Joint Details

(a) Dummy Groove

(b) Premolded Strip

Typical Expansion Joint Detail

(Huang, 1993)
Typical Construction Joint Detail

First slab

Smooth, lubricated dowel bar

(a) Butt Joint at Contraction Joint.

(b) Key Joint for Emergency.

Typical Longitudinal Joint Detail

Deformed bar

(a) Dummy Groove.

(b) Ribbon or Premolded Strip.

(c) Deformed Plate.

Full Width Construction

(Huang, 1993)
Typical Longitudinal Joint Detail

Groove 1/8" to 1/4" wide and 1" deep

Joint Spacing

- Local experience is best guide
- Rules of thumb:
  - JCP joint spacing (feet) ≤ 2D (inches)
  - W/L ≤ 1.25

Lane-at-a-Time Construction

(Huang, 1993)
Joint Dimensions

- Width controlled by joint sealant extension
- Depths:
  - Contraction joints: D/4
  - Longitudinal joints: D/3
- Joints may be formed by:
  - Sawing
  - Inserts
  - Forming

Joint Sealant Dimension

Governed by expected joint movement, sealant resilience

\[
\Delta L = \frac{CL(\alpha_s \times DT_D + Z)}{S} \times 100
\]

where

- \( \Delta L \) = the joint opening caused by temperature changes and drying shrinkage of the PCC, in.,
- \( S \) = allowable strain of joint sealant material. Most current sealants are designed to withstand strains of 25 to 35 percent, thus 25 percent may be used as a conservative value,
- \( \alpha_s \) = the thermal coefficient of contraction of portland cement concrete, °F,
- \( Z \) = the drying shrinkage coefficient of the PCC slab, which can be neglected for a resealing project, in./in.,
- \( L \) = joint spacing, in.,
- \( DT_D \) = the temperature range, °F, and
- \( C \) = the adjustment factor due to subbase/slab friction restraint. Use 0.65 for stabilized subbase, 0.80 for granular base.

\[ \text{(AASHTO, 1993)} \]
Design Inputs

Z

$\alpha_c$

Table 2.9. Approximate Relationship Between Shrinkage and Indirect Tensile Strength of Portland Cement Concrete ($\sigma$)

<table>
<thead>
<tr>
<th>Indirect Tensile Strength (psi)</th>
<th>Shrinkage (in./in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300 (or less)</td>
<td>0.0008</td>
</tr>
<tr>
<td>400</td>
<td>0.0006</td>
</tr>
<tr>
<td>500</td>
<td>0.00045</td>
</tr>
<tr>
<td>600</td>
<td>0.0003</td>
</tr>
<tr>
<td>700 (or greater)</td>
<td>0.0002</td>
</tr>
</tbody>
</table>

Table 2.10. Recommended Value of the Thermal Coefficient of PCC as a Function of Aggregate Types ($\alpha_c$)

<table>
<thead>
<tr>
<th>Type of Coarse Aggregate</th>
<th>Concrete Thermal Coefficient ($10^{-5}/{^\circ}F$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>6.6</td>
</tr>
<tr>
<td>Sandstone</td>
<td>6.5</td>
</tr>
<tr>
<td>Gravel</td>
<td>6.0</td>
</tr>
<tr>
<td>Granite</td>
<td>5.3</td>
</tr>
<tr>
<td>Basalt</td>
<td>4.8</td>
</tr>
<tr>
<td>Limestone</td>
<td>3.8</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)

Reinforcement Design (JRCP)

- Purpose of reinforcement is not to prevent cracking, but to hold tightly closed any cracks that may form
- Physical mechanisms:
  - Thermal/moisture contraction
  - Friction resistance from underlying material
- Design based on friction stress analysis

(Huang, 1993)
Dowel Bars: Transverse Joint Load Transfer

- “...size and spacing should be determined by the local agency’s procedures and/or experience.”
- Guidelines:
  - Dowel bar diameter = D/8 (inches)
  - Dowel spacing: 12 inches
  - Dowel length: 18 inches

Friction Stresses

Induces tensile stresses in concrete
Causes opening of transverse joints

\[ \sigma_c = \frac{\gamma L f_u}{2} \]

(a) Free Body Diagram

(b) Variation of Frictional Stress

(Huang, 1993)
Applies to both longitudinal and transverse steel reinforcement

(Generally, $P_s=0$ for $L<\sim15$ feet)

(Friction Factor)

Example:
$L = 36$ ft.
$F = 1.8$
$f_s = 30,000$ psi

Solution:
$P_s = 0.065$

Figure 3.8. Reinforcement Design Chart for Jointed Reinforced Concrete Pavements

Friction Factor

Table 2.8. Recommended Friction Factors (F)

<table>
<thead>
<tr>
<th>Type of Material Beneath Slab</th>
<th>Friction Factor (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface treatment</td>
<td>2.2</td>
</tr>
<tr>
<td>Lime stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Asphalt stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>Cement stabilization</td>
<td>1.8</td>
</tr>
<tr>
<td>River gravel</td>
<td>1.5</td>
</tr>
<tr>
<td>Crushed stone</td>
<td>1.5</td>
</tr>
<tr>
<td>Sandstone</td>
<td>1.2</td>
</tr>
<tr>
<td>Natural subgrade</td>
<td>0.9</td>
</tr>
</tbody>
</table>

(AASHTO, 1993)
Steel Working Stress

Based on preventing fracture and limiting permanent deformation.

(AASHTO, 1993)

Table 3.7. Allowable Steel Working Stress, ksi (10)

<table>
<thead>
<tr>
<th>Indirect Tensile Strength of Concrete at 28 days, psi</th>
<th>Reinforcing Bar Size*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. 4</td>
</tr>
<tr>
<td>300 (or less)</td>
<td>65</td>
</tr>
<tr>
<td>400</td>
<td>67</td>
</tr>
<tr>
<td>500</td>
<td>67</td>
</tr>
<tr>
<td>600</td>
<td>67</td>
</tr>
<tr>
<td>700</td>
<td>67</td>
</tr>
<tr>
<td>800 (or greater)</td>
<td>67</td>
</tr>
</tbody>
</table>

*For DWF proportional adjustments may be made using the wire diameter to bar diameter.

(AASHTO, 1993)

Transverse Tie Bars

(AASHTO, 1993)

Example: Distance from free edge = 24 ft.
D = 15 in.
Answer: Spacing = 16 in.

Figure 3.13. Recommended Maximum Tie Bar Spacings for PCC Pavements Assuming \( \frac{3}{4} \)-inch Diameter Tie Bars, Grade 40 Steel, and Subgrade Friction Factor of 1.5
**Transverse Tie Bars**

Example: Distance from free edge 24 ft.
D 10 in.
Answer: Spacing = 24 in.

Figure 3.14. Recommended Maximum Tie Bar Spacings for PCC Pavements Assuming 0.06-inch Diameter Tie Bars, Grade 40 Steel, and Slab Grade Friction of 1.3

(AASHTO, 1993)