WSDOT PAVEMENT GUIDE

Volume 2

Pavement Notes

For Design, Evaluation and Rehabilitation

Washington State Department of Transportation

First Time Users Click Here

February 1995

FOREWORD

This guide has been prepared for WSDOT personnel in designing, constructing and maintaining pavement structures. Volume 2 (Pavement Notes) contains 11 sections and is used to provide an overview of information important in designing pavement structures. Such information is intended to provide insight into the pavement design, evaluation, and rehabilitation process.

The units used in this Volume are a mixture of metric and U.S. customary. This is due, in part, because most of the referenced materials and procedures are still in U.S. Customary units. Eventually, this volume will be revised to contain only metric units.

This volume does not constitute a WSDOT standard.

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SECTION 1.0

AN INTRODUCTION TO PAVEMENT STRUCTURES

This SECTION is used to provide an overview of various pavement statistics, terminology, design practices (state, national, and international), and historical development.

1. OVERVIEW

1.1 NATIONAL HIGHWAY SYSTEMS

1.1.1 HIGHWAY SYSTEMS AND JURISDICTIONAL CONTROL

Highway or highway systems are referred to often as either a functional and/or Federal-aid system. All roads are classified functionally with a subset designated as Federal-aid systems which are eligible for Federal funding.

Jurisdictional Control of U. S. Roads and Streets [after Ref. (1.8)]

Jurisdiction	Centerline Kilometers (1989)		
	Rural	Urban	Total
State	1 133 221	154 510	1 287 731
Local	3 611 617	1 033 992	4 645 610
Federal	295 094	1 566	296 660
Total	5 039 933	1 190 068	6 230 004

Note: Federal jurisdiction includes roads in national forests and parks and roads on military and Indian reservations.

As of 1997, 61 percent of all roads and streets are paved (39 percent unpaved). In 1949, only 22 percent of all roads and streets were paved (78 percent unpaved).

1.1.1.1 Federal-aid Systems

(a) The National System of Interstate and Defense Highways
(b) The Federal-Aid Primary System
(c) The Federal-Aid Secondary System
(d) The Federal-Aid Urban System
(e) Special Federal Highway Systems

1.1.1.2 Functional Systems

(a) Principal Arterial System. Includes Interstate, other freeways and expressways (in urban areas only), and other principal arterials.

(b) Minor Arterial System

(c) Collector System

1.1.1.3 National Highway System (NHS)

The NHS plan was submitted to Congress on December 9, 1993, and was a requirement of Section 1006(a) of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). The basic goal of the NHS is to better integrate the various transportation modes in the U.S. It is currently envisioned that the NHS will serve

- 104 major ports
- 143 major airports
- 321 major Amtrak stations
- 191 rail/truck terminals
- 242 military and defense-related installations
- 53 border crossings (Canada—32, Mexico—21)

The total mileage in the system (as proposed) will be 255 361 km (75 percent rural and 25 percent urban). The NHS will include all of the current Interstate system.

1.1.2 OVERVIEW OF FEDERAL-AID SYSTEMS

1.1.2.1 Overview of Mileage — Nationwide Federal-Aid System [after Ref (1.8)]

Federal-Aid System	Centerline Kilometers (1989)		
	Rural	Urban	Total
Interstate	53 596	18 227	71 823
Primary	363 268	53 664	416 931
Secondary	643 868	—	643 868
Urban	—	236 630	236 630
Total Federal-Aid Systems	1 060 732	308 521	1 369 252
Not On Federal-Aid Systems	3 979 201	881 547	4 860 748

1.1.2.2 Interstate System

The 1956 Federal-Aid Highway Act created this system. The system was originally expected to cost $27,000,000,000 in 1956 dollars. The System is essentially complete (about 72 000 centerline km) [1.5]. From Reference 1.1 (p.117) ... "The cost-to-complete the System was reduced by nearly $14 billion by the provisions of the Federal-Aid Highway Act of 1981, which limited costs "...to the construction necessary to provide a minimum level of acceptable service on the Interstate System which shall consist of: (1) full access control; (2) a pavement design to accommodate the types and volumes of traffic anticipated for the 20-year period from date of authorization of the initial basic construction contract; (3) essential environmental requirements..."

The current emphasis is to preserve the System. In 1976 the 3R program was established as a national priority (resurfacing, restoration and rehabilitation). The 1981 Highway Act nearly tripled authorizations and established reconstruction as the fourth R (i.e., now known as 4R). The federal FY1985 authorization for 4R was $2,800,000,000 (compared to 1976 level of $175,000,000).

1.1.2.3 Primary System

This was the original Federal-aid system (established by the Federal Highway Act of 1921). Currently, this system consists of connected main highways important to interstate, statewide and regional travel. There are 364 000 km (or 89 percent of the total Primary System) of rural routes with 53 000 km (11 percent) of Primary urban extensions. The typical rural Primary route is a two-lane highway with "high-type" pavement, lane widths of 3.4 m or more, shoulders of adequate width and carrying about 2,500 vehicles a day (ADT) with the percent of ADT being trucks from 10 to 30 percent. The typical urban Primary route is four to six lanes of "high-type" pavement that are undivided and without access control. Typical ADT is about 20,000.

1.1.2.4 Secondary System

This system originally consisted of farm-to-market routes (established in 1944) and was realigned by Congress in 1976 so that only routes that functionally served as major rural collectors would be in the system. The secondary system comprises the largest single Federal-Aid System (644 000 km).

1.1.2.5 Urban System

Established by the Federal-Aid Highway Act of 1970 and consists of selected urban arterial and collector routes. The System carries 44 percent of urban travel on 7 percent of all urban mileage.

1.1.2.6 Special Federal Highway Systems

This system is composed of about 356 000 km of highways and roads in national forests, national parks, and other federally administered areas [1.4].

1.2 STATE PAVEMENT RELATED STATISTICS [1.6]

1.2.1 TOTAL CENTERLINE KILOMETERS — STREETS, ROADS, AND HIGHWAYS IN WASHINGTON STATE (AS OF 1997)

Agency	Paved	Unpaved	Total
WSDOT			
• Interstate	1 230		1 230
• Rural	8 770	10	8 780
• Urban	1 320		1 320
Total	11 320	10	11 330
County			
• Rural			56 780
• Urban			3 020
• Urban Local Streets			6 750
Total	41 720	24 830	66 550
City			
• Rural			3 660
• Urban			4 600
• Urban Local Streets			12 100
Total	19 140	1 220	20 360

Total (including mileage for other port, state, and federal roads) 128 320

(From WSDOT (Reference 1.6))

1.2.2 WSDOT MAINTENANCE STATISTICS (1990)

• Physical Features		
• Total lane kilometers	27 853	
• Total centerline kilometers[1]	11 222	
• Four or more lanes, divided	1 931	
• Bituminous (BST)	3 689	(33%)
• Asphalt Concrete (ACP)	6 711	(60%)
• Portland Cement Concrete (PCC)	774	(7%)
• Gravel	48	
• Regions	6	
• Maintenance areas	25	
• Major mountain passes	10	
• Bridges	3,056	
• Timber	138	
• Suspension	2	
• Floating	4	
• Moveable span	23	
• Number of major rest areas	38	
• Personnel		
• Maintenance Superintendents	30	
• Assistant Superintendents	12	
• Maintenance Supervisors	78	
• Maintenance Lead Technicians	164	
• Maintenance Technicians	601	
• Total Trades and Warehouse	282	
• Equipment Personnel	171	
• Seasonals	78	
• Temporaries	172	
• Total	1,588	
• Equipment and Material		
• Pieces of maintenance equipment	3,031	
• Dump trucks	440	
• Stockpile sites	210	
• Cubic meters of sand used per year	165 000	
• Sand used in mountains (m^3)	69 000	

[1]Routes "owned" and "maintained" by WSDOT vary a bit due to maintenance agreements with other agencies.

- Equipment and Material (continued)
  - Salt used (tonnes) 3 082
  - Other de-icers (tonnes)
    - Urea 295
    - Quiksalt 1 558
    - CG-90 865

- Financial
  - Total Maintenance $82,000,000
    - Snow and ice related $17,000,000
    - Snow and ice per lane-kilometer $600
    - Rest area maintenance $2,300,000

2. CURRENT PAVEMENT DESIGN PRACTICE

2.1 TYPES OF PAVEMENT STRUCTURES

Basically, all hard surfaced pavement types can be categorized into two groups, flexible and rigid. Flexible pavements are those which are surfaced with bituminous (or asphalt) materials in the surface (or wearing) course. These can be either in the form of pavement surfaces such as a bituminous surface course (BST) generally found on lower volume (or lower traffic) roads. On the other hand, AC surface courses are generally used on higher volume roads such as the Interstate highway network (refer to Figure 1.1). These types of pavements are called "flexible" since the total pavement structure "bends" or "deflects" due to traffic loads. Further, the flexible pavement structure is generally composed of several layers of materials which can accommodate this "flexing" (Figure 1.2). On the other hand, rigid pavements are composed of a PCC surface course (Figures 1.3 and 1.4). Such pavements are substantially "stiffer" than flexible pavements due to the high modulus of elasticity of the PCC material. Further, these pavements can have reinforcing steel which is generally used to reduce or eliminate "joints." PCC joints are a design detail which can vary greatly between the various State Highway Agencies (SHAs). Some states use joints (transverse across the lane) which are closely spaced (say every 3 to 4.6 m) and others use reinforcing steel to increase the allowable distance between joints to 12 m or more or to eliminate them completely. The current version of the American Association of State Highway and Transportation Officials (AASHTO) pavement design guide [1.3] lists design procedures for three rigid pavement types (based on joints or lack of):

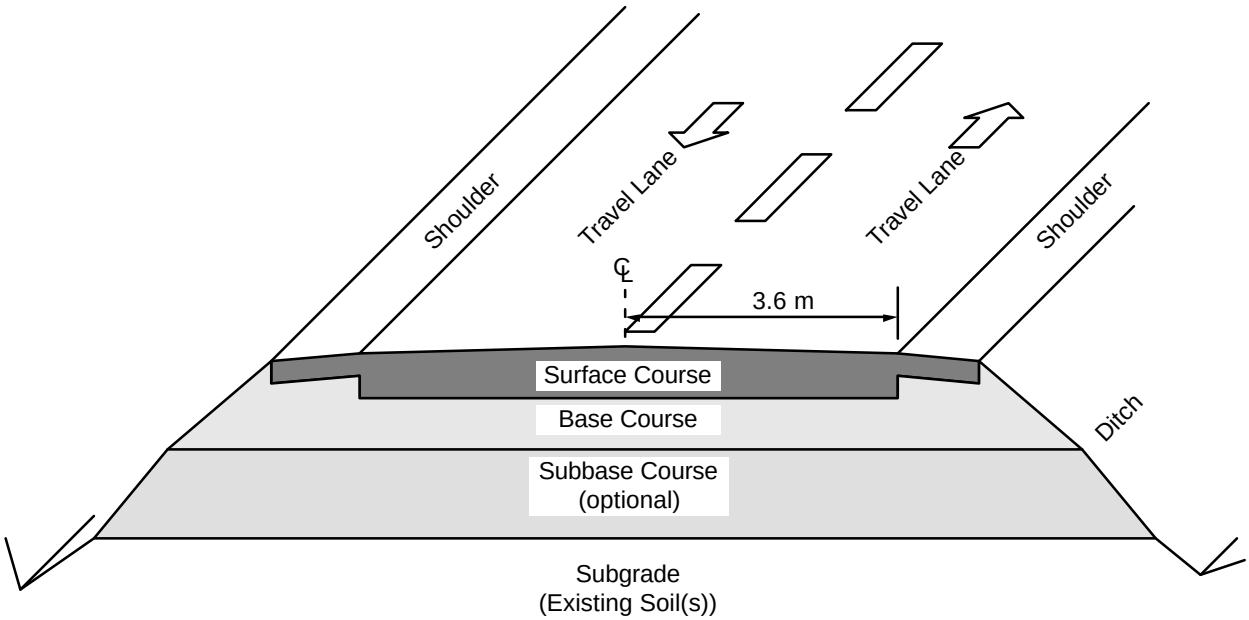


Figure 1.1. Typical Flexible Pavement Cross Section

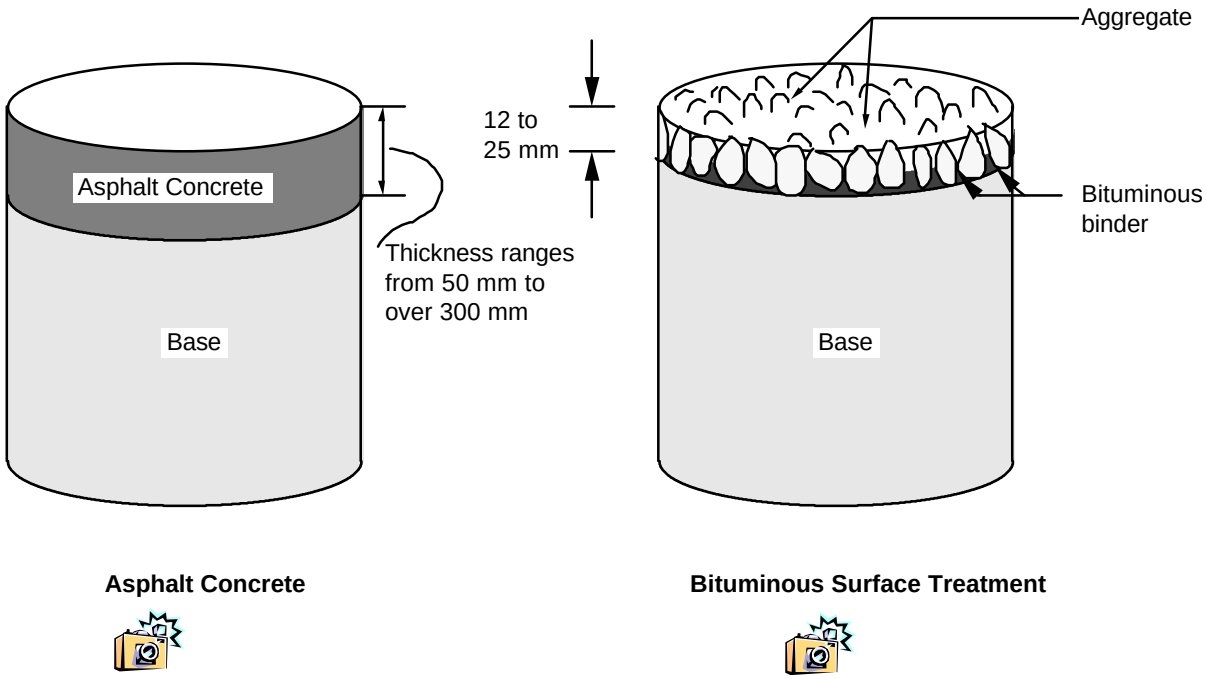


Figure 1.2. Types of Flexible Pavement Surface Courses

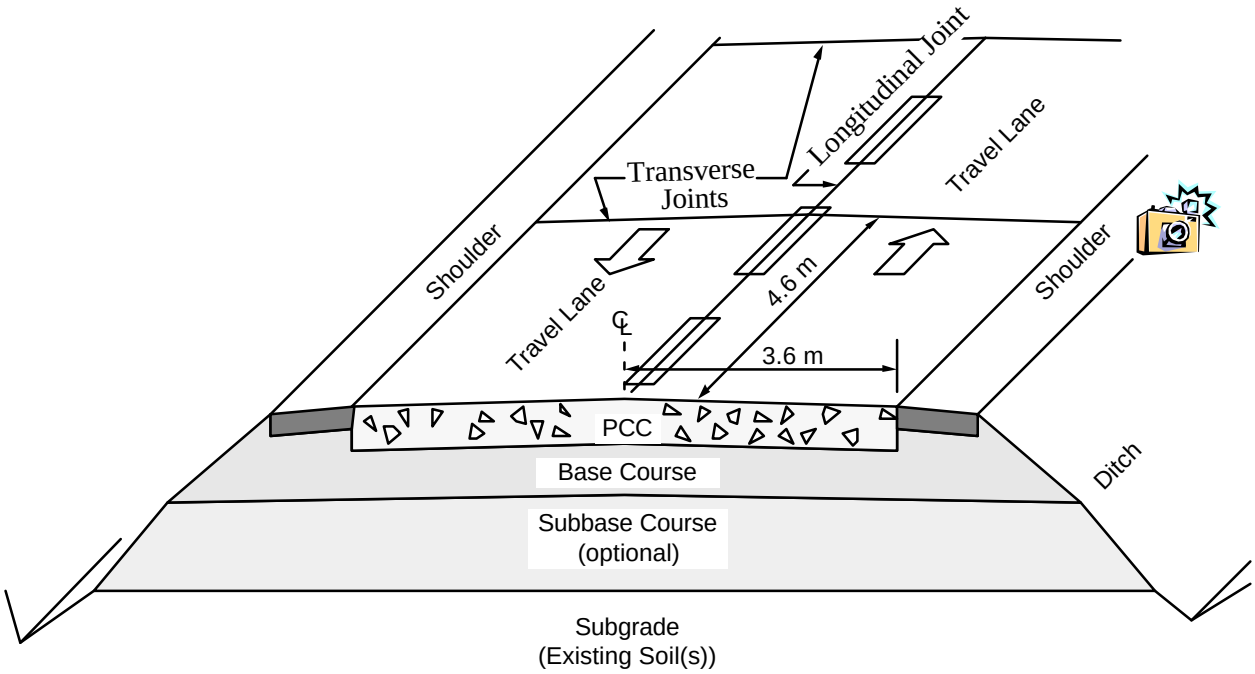


Figure 1.3. Typical Rigid Pavement Cross Section

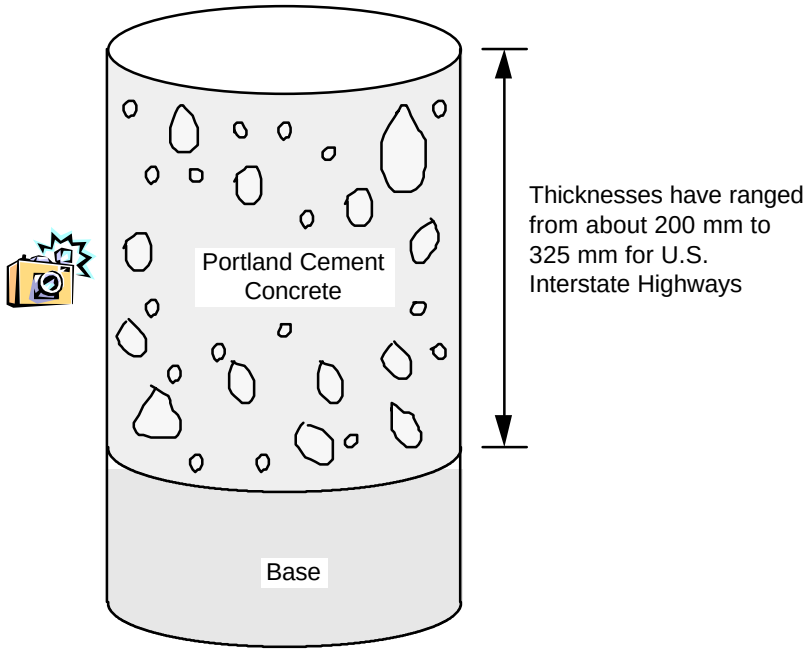


Figure 1.4. PCC Surface Courses

(a) Plain jointed concrete pavements (JCP)

No reinforcing steel is used with a small (or short) joint spacing (say 3 to 4.6 m). Dowel bars are sometimes used to aid load transfer between the slabs at transverse joints (JDCP).

(b) Jointed reinforced concrete pavement (JRCP)

Reinforcing steel is used to increase the joint spacing (say 12 m or more).

(c) Continuously reinforced concrete pavement (CRCP)

Reinforcing steel is used to eliminate transverse joints.

Figure 1.5 provides an overview of which states have used these different types of PCC pavement.

Overall, it must be somewhat confusing as to why one pavement is used versus another. Basically, SHAs previously and currently select pavement type either on a policy basis or economics or both. Flexible pavements generally can serve 10 to 15 years with relatively little maintenance. Rigid pavements, on the other hand, generally serve 20 to 30 years with little maintenance. Thus, it should come as no surprise that PCC pavements are often used in urban, high traffic areas. But, naturally, there are trade-offs. For example, when a flexible pavement requires major rehabilitation, the options are generally less expensive and quicker to perform than for rigid pavements.

To put a bit of perspective on pavement types, the following percentage of the Interstate system is flexible and rigid [1.5] as of 1988:

Interstate System Percentages			
Rural		Urban	
Flexible	Rigid	Flexible	Rigid
61%	39%	49%	51%

Total Interstate Mileage (rural and urban): 72 000 km

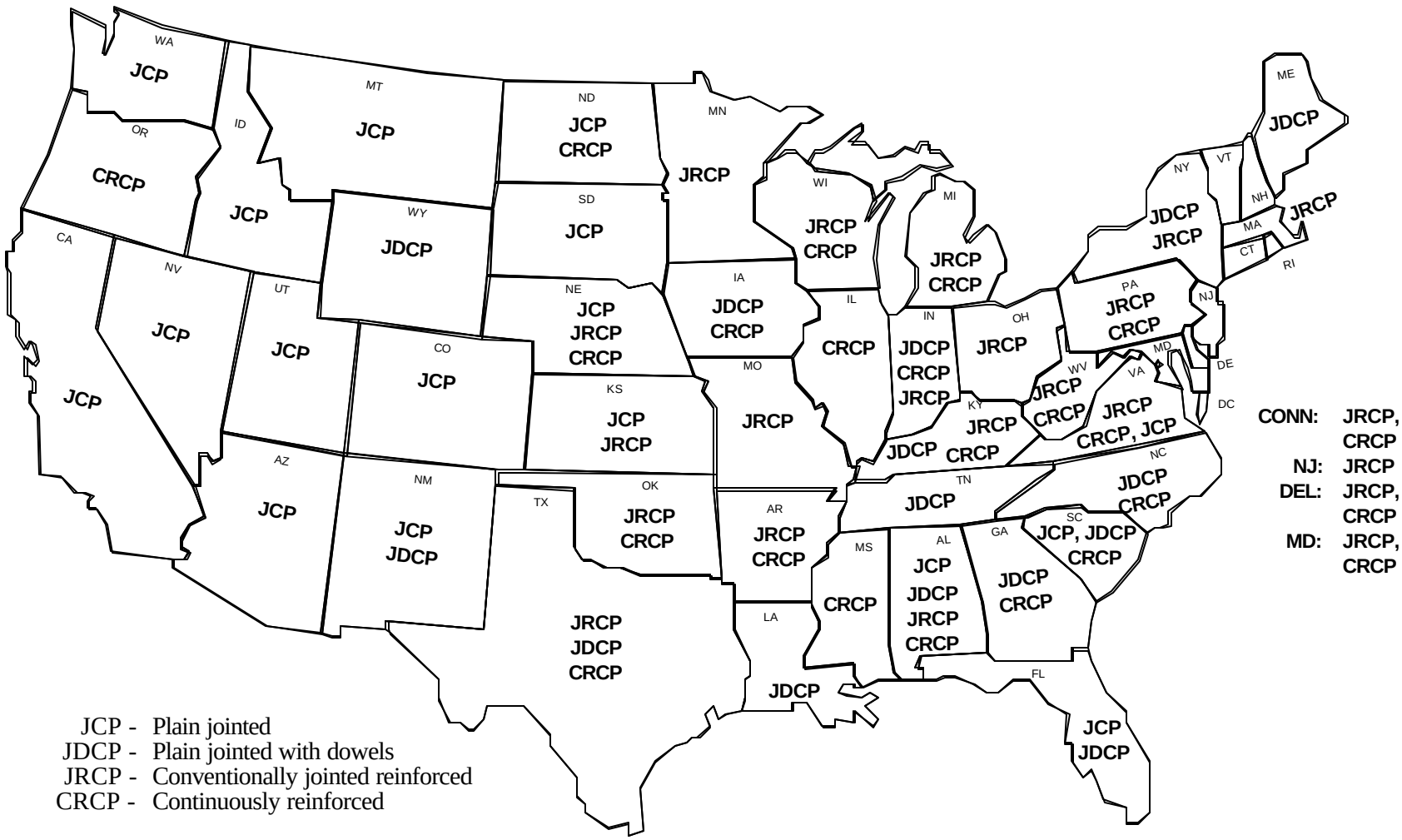


Figure 1.5. PCC Pavement Types Used in U.S. [1.9]

2.2 DEFINITIONS

2.2.1 SURFACE COURSE

2.2.1.1 Asphalt Concrete

(a) Increases ability of pavement structure to support loads

(b) Provides a waterproof covering for the pavement structure

(c) Composed of a mixture of aggregate (filler) and asphalt cement (binder) generally mixed at temperatures of about 150 °C

2.2.1.2 Surface Treatment

(a) Does little to increase ability of pavement to support loads
(b) Does provide a waterproof covering for the pavement structure and confinement of underlying base course

2.2.1.3 Portland Cement Concrete

(a) Distributes loads over wide area and provides a waterproof covering for the complete pavement structure

(b) Composed of aggregate and portland cement binder

2.2.2 BASE

2.2.2.1 General

Generally densified, graded aggregate (either stabilized or unstabilized) which helps to distribute load stresses and may provide drainage (if aggregate is open-graded). The AASHTO definition (AASHTO M146) is "the layer used in a pavement system to reinforce and protect the subgrade or subbase."

2.2.2.2 Stabilized Base

(a) Stabilizing agents typically used are asphalt cement or portland cement
(b) Can be designed to provide drainage

2.2.3 SUBBASE

Generally material that is of lower quality than the base course but better than the subgrade soils. The AASHTO definition (from AASHTO M146) is "the layer used in the pavement system between the subgrade and the base course."

2.2.3.1 Stabilized Subbase

Stabilizing agents typically used are asphalt cement, portland cement, or lime

2.2.4 SUBGRADE (ROADBED)

2.2.4.1 General

Generally composed of existing soils along the route of the pavement. AASHTO uses the term "roadbed soils."

2.2.4.2 AASHTO Terms

From AASHTO M146:

(a) Embankment Foundation: The material on which an embankment is placed.

(b) Embankment Fill: A raised structure of soil, soil-aggregate, or rock.

(c) Subgrade (Basement Soil): The prepared and compacted soil immediately below the pavement system and extending to such depth as will affect the structural design.

2.3 PAVEMENT DESIGN PROCEDURES

2.3.1 INTRODUCTION

There are probably almost as many ways (procedures) to design pavement structures as there are pavement type combinations. Most of these design procedures have evolved through local and/or national practice. Such procedures have been developed by agencies and companies as diverse as AASHTO, the U.S. Corps of Engineers, and the various SHAs, to energy companies such as the Chevron and Shell Oil Companies, and industry supported associations such as the Portland Cement Association and the Asphalt Institute.

The variation of design practice is no doubt a bit confusing since the pavement structures use can vary greatly in type and performance ("performance" is a word often

used to describe how "well" and "long" a pavement lasts during its "life"). This variability among the SHAs was illustrated by results from a 1987 questionnaire on typical "lives" for asphalt concrete surfaces [1.10], which showed an average life of 12.5 years nationally but ranged from a low of 7 years to a high of 21 years. Possibly, rather than viewing national pavement practice monolithically, it is better to think of each state as its own "country." Each individual SHA feels that it has unique materials, traffic, and other constraints which cause it to have its own, somewhat unique, practice; however, this trend is changing (as will be illustrated in Paragraph 2.4). Both builders and users of pavements in the USA must remember that pavements are built of the closest, available, appropriate materials (unlike the manufacturing of a bus, truck, or aircraft).

2.4 TRENDS IN STRUCTURAL DESIGN PRACTICE

The following information overviews national pavement structural design practice. WSDOT uses the AASHTO design methods for new or reconstructed pavements.

2.4.1 FLEXIBLE PAVEMENTS

A recent NCHRP Synthesis report [1.11] contains a 1991 survey of 48 SHAs and six Canadian provinces. This survey was used to overview state practice on

- Procedures to determine thickness,
- Layer compositions and configurations,
- Drainage treatments,
- Traffic characterization,
- Material characterization, and
- Miscellaneous design features.

Specifically, the agencies which responded the NCHRP survey were:

States

- Alabama
- Alaska
- Arizona
- Arkansas
- California
- Colorado
- Connecticut
- Delaware
- Florida
- Georgia
- Hawaii
- Idaho
- Illinois
- Iowa
- Kansas
- Kentucky
- Louisiana
- Maine
- Maryland
- Massachusetts
- Michigan
- Minnesota
- Missouri
- Montana
- Nebraska
- Nevada
- New Hampshire
- New Jersey
- New Mexico
- New York
- North Carolina
- North Dakota
- Oklahoma
- Ohio
- Oregon
- Pennsylvania
- Rhode Island
- South Carolina
- South Dakota
- Tennessee
- Texas
- Utah
- Vermont
- Virginia
- Washington
- West Virginia
- Wisconsin
- Wyoming

Canadian Provinces

- Alberta
- British Columbia
- Nova Scotia
- Ontario
- Quebec
- Saskatchewan

These results will be summarized in the following paragraphs. Paragraph 2.4.1.8 contains results from Linden et al. [1.10] on typical AC surface course lifes and distress modes at "failure."

2.4.1.1 Thickness

A summary of the agency responses is shown below. This shows that most states use empirical thickness design procedures and most of those use the *AASHTO Guide* (the 1972 and 1986 versions are used about equally). Four agencies indicated that they use a mechanistic-empirical design process, and five use The Asphalt Institute method (Manual Series No. 1) which is also mechanistically based.

A 1988 survey of SHAs [1.15] revealed that 18 used the *AASHTO Guide* for flexible pavement thickness determination and an additional 14 SHAs "partially" used the Guide (i.e., a total of 32 SHAs used the *AASHTO Guide* (partially or fully) for flexible pavements). This survey revealed that a number of the SHAs still used the 1972 *AASHTO Interim Guide* (similar to the 1991 survey results).

Design Procedure	No. of Agencies
• AASHTO 72	23
• AASHTO 86	20
• Asphalt Institute	5
• Mechanistic-Empirical	4
• Other	11

2.4.1.2 Design Period

Of the 54 agencies which responded to the questionnaire [1.11], the following summarizes their practice.

No. of Agencies	Design Period (years)
1	10
1	12
4	15
1	15-20
42	20
5	30

The design period is the time from original construction to a terminal condition for a pavement structure. AASHTO essentially defines design period, design life and performance period as being the same terms. Recall from Volume 1 of this Guide that WSDOT uses design periods of 20 to 40 years (depends on the highway classification and ESAL level).

2.4.1.3 Traffic Loading

For flexible pavement, 53 out of the 54 agencies use 80 kN equivalent single axle loads (ESALs) to characterize the effects of mixed traffic [1.11]. For rigid pavement, only six agencies do not use ESALs (New York and the five agencies which use the PCA design procedure).

2.4.1.4 Life Cycle Cost Analysis Periods

The survey [1.11] revealed that 35 SHAs currently use life cycle cost analysis (LCC) in the pavement type selection process. A summary of the responses relative to analysis periods is shown below. It is apparent that most SHAs use analysis periods of 30 to 40 years (27 agencies).

Analysis Period (years)	No. of Agencies
20	4
30	10
30-40	8
40	9
40-50	1
50	2
greater than 50	1
	35

2.4.1.5 Materials Characterization

The NCHRP Synthesis survey [1.11] can be used to show the most common tests used by SHAs for various flexible pavement materials (updated to include WSDOT, which apparently did not respond to this portion of the survey).

(a) Asphalt Concrete

No. of Agencies	Test/Process
40	Marshall Stability
7	Hveem Stability
4	Resilient Modulus

(b) Unstabilized Aggregate Base/Subbase

No. of Agencies	Test/Process
14	CBR
10	R-value
3	Elastic Modulus

The remainder of the responding agencies mostly use grading requirements. Thus, only 26 agencies stated that they use a specific test and associated strength or stiffness criterion.

(c) Subgrade

No. of Agencies	Test/Process
25	CBR
13	R-value
10	Elastic Modulus

(d) Asphalt Treated Permeable Material

No. of Agencies	Test/Process
1	CBR
3	R-value
2	Elastic Modulus or Resilient Modulus

2.4.1.6 Internal Drainage

According to the SHA survey [1.11], ten states and one Canadian province incorporate either unbound or treated open graded permeable materials into flexible pavement structures on a regular basis. Further, an additional 12 SHAs are doing same on an experimental basis.

Interestingly, most of the agencies surveyed which use untreated open graded layers assign an AASHTO layer coefficient ("a") of about 0.14 (same as a "dense" graded base). Further, SHAs using asphalt treated open graded layers use AASHTO layer coefficients ranging from a high of 0.30 to a low of 0.10 (average of about 0.20 for 14 agencies). More information will be presented on layer coefficients in SECTION 5.0 of this Volume.

2.4.1.7 Changes in Flexible Pavement Design Procedures

The summary below overviews the stated design changes that SHAs plan to make. Interestingly, 22 states plan to adopt mechanistic-empirical design procedures eventually. Further, 12 states plan to adopt resilient modulus testing and 10 states plan to adopt the 1986 *AASHTO Guide*.

Anticipated Changes	No. of Agencies
• Mechanistic Design	22
• Resilient Modulus Testing	12
• Adopt AASHTO 86	10
• Use Permeable Bases	6
• Use Life Cycle Costing	4
• Use FWD data in design	4

2.4.1.8 Pavement Life and Distress Modes

The typical "life" of an AC surfacing was obtained from 48 SHAs in a 1987 survey reported by Linden et al. [1.10]. "Life" was defined as the time between construction and the time when the next overlay or rehabilitation was needed.

Average Pavement Life (yrs)	Number of SHAs
7	1
8	2
9	3
10	10
10.5	1
11	1
12	6
12.5	5
13	1
13.5	3
15	7
17.5	3
20	2
21	1

The principal modes of failure noted by SHAs at the end of an AC surfacing life were:

Mode	Number of SHAs*
Fatigue Cracking	20
Rutting	14
Cracking (non-specific)	12
Thermal Cracking	6
Stripping	5
Weathering	4
Raveling	3
Reflective Cracking	2
Other Modes	10

*Multiple modes noted by some SHAs.

2.4.2 RIGID PAVEMENTS

2.4.2.1 Thickness

A summary of the agency responses for plain jointed concrete pavement is shown below. This reveals that most agencies (23) use the 1986 *AASHTO Guide* followed by 12 agencies which use the 1972 *AASHTO Guide*. Six agencies use the Portland Cement Association method. Thus, a total of 35 of the 54 agencies (approximately 65 percent) use AASHTO design procedures for rigid pavement design.

Design Procedure	No. of Agencies
• AASHTO 72	12
• AASHTO 86	23
• PCA	6
• Other	6

2.4.2.2 Design Period

Of the 44 agencies which responded to this question, their responses were [1.11]:

No. of Agencies	Design Period (years)
1	12-20
30	20
2	20-25
1	20-30
8	30
1	35

1 40

Recall from Volume 1 of this Guide that WSDOT uses design periods of 20 to 40 years (depends on the highway classification and ESAL level).

2.4.2.3 Traffic Loading

Refer to Paragraph 2.4.1.3.

2.4.2.4 Materials Characterization

(a) Portland Cement Concrete (PCC)

The vast majority of the agencies surveyed require PCC compressive strengths ranging from a high of 35.9 MPa (5,210 psi) to a low of 20.7 MPa (3,000 psi) (average of about 24.8 MPa (3,600 psi) for the 15 agencies which reported requirements). A majority of the agencies reported a standard cure period of 28 days. Only four agencies used something less (ranged from 3 to 14 days).

(b) Flexural Strength

The flexural strength requirements ranged from a high of 5.0 MPa (720 psi) to a low of 3.0 MPa (440 psi) (average of about 4.1 MPa (600 psi) for the 28 agencies which reported). Most agencies use a third point loading configuration.

2.4.2.5 Internal Drainage

According to the SHA survey [1.11], a number of states are using internal drainage in their rigid pavements. This specific information and more follows:

Drainage Feature	No. of Agencies
• Untreated Permeable Layer	17
• Daylighted	(10)
• Collector and Outlet Pipes	(12)
• Treated Permeable Layer	16
• Daylighted	(3)
• Collector and Outlet Pipes	(14)

2.4.2.6 Shoulder Design

Of 46 agencies reporting, about 60 percent (27 agencies) use "full depth" PCC shoulders. The remaining states use practices ranging from use of "standard sections" to designing the shoulder as a percentage of main lane traffic.

2.4.3 FLEXIBLE OVERLAYS ON FLEXIBLE PAVEMENT

The summary below overviews the responses from the agencies with regard to how they design AC overlays for flexible pavements (some agencies reported more than one procedure). Most use some type of deflection analysis (each of these terms will be more carefully defined in SECTION 7.0 of this Volume).

Overlay Design Procedure	No. of Agencies
• Judgment	26
• Standard Thickness	11
• Component Analysis	25
• Mechanistic Based	6
• Deflection Analysis	28
• AASHTO 86	10

To obtain deflection measurements most of the agencies use FWDs. A summary follows:

Deflection Measurement Devices	No. of Agencies
• Falling Weight Deflectometer	27
• Dynaflect	16
• Road Rater	7
• Benkelman Beam	7
• Other	2

The states were also asked about methods which are used for reflection crack mitigation. The following responses were obtained [1.11]:

Mitigation Method and Performance	No. of Agencies
• Increased AC Thickness	
• Good Performance	22
• Marginal Performance	18
• Poor Performance	3
• Fabric	
• Good Performance	13
• Marginal Performance	21
• Poor Performance	10

Mitigation Method and Performance	No. of Agencies
• Stress Absorbing Membrane Interlayer	
• Good Performance	9
• Marginal Performance	11
• Poor Performance	6
• Heater Scarification of Existing Surface	
• Good Performance	5
• Marginal Performance	10
• Poor Performance	6

2.4.4 FLEXIBLE OVERLAYS ON RIGID PAVEMENT

As shown below, AC overlays placed on rigid pavements are mostly designed by judgment or standard thicknesses (45 out of 77 responses).

Overlay Design Procedure	No. of Agencies
• Judgment	25
• Standard Thickness	20
• Component Analysis	14
• Mechanistic Based	2
• Deflection Analysis	7
• AASHTO 86	9

Some of the reported reflection crack control mitigation methods fall into the following categories:

Mitigation Method and Performance	No. of Agencies
• Increased AC Thickness	
• Good Performance	12
• Marginal Performance	17
• Poor Performance	6
• Fabric	
• Good Performance	3
• Marginal Performance	17
• Poor Performance	13
• Crack and Seat Overlay	
• Good Performance	9
• Poor Performance	2

Mitigation Method and Performance	No. of Agencies

• Saw and Seal Overlay	
• Good Performance	14
• Marginal Performance	0
• Poor Performance	1
• Stress Absorbing Membrane Interlayers	
• Good Performance	4
• Marginal Performance	8
• Poor Performance	2

2.4.5 RIGID OVERLAYS ON RIGID PAVEMENT

The summary below overviews practice among the SHAs with regard to how rigid overlays are designed for rigid pavements:

Overlay Design Procedure	No. of Agencies
• Judgment	6
• Standard Thickness	7
• Component Analysis	6
• Mechanistic Based	2
• Deflection Analysis	2
• AASHTO 86	7

2.4.6 RIGID PAVEMENT JOINTS

2.4.6.1 Joint Spacing

A slightly dated survey done by the Portland Cement Association (PCA) for the FHWA [1.9] in 1977 is helpful in understanding PCC joint practice and where WSDOT practice "fits." When the PCA did their survey, 18 SHAs used JCP without dowel bars. JDCP were used by 11 SHAs.

Generally, at that time, contraction joint spacing for JCP ranged from 4.6 to 7.6 m for slab thicknesses of 200 to 250 mm. The average joint spacing was 4.7 m.

The JDCP joint spacings ranged from 4.7 to 9.1 m for slab thicknesses ranging from 200 to 330 mm; however, thicknesses of 230 and 250 mm were the most frequently reported.

WSDOT contraction joint practice has evolved over time. During the 1940s - 1950s, straight (non-skewed) joints spaced 4.6 m apart were used. Some joints were skewed as early as 1964. A random spacing was adopted in 1966 which ranged from 4.3 to 5.6 m. The random spacing was reduced to a range of 2.7 to 4.3 m in 1967 (an average of 3.5

m). This is the same pattern still used today (actually a repeating spacing of 2.7, 3.0, 4.3 and 4.0 m).

2.4.6.2 Joint Distress

The PCA 1977 survey of 27 in-service pavements located in the states of Georgia, Iowa, Wisconsin, Oklahoma and California revealed for undoweled JCP (most of the pavements were 5 to 12 years old) an average faulting of the transverse joints of 2.2 mm. The faulting range varied from 0.0 to 6.9 mm. Generally, faults of 2.3 mm or less which are evenly distributed are considered tolerable [1.9].

In contrast, four projects surveyed by the PCA in Texas with doweled joints (9 to 12 years old) had no faulting.

Overall, WSDOT's experience with faulting is better in that the JCP will eventually fault but at considerably higher ages than reported by most states (25 to 35 years). If dowel bars had been used, it is unclear, at this time, as to how much improvement in joint performance WSDOT might have achieved. A good case can be made for using dowel bars in heavily trafficked locations (which is where PCC pavement is normally used).

2.4.6.3 Washington State Cities and Counties

Based on the results of a survey conducted by the Washington State Transportation Policy Plan Subcommittee on Weight Restrictions and Road Closures during 1994, the following flexible pavement design procedures were noted by the 12 counties and 9 cities which responded (some noted more than one procedure):

AASHTO	Counties	7
	Cities	5
Asphalt Institute	Counties	4
	Cities	1
Other	Counties	2
	Cities	3

2.5 TRENDS IN ASPHALT CONCRETE MIXTURE DESIGN PRACTICE

2.5.1 INTRODUCTION

This section will provide a brief overview of the various types of asphalt mixtures as used in the U.S. and Europe.

2.5.2 UNITED STATES

2.5.2.1 TYPES OF MIXTURES

Hot mix asphalt concrete as used in the U.S. can be divided into three categories with specific mixes associated with each [mostly from Ref. 1.13]:

Dense Graded	Open Graded	Gap Graded
• Asphalt Concrete • Large Stone • Sand Mix	• Asphalt Treated Permeable • Friction Course	• Stone Mastic Asphalt

(a) Asphalt Concrete

This mixture consists of a continuous aggregate grading which maximizes the aggregate particles and minimizes the required asphalt cement. This is typical of WSDOT Class A or Class B mix.

(b) Large Stone Hot Mix

The hot mix contains coarse aggregate with a nomial size of 25 mm or greater. Fundamentally, it is a dense mix and not unlike WSDOT Class E.

(c) Sand Mix

Basically, hot mix without coarse aggregate.

(d) Open Graded

The open graded friction coarse is generally placed as a thin layer with only modest compactive effort. This type of mix conforms to WSDOT Class D. The mix consists mostly of coarse aggregate with only a small amount of fine aggregate.

The open graded permeable is similar to WSDOT Class D but its purpose is to allow for rapid drainage of water from within a pavement structure. Generally, the

nomial size of the coarse aggregate is larger than for an open graded friction coarse.

(e) Gap Graded

This type of mix can be quite similar to an open graded mix except that the amount of fine aggregate is greater (more).

(f) Stone Mastic Asphalt (SMA)

SMA mixtures are more fully described in Paragraph 2.5.3.2(d). It is important to note that this mix is being trial tested in the U.S. starting in 1991 in Georgia. Interest in this type of mix increased after the September 1990 European Asphalt Study Tour with the goal of a more rut-resistant hot-mix (with the side benefit of greater studded tire wear resistance). This kind of mix has not been used in Washington state as of October 1992; however, a mix of presumably similar performance characteristics (WSDOT Modified Class D ("Oregon Class F")) was placed on I-5, MP 85.5 to MP 88.3, as a 46 mm overlay (PBA-6 binder) during August 1992.

2.5.2.2 MIXTURE DESIGN

In the U.S., approximately 454,000,000 metric tons (500,000,000 tons) of hot mix asphalt in its various forms was placed during 1988 [1.12]. Three mixture design procedures mostly are used by the SHAs:

Mixture Design Model	No. of SHAs
• Hveem	10
• Marshall	38
• Superpave	?

The Hveem process was mostly used by western SHAs (Washington, Oregon, Idaho, California, Nevada, Colorado, and Hawaii).

2.5.2.3 Asphalt Cement Grading System

There are five basic asphalt cement grading systems being used in the U.S. today. These are penetration, AC, AR, PBA, and PG systems.

The penetration system grades asphalt cement based on a standard penetration test (refer to ASTM D 946 for grading system and ASTM D 5 for the penetration test procedure). The basic penetration grades are (penetration at 25 °C, 100 g, 5 seconds):

Penetration Grade
40-50
60-70
85-100
120-150
200-300

The viscosity grading system (AC grade) is based on the "original" asphalt cement (unaged). The grading system is contained in ASTM D 3381 and is mostly based on the asphalt cement viscosity measured at 60 °C (refer to ASTM D 2171 for the test procedure). The unit of measure is Poises (P). The viscosity grades are:

Viscosity (P)	AC Grade
250 ± 50	AC - 2.5
500 ± 100	AC - 5
1000 ± 200	AC - 10
2000 ± 400	AC - 20
4000 ± 800	AC - 40

The aged residue (AR) grading system is similar to the AC grading system except the viscosity is measured on the asphalt cement after aging in the rolling thin film oven (RTFO). The viscosity test is the same as before (ASTM D 2171 at 60 °C). The AR system grades are:

Viscosity (P)	AR Grade
1000 ± 250	AR - 1000
2000 ± 500	AR - 2000
4000 ± 1000	AR - 4000
8000 ± 2000	AR - 8000
16000 ± 4000	AR - 16000

The performance based asphalt (PBA) binder grades were developed by the Pacific Coast Conference on Asphalt Specifications. During the 1992 construction season, WSDOT built a few selected jobs using asphalt cement tied to the new grading system. The grading system is currently based on traditional asphalt cement tests such as

- penetration (AASHTO T49: 4 °C, 200 g, 60 seconds on RTFO aged residue, and 25 °C, 100 g, 5 seconds on RFTO aged residue,

- ductility (AASHTO T51: 7.2 °C, 1 cm/minute on RTFO aged residue and 25 °C, 5 cm/minute on tilt oven residue, and

- absolute viscosity (AASHTO T202: 60 °C on original binder, RTFO aged residue and tilt oven residue.

The PBA system grades with associated absolute viscosities are:

Original Viscosity (P)	RTFO Viscosity (P)	PBA Grade
800+	2500 - 5000	PBA-1
1600+	8000 -	PBA-1a
1100+	2500 - 6000	PBA-2
1100+	3000+	PBA-3
2800+	14000 -	PBA-4
2000+	4000+	PBA-5
2000+	5000+	PBA-6
1100+	3000+	PBA-7

These PBA grades can be selected according to environmental guidelines as follows:

PBA Grade	Climate Characterization		
	General Description	Lowest Recorded Temperature	Highest Mean Monthly Temperature
PBA-1	Moderate	Above -12 °C	Below 32 °C
PBA-1a	Moderate	Above -12 °C	Below 32 °C
PBA-2	Moderate/Cold	Above -29 °C	Below 32 °C
PBA-3	Moderate/ Very Cold	Below -29 °C	Below 32 °C
PBA-4	Hot	Above -23 °C	Above 32 °C
PBA-5	Hot/Cold	Above -29 °C	Above 32 °C
PBA-6	Hot/Very Cold	Below -29 °C	Above 32 °C
PBA-7	Very Hot	Above -23 °C	Above 38 °C

As noted in Paragraph 2.5.2.1(f) and 2.5.4.1(c), the Modified Class D mix placed on I-5 during August 1992 used a PBA-6 binder (hot/very cold climate).

As part of the development of the Superpave mix design system (Superior PERforming Asphalt PAVEments), the Performance Graded (PG) binder system was developed.

This binder system is in the process of being widely adopted by SHAs including WSDOT. The overall goal is asphalt cement binders which are more likely to perform satisfactorily in the field.

The associated binder tests are:

- Rotational Viscometer (RV): Used to characterize the stiffness of the binder at 135°C.
- Dynamic Shear Rheometer (DSR): Used to characterize the viscoelastic properties of the binder at temperatures ranging form 4°C to 84°C.
- Bending Beam Rheometer (BBR): Used to characterize the low temperature stiffness of the binder (0°C to -36°C).
- Direct Tension Tester (DTT): Used to measure the tensile failure strain at low temperatures.
- Rolling Thin Film Oven (RTFO): Ages binders to simulate oxidative hardening that occurs during plant mixing and placing.
- Pressure Aging Vessel (PAV): Ages binders to simulate 5 to 10 years of service life.
- Cleveland Open Cup: Determines the flash point temperature (in essence, a safety test).

The PG binder grading system is based on the average seven day maximum pavement temperature (ranges from < 46°C to < 82°C) and a minimum design temperature (ranges from > -36°C to > -10°C). The binder tests (RV, DSR, BBR, DTT) specification values are fixed but the test temperatures are varied to meet the specification values. Currently, WSDOT will use three PG binders across the state; however, the grades can be increased for higher level service conditions. These three binders are PG 58-22 (Western Washington), PG 58-34 (Northcentral and Northeastern Washington), and PG 64-28 (Southcentral and Southeastern Washington). An FHWA produced table for PG binders is linked.

2.5.3 EUROPE

2.5.3.1 Introduction

Europe is especially interesting with regard to hot-mix asphalt. In general, they use a wider range of mixes some of which appear to have application in the U.S. and specifically Washington State. As a comparison, a few statistics compiled by van der Heide [1.14] are helpful. Table 1.1 provides a comparison between the U.S., the twelve countries of the European Community (EC) and the five countries of the European Free

Trade Association (EFTA). It appears that the only common factor is the annual consumption of hot mix per kilometer of road.

Table 1.1 Basic U.S. and European Statistics [modified from Ref. 1.14]

Parameter	U.S.	Europe		
		EC*	EFTA*	Total
Area (x 10^6 km^2)	9.4	2.3	1.1	3.4
Population (x 10^6)	250	315	35	350
Road Length (x 10^6 km)	6.25	2.62	0.48	3.10
Population/km Road	40	120	73	113
Hot Mix Use (x 10^6 tonnes)	480	205	35	240
Tons Hot Mix/Pop.	1.92	0.65	1.00	0.69
Tons Hot Mix/km Road	77	78	73	77
Tons Hot Mix/km^2 Area	51	91	32	71

*EC States

- Austria
- Belgium
- Denmark
- France
- Germany
- Hungary
- Ireland
- Italy
- Netherlands
- Portugal
- Spain
- UK

*EFTA States

- Sweden
- Norway
- Finland
- Switzerland
- Austria

2.5.3.2 Types of Mixtures

European mixture practice can be broken down into the following categories [1.14]:

- Asphalt concrete (wearing, binder and base courses)
- Porous (open graded) asphalt concrete (wearing course)
- Stone mastic asphalt concrete (wearing course)
- Hot rolled asphalt concrete (wearing course)
- Gusasphalt (wearing course)

Table 1.2 is used to overview the production of these mix types.

(a) Asphalt Concrete

This type of mix is much the same as used in the U.S. which results in a mix with relatively low air voids and as little binder as necessary [1.14]. The aggregates are dense graded and, in general, the sand size particles are manufactured. As shown in Table 1.2, this type of mix still dominates European practice.

(b) Porous (Open Graded) Asphalt Concrete

The use of porous asphalt concrete is increasing in Europe in order to provide enhanced safety and road noise reduction [1.14]. The void content is typically about 20 percent. Of the total tonnage of hot mix placed in Europe in 1990, about 1.6 percent was open graded (refer to Table 1.2).

(c) Gusasphalt

This mix is a mastic asphalt with essentially no coarse aggregate structure [1.14]. The binder is typically quite stiff (penetration grades 20-30 or 35-50). The mix has a low resistance against permanent deformation and, as such, is placed only in very thin layers (15-30 mm) Its usage in Europe represents only 0.5 percent of all hot mix production.

(d) Stone Mastic Asphalt Concrete

The initial development of stone mastic asphalt concrete (SMA) was in Germany in 1970 [1.14]. The goal was to find a replacement for Gusasphalt and retain the durability but improve on the permanent deformation characteristics. Essentially, SMA is an open graded mix filled with mastic asphalt. The open graded structure forms a strong aggregate "skeleton" which provides mix stability. The mastic asphalt forms a "stuffing" for the voids. This mastic is basically a mixture of filler and sand overfilled with asphalt cement. It typically costs about 20 percent more than traditional asphalt concrete in Europe [1.14]. Table 1.2 reveals that SMA constitutes about 1 percent of the hot mix market in Europe. The countries

Table 1.2 European Hot Mix Production in 1990 [after Ref. 1.14]

Country	1990 Production (x 10^6 metric tons)						
	Total	Asphalt Concrete	Porous Asphalt	Gusasphalt	Stone Mastic	Hot Rolled	Other
Austria	9.5	8.5	0.5	0.1	0.2	—	0.2
Belgium	4.2	3.8	0.4	—	—	—	trace
Denmark	3.2	3.1	0.1	0.1	0.1	—	—
Finland	6.5	5.2	—	0.1	0.5	—	0.7
France	38.0	36.0	1.4	—	—	—	0.6
Germany	41.3	39.2	0.4	0.4	0.4	—	0.9
Hungary	2.4	2.0	0.1	0.1	—	0.2	—
Ireland	1.0	0.8	—	—	—	0.2	—
Italy	45.5	—	—	—	—	—	—
Netherlands	7.5	6.8	0.4	trace	0.3	—	—
Norway	3.9	3.1	0.1	—	0.2	0.1	0.4
Portugal	5.3	5.2	—	—	—	—	0.1
Spain	23.0	—	—	—	—	—	—
Sweden	7.4	6.7	0.2	—	0.2	0.4	—
Switzerland	5.4	4.3	0.3	0.3	0.4	—	—
United Kingdom	36.5	28.0	—	—	—	8.0	0.5
	240.6	152.7*	3.9	1.1	2.3	8.9	3.4

*221.5 metric tons if all hot mix production in Italy and Spain added.

reporting the largest use are Finland, Germany, Switzerland, Netherlands, Norway and Sweden (total of 2,000,000 metric tons in 1990). Presumably, most of this production is used as a wearing course in heavy traffic applications.

According to Stuart [1.65], European SMA practice includes:

- Maximum aggregate sizes
  - Germany
    - 11.2 mm
    - 8 mm
    - 5 mm
  - Sweden
    - 16 mm
    - 11.3 mm
  - Norway
    - 16 mm
  - Finland
    - 20 mm
- Crushed aggregate

  Generally 100 percent of both the coarse and fine aggregates are crushed.
- Binder grades
  - Northern Europe

    Generally use 80-85 penetration grade asphalt based on 25 °C testing
  - Southern Europe

    Use 65 penetration grade asphalt based on 25 °C testing
- Stabilizing additives

  More than 85 percent of the SMA mixtures used in Sweden and Germany use fibers such as cellulose or rock wool (fibers are generally less than 5 mm in length).

- Mixture design
  - Use 50-blow Marshall. The flow and stability requirements are often not used.
  - Design parameters
    - Air voids

      Generally designed for 3 percent air voids (laboratory) within a tolerance of 2-4 percent. Below 2 percent expect flushing.
    - Binder contents (percent by weight of total mix)
      - Germany

        For maximum aggregate size of 8 mm or larger, a binder content range of 6.5 to 7.5 percent is used.
      - Sweden

        For maximum aggregate size of 11.3 mm, target binder content is 6.6 percent. For a maximum aggregate size of 16 mm, target binder content is 6.3 percent.
- Construction
  - Compaction

    The field air void levels are generally 3 to 4 percent or close to 100 percent of laboratory.
- Performance
  - Verbal reports suggest that SMA mixtures should last 20 to 40 percent longer than dense-graded mixtures; however, as of 1992, the majority of European SMA pavements are less than 8 years old.

(e) Hot Rolled Asphalt Concrete

Basically, this material has a sand structure with precoated "chippings" rolled in while still hot. This mix is mostly used in the United Kingdom where it is the primary mix for wearing courses [1.14].

2.5.3.3 Mixture Design

In Europe, van der Heide [1.14] identified three ways in which mixtures are designed:

- A system where no mix design procedure is specified, but only a mix type (defined by composition limits) and end-result requirements.
- A system, where relatively narrow composition limits are specified with a limited laboratory testing procedure.
- A system with relatively wide composition limits and an extensive laboratory testing procedure.

Typically, laboratory compaction and testing conforms to the Marshall method (or modification thereto); however, much unique practice exists in Europe (particularly France).

2.5.3.4 Asphalt Cement Grading System

A penetration based grading system is used in Europe.

2.5.4 WASHINGTON STATE

2.5.4.1 Types of Mixtures

(a) Class A

Dense graded hot mix leveling or wearing course with 90 percent fracture (at least one fractured face) on the coarse particles retained on a 2.0 mm sieve (otherwise, same as Class B). Its primary use is as a surface course for locations with high traffic levels or when the potential for rutting <u>within</u> the AC layer exists.

(b) Class B

Dense graded hot mix designated as a leveling or wearing course with a minimum of 75 percent crushed coarse particles (retained on a 2.0 mm sieve). The maximum aggregate size is 19.0 mm (100 percent passing); refer to Table 1.3 for specific requirements. This is WSDOT's "standard" surface course paving mix.

Table 1.3. WSDOT Asphalt Concrete and Asphalt Treated Base Requirements [from Ref. 1.16]

Sieve Size	Percent Passing							
	Class A and Class B	Class D	Mod. D	Class E	Class F	Class G	ATB	OGEAP
50.0 mm							100	
31.5 mm				100				
25.0 mm			99-100	90-100				
19.0 mm	100		85-96		100			100
16.0 mm				67-86				
12.5 mm	90-100	100	60-71	60-80	80-100	100	56-100	60-80
9.5 mm	75-90	97-100				97-100		
6.3 mm	55-75		17-31	40-62	45-78	60-88	40-78	5-35
4.75 mm		30-50						
2.36 mm		5-15						
2.00 mm	32-42		7-19	25-40	30-50	32-53	22-57	0-10
425 µm	11-24			10-23		11-24	8-32	
75 µm	3-7	2-5	1-6	2-9	2-8	3-7	2-9	0-2

(c) Class D and Modified D

An open graded hot mix wearing course. The open graded aggregate structure provides improved drainage of water from the pavement surface. The maximum coarse aggregate size is 12.5 mm (see Table 1.3). Provides reduced spray and noise but has higher construction costs and risks than for dense graded mixes. Requires a fog seal about every 5 years. Work performed in a Nordic sponsored research project (participating countries were Sweden, Denmark, and Norway) [1.74] showed that mixes similar to Class D reduced traffic noise emissions by 3 to 5 dB(A). This reduction is equivalent to a 50 percent reduction in traffic. Unfortunately, the effect of the noise reduction was soon lost (in some cases within less than a year) with such losses being greatly accelerated by studded tire use.

Typically, Class D mixes are placed as 18 mm (0.06 ft.) thick wearing courses. The performance of these mixes has varied on the state route system. For example, the South Central Region has had good success with Class D's generally lasting 12 to 15 years. Typically, the ADT on those South Central routes were less than 15,000 (both directions). In the Eastern Region, Class D wearing courses lasted about 11 years when the ADT was less than 15,000; however, noticeable rutting (wear actually) occurred after only 7 to 8 years. When the ADT was substantially above 15,000, shorter lives resulted (4 to 8 years in some cases). The bottomline is that clear evidence exists that this type of wearing course is susceptible to studded tire wear. Thus, this mix is seldom used on high ADT routes or where high studded tire use is expected.

A Modified Class D ("Oregon Class F") is being trial-tested by WSDOT. A major project was placed on northbound I-5, MP 85.5 to MP 88.3 (3 750 metric tons), during August 1992. This mix is classified as an open-graded mix whose gradation band is shown in Table 1.3. The mix is expected to exhibit superior rutting and studded tire wear resistance. On the project placed on I-5 and using a polymer modified asphalt binder (PBA-6), the cost was $44 per metric ton. Normal Class A or B hot-mix would typically cost $28 to $34 per metric ton using AR-4000W.

(d) Class E

Dense graded hot mix primarily intended for use as a base course layer. The maximum aggregate size is 31.5 mm (see Table 1.3). WSDOT "standard" <u>base</u> paving mix.

(e) Class F

Dense graded hot mix similar to Class B but a bit coarser (100 percent passing the 19.0 mm sieve — refer to Table 1.3). This is used in lieu of Class B where rock

sources cannot meet Class B grading requirements. Thus, it has a higher performance risk.

(f) Class G

Dense graded hot mix for thin lifts (say 30 mm or less). The maximum aggregate size is about 9.5 mm. (see Table 1.3). Gradation allows for use as thin, hot maintenance seals.

(g) ATB

Dense graded hot mix with a wide gradation band intended for use as a base course layer. The maximum aggregate size is 50.0 mm (see Table 1.3). Can be used as a District level option to facilitate construction. Very useful in widening applications.

(h) OGEAP (open graded emulsion asphalt pavement)

Open graded emulsion mix (19.0 to 2.0 mm) is used exclusively in eastern Washington generally as a substitute for Class B. It costs less than Class B and provides good fatigue performance. Construction difficulties can result in a rough ride. Further, issues pertaining to "sealing" the mix following construction need to be resolved.

(i) ATPB/CTPB

Asphalt treated or cement treated permeable bases are used as drainable bases. Mostly, ATPB or CTPB are used as the base layer directly under PCC slabs. The WSDOT gradation conforms to AASHTO Grading No. 57. Drainable bases are more fully described in SECTION 9.0, Paragraph 4 of this Volume.

(j) Bituminous Surface Treatments (BST)

A Bituminous Surface Treatment (BST) is an application of aggregate cover placed over on a application of binder. BST's are used as a surface course for an new pavement or to maintain an existing flexible asphalt pavement. About 26 percent of all WSDOT lane-kilometers are BST surfaced with the majority in the three eastern WSDOT Regions.

For BST construction, WSDOT typically uses cationic emulsified asphalt binders such as CRS-2, CMS-2 or CSS-1 rather than paving grades or cutbacks. Polymer emulsions have provided better chip retention with less windshield damage and less chip loss due to brooming. Polymer emulsions, such as CRS-2P, are recommended in western Washington mainly because of cooler ambient air temperatures that contribute to chip retention problems during construction. In eastern Washington, the "sticky" nature of this polymer emulsion has led to its use on entire contracts and not just in shady areas or mountain passes as was done in the past.

The four WSDOT BST types are:

- Class A
  - 19.0 to 12.5 mm aggregate on prime shot (first shot)
  - 12.5 to 6.3 mm aggregate over tack coat binder (second shot)
  - 6.3 to 0 mm choke aggregate
  - Optional fog seal
  - Primarily used for new construction and is know as a “triple shot BST.” May be used as a two shot seal on “distressed” pavement to reduce or delay reflection cracking of an AC overlay.
- Class B
  - 19.0 to 6.3 mm aggregate with single shot binder
  - 6.3 to 0 mm choke aggregate
  - Optional fog seal application
  - Standard seal coat with large rock resulting in a coarse texture.
- Class C
  - 12.5 to 6.3 mm aggregate with single shot binder
  - 6.3 to 0 mm choke aggregate
  - Optional fog seal application
  - Standard seal coat over “old” pavement.
  - BST Class C is sometimes modified to allow sealing of fatigued (alligator) cracked areas prior to an AC overlay. The rock application rate is reduced to 8.1 kg per square meter and the binder is shot in the 2.0 liters per square meter range.
- Class D
  - 9.5 to 2.0 mm aggregate with single shot binder

- 6.3 to 0 mm choke aggregate
- Optional fog seal application
- Standard seal coat with fine rock used over "old" pavement. Grading of rock does not require the use of choke stone. A Class D BST provides a quieter, smoother, and more uniform surface. Rock retention is good; windshield damage complaints are minimal. Recommended for areas with bicycle traffic.

- Preseal
  - 6.3 to 0 mm aggregate with single shot binder
  - Used to seal cracks in preparation for either a BST (say a Class B, C, or D) or AC overlay in areas with significant fatigue (alligator) cracking.

2.5.4.2 Mixture Design

WSDOT currently uses both the Hveem and Superpave asphalt concrete mixture design procedures. The following provides a short overview of that mix design system.

The Superpave (SUperior PERforming Asphalt PAVEments) system was developed during the Strategic Highway Research Program to allow the pavement community to design asphalt concrete mixtures that will perform better given the extremes of temperature and traffic loads. The Superpave system primarily addresses three pavement distress types: permanent deformation or rutting, low temperature cracking, and fatigue cracking. The system consists of two interrelated elements: asphalt binder selection and specification, and the volumetric mix design and analysis system.

The Superpave binder specification is a performance-based specification. It classifies binders into performance grades (PG) based on a range of climates and pavement temperatures. The first number indicates the high-temperature grade; the second number indicates the low-temperature grade. For example, a binder classified PG 58-22 would meet the required physical properties at pavement temperatures as high as 58°C and as low as -22°C (also refer to Paragraph 2.5.2.3). The mix designer selects a Superpave binder based on the climate in which the pavement will serve and the traffic it will bear.

WSDOT primarily uses three grades of binders with some adjustments being made for traffic. These primary grades are:

- Western Washington PG 58-22
- Northeastern Washington PG 58-34
- Southeastern Washington PG 64-28

The physical properties required for the binders are the same for all grades, but the temperature at which those properties are attained is determined by the specific climatic conditions at the paving location.

The Superpave mix design system is based on volumetric proportioning of the asphalt cement and aggregate materials. These materials are laboratory compacted using the Superpave gyratory compactor. The gyratory compactor kneads the mixture to fabricate test specimens by simulating traffic loading and construction compaction. The level or amount of compaction is dependent on the environmental conditions and traffic levels expected at the job site.

Specimens fabricated with the gyratory compactor are used to determine the volumetric properties (air voids, voids in the mineral aggregate, and voids filled with asphalt) of Superpave mixes. These properties, measured in the laboratory, indicate how well the mixture will perform in the field. The gyratory compactor is also well-suited for quality control/quality assurance at the job site to verify that the delivered asphalt mix meets the volumetric specifications. The FHWA has summarized relevant elements of the mix design system.

The Superpave mix design system also includes specifications and procedures for an aggregate quality test, aggregate angularity, as well as gradation requirements to insure that the mixture has a high degree of internal friction and thus high shear strength. The design goal is for a strong aggregate skeleton that will resist rutting yet include enough asphalt binder and voids to allow for adequate mix durability.

An eventual goal of the Superpave system is to allow prediction of mix field performance. The research associated with the required tests and models to allow for such predictions is underway as of July 1998 (funded by the FHWA).

WSDOT purchased its initial Superpave test equipment in 1993.

2.5.4.3 Bituminous Grading System

The bituminous materials currently used by WSDOT (July 1998) can be categorized as follows:

(a) Medium Curing (MC) Liquid Asphalt

- MC 70
- MC 250
- MC 800
- MC 3000

} The basis for these grades is kinematic viscosity at 60 °C (cSt).

(b) Rapid Curing (RC) Liquid Asphalt

- RC 70
- RC 250
- RC 800
- RC 3000

} The basis for these grades is also kinematic viscosity at 60 °C (cSt).

(c) Paving Asphalt

- AR 4000W
- AR 2000W

} The primary basis for these two grades is absolute viscosity at 60 °C (Poise).

- PBA-6
- PG 58-22
- PG 58-34
- PG 64-18

(d) Cationic Emulsified Asphalt

(i) Rapid Setting
- CRS-1
- CRS-2

(ii) Medium Setting
- CMS-2S
- CMS-2
- CMS-2h

(iii) Slow Setting
- CSS-1
- CSS-1h

(iv) Recycling Agent
- RA5
- RA25
- RA75
- RA250
- RA500

3. DESIGN OF PAVEMENTS — PAST TO PRESENT

3.1 INTRODUCTION

A brief view of how pavement design, construction and performance has evolved should help provide perspective on present and, possible, future practice. This short view into the past will start with the Romans, then move on to the Macadam and Telford era, then into the first 150 years of asphalt and portland cement concrete pavement. The evolution of pavement design will emphasize the U.S.A. and the U.K. a bit more than for other parts of the world.

3.2 ROMAN ROADS

In fairness, the Carthaginians are generally credited with being the first to construct and maintain a road system (about 600 B.C.) according to Tillson [1.32]. The Romans eventually decided that their neighbors across the Mediterranean were a bit of a threat to the empire destroying Carthage in 146 B.C. (The Carthage ruins are located in Tunisia (Northern Africa) next door to Algeria (on the left) and Libya (on the right — so to speak).) It is suggested that the Romans took up the practice of a military road system from the Carthaginians. It is estimated that the Romans built about 87 000 km of roads within their empire (about equal to the length of the U.S. Interstate system).

Apparently, there is no record of "traditional" roads in the U.K. prior to the Romans [1.17]. For the most part, the main Roman roads in the U.K. (total of about 4 100 km) was for military purposes in that they connected camps which were about 30 km apart (or about one day's march) [1.17, 1.22, 1.23]. Since the primary purpose of these roads was for foot soldiers, the roads were straight, but virtually without regard to grade. They generated high noise levels, were rough and labor intensive (slave and "statue" labor often used).

The Roman design for their primary U.K. roads generally consisted of four layers (top to bottom) as follows [1.17]:

- Summa Crusta (surfacing): Smooth, polygonal blocks bedded in underlying layer.

- Nucleus: A kind of base layer composed of gravel and sand with lime cement.
- Rudus: The third layer was composed of rubble masonry and smaller stones also set in lime mortar.
- Statumen: Two or three courses of flat stones set in lime mortar.

The total thickness was as much as 0.9 m and road widths of 4.3 m or less. An illustration of Roman pavement structure near Radstock, England, is shown as Figure 1.6. Roman roads in some countries have been up to 2.4 m thick. These structures had crowned (sloped) surfaces to enhance drainage and often incorporated ditches and/or underground drains.

As one might expect, Roman road building was varied to suit local conditions and materials — not unlike today actually. The Romans departed the U.K. about AD 406. Road design and construction languished for about 1,200 years thereafter.

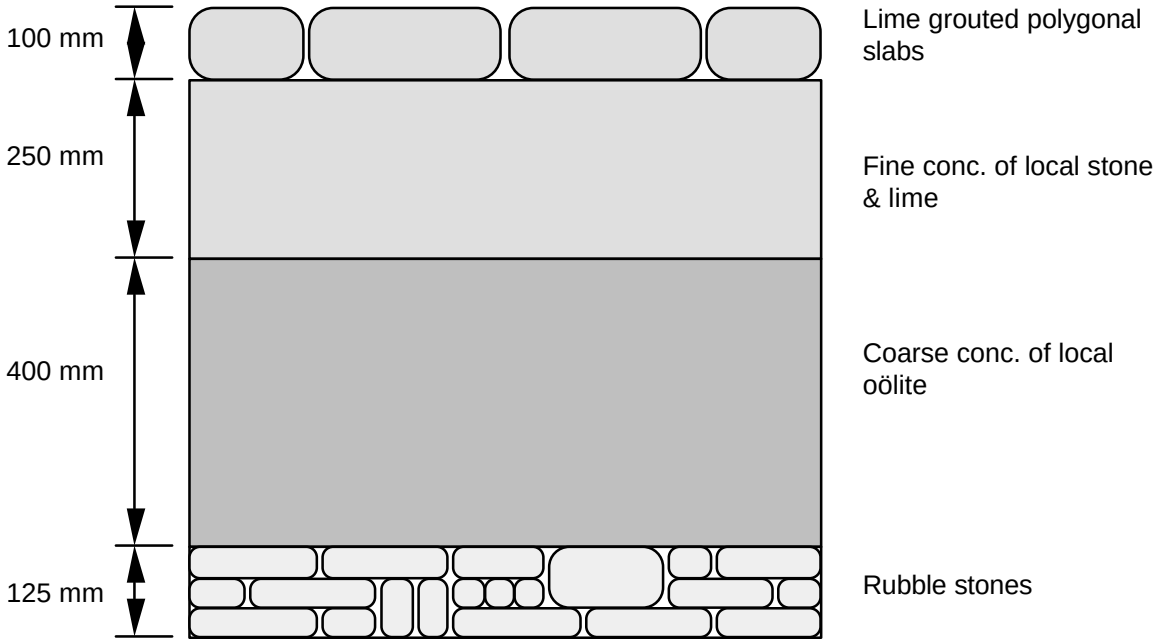


Figure 1.6. Roman Pavement Structure Near Radstock, England (southwestern part of U.K. near Bristol) [after Collins and Hart (1.17)]

Roman road construction was not inexpensive. Updated construction estimates of the Appian Way in Italy are about $2,000,000 per km (updated estimates following Rose [1.22] and Leger [1.23]).

The oldest known road in the U.K. is near the River Brue in southwestern England [1.30]. Actually, the "road" is a 6,000 year old walkway which was discovered in 1970 in a peat bog. The construction of the road coincides with the arrival of the first farmers in the U.K. about 4,000 B.C.

3.3 TELFORD AND MACADAM

3.3.1 TELFORD

Thomas Telford (born 1757) served his apprenticeship as a building mason [1.18]. Because of this, he extended his masonry knowledge to bridge building. During lean times, he carved grave-stones and other ornamental work (about 1780). Eventually, Telford became the "Surveyor of Public Works" for the county of Salop [1.18], thus turning his attention more to roads.

Telford attempted, where possible, to build roads on relatively flat grades (no more than 1 in 30) in order to reduce the number of horses needed to haul cargo. Further, the pavement section was about 350 to 450 mm in depth and generally specified in three layers. The bottom layer was comprised of large stones (100 mm) wide and 75 to 180 mm in depth) [1.17]. It is this specific layer which makes the Telford design unique [1.24]. On top of this were placed two layers of stones of 65 mm maximum size (about 150 to 250 mm total thickness) followed by a wearing course of gravel about 40 mm thick (refer to Figure 1.7). It was estimated that this system would support a load corresponding to 88 N/mm (500 lb per in. of width).

3.3.2 MACADAM

John Macadam (born 1756) observed that most of the "paved" U.K. roads in early 1800s were composed of rounded gravel [1.18]. He knew that angular aggregate over a well-compacted subgrade would perform substantially better. He used a sloped subgrade surface to improve drainage (unlike Telford who used a flat subgrade surface) on which he placed angular aggregate (hand-broken, maximum size 75 mm) in two layers for a total depth of about 200 mm [1.20]. On top of this, the wearing course was placed (about 50 mm thick with a maximum aggregate size of 25 mm) [1.17]. Macadam's reason for the 25 mm maximum aggregate size was to provide a "smooth" ride for wagon wheels. Thus, the total depth of a typical Macadam pavement was about 250 mm (refer to Figure 1.8). An interesting quote attributed to Macadam about allowable maximum aggregate sizes was that "no stone larger than will enter a man's mouth should go into a road" [1.20]. The largest permissible load for this type of design was estimated to be 158 N/mm (900 lb per in. width).

In 1815, Macadam was appointed "surveyor-general" of the Bristol roads and was now able to use his design on numerous projects. It proved successful enough that the term "macadamized" became a term for this type of pavement design and construction. The

term "macadam" is also used to indicate "broken stone" pavement [1.24]. By 1850, about 2 200 km of macadam type pavements were in use in the urban areas of the UK.

Macadam realized that the layers of broken stone would eventually become "bound" together by fines generated by traffic. With the introduction of the rock crusher, large mounds of stone dust and screenings were generated [1.20]. This resulted in use of such fines resulting in the more traditional dense graded base materials which in turn produced pavement thicknesses as thin as 100 to 150 mm.

The first macadam pavement in the U.S. was constructed in Maryland in 1823.

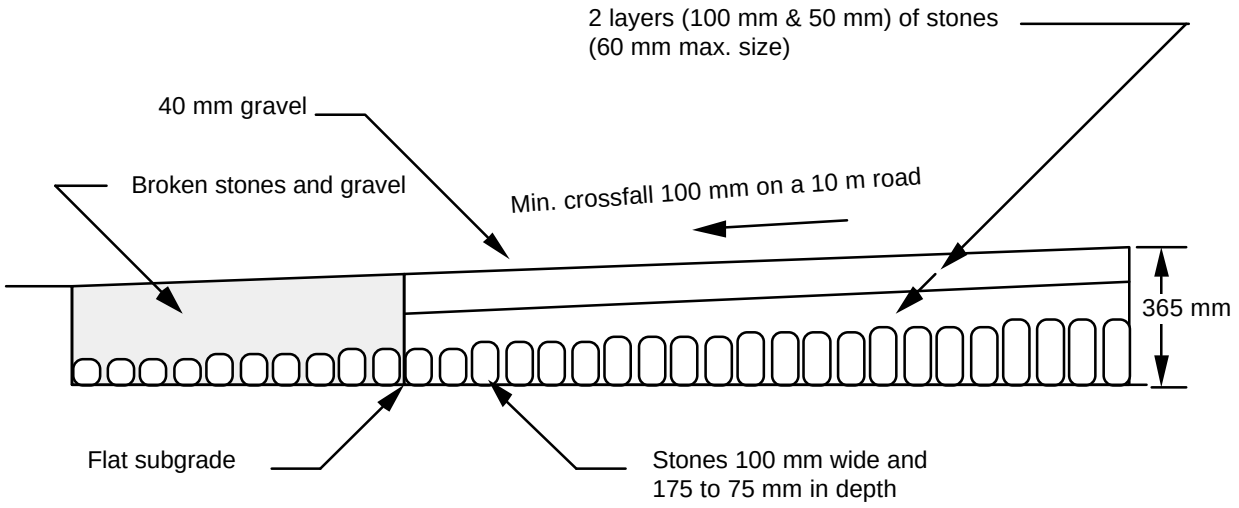


Figure 1.7. Typical Telford Pavement Structure
[after Collins and Hart (1.17)]

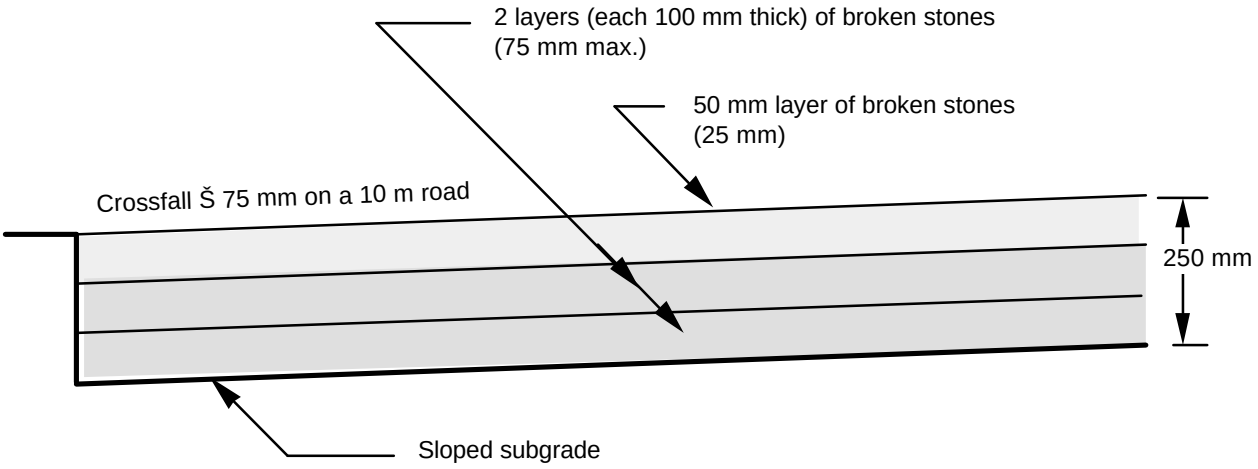


Figure 1.8. Typical Macadam Pavement Structure
[after Collins and Hart (1.18)]

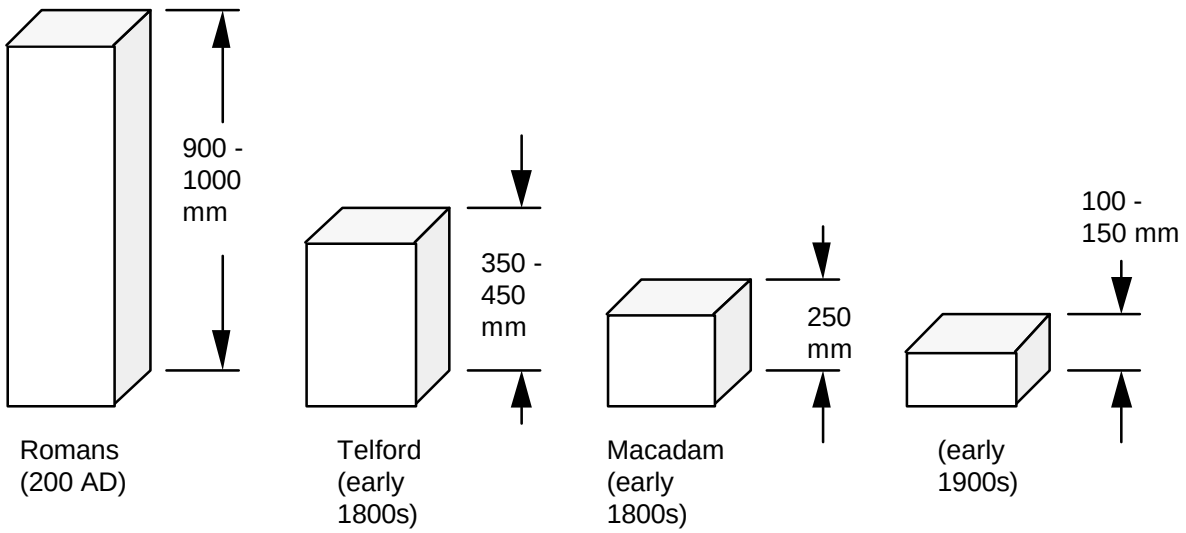


Figure 1.9. Comparison of Pavement Structures—Roman to Macadam Designs

3.4 EARLY THICKNESS TRENDS

Thus, we have seen pavement structures decrease from about 0.9 m (3 feet) for Roman designs to 350 to 450 mm for Telford designs, to about 250 mm for Macadam designs, to 100 mm at about the turn of the century (refer to Figure 1.9). (Naturally, the thinnest pavements were not always used.) The Massachusetts Highway Commission standard cross-section for macadam construction was 150 mm thick as reported by Gillette [1.20] in 1906. This thickness was also used on New York state roads at about that time.

Up to the early 1900s, the design emphasis was placed on the use of fixed standards occasionally modified for local soil conditions. Further, the need for more durable pavements was mandated by the changing vehicle fleet. The following partial quote by L. W. Page, Director of U. S. Office of Public Roads (contained in a 1907 report [1.26]) illustrates the problem:

> "...The existence of our macadam roads depends upon the retention of the road-dust formed by the wearing of the surface. But the action of rubber-tire motor-cars moving at high speed soon strips the macadam road of all fine material, the result being that the road soon disintegrates..."

3.5 EARLY BITUMINOUS PAVEMENT

3.5.1 TAR MACADAM

It appears that the first tar macadam pavement was placed outside of Nottingham (Lincoln Road) in 1848 [1.17, 1.19]. At that time, such pavements were considered suitable only for light traffic (not for urban streets). Coal tar (the binder) had been available in the U.K. from about 1800 as a residue from coal-gas lighting. Possibly this was one of the earliest efforts to recycle waste materials into a pavement!

Soon after the Nottingham project, tar macadam projects were built in Paris (1854) and Knoxville, Tennessee (1866) [1.19]. In 1871 in Washington, D.C., a "tar concrete" was extensively used. Sulfuric acid was used as a hardening agent and various materials such as sawdust, ashes, etc. were used in the mixture [1.19]. Over a seven-year period, 630,000 m^2 were placed. In part, due to lack of attention in specifying the tar, most of these streets failed within a few years of construction. This resulted in tar being discredited, thereby boosting the asphalt industry [1.19]. However, some of these tar-bound surface courses in Washington, D.C., survived substantially longer, about 30 years. For these mixes, the tar binder constituted about 6 percent by weight of the total mix (air voids of about 17 percent). Further, the aggregate was crushed with about 20 percent passing the No. 10 sieve. The wearing course was about 50 mm thick.

As a side note, the term "Tarmac" was a proprietary product in the U.K. in the early 1900s [1.19]. Actually it was a plant mixed material, but was applied to the road surface "cold." Tarmac consisted of crushed blast furnace slag coated with tar, pitch, portland cement and a resin.

3.5.2 SHEET ASPHALT

Sheet asphalt placed on a concrete base (foundation) became popular during the mid-1800s with the first such pavement of this type being built in Paris in 1858. The first such pavement placed in the U.S. was in Newark, New Jersey, in 1870. Baker [1.24] describes this pavement system as (1) a wearing course 40 to 50 mm thick composed of asphalt cement and sand, (2) a binder course (about 40 mm thick) composed of broken stone and asphalt cement, and finally, (3) a base layer of hydraulic cement concrete or pavement rubble (old granite blocks, bricks, etc.). Generally, the concrete layer was 100 mm thick for "light" traffic and 150 mm thick for "heavy" traffic [1.24]. The final thickness was based on the weight of the traffic, the strength of the concrete and the soil support.

3.5.3 BITULITHIC PAVEMENTS

In 1901 and 1903, Frederick J. Warren was issued patents for the early "hot mix" paving materials. A typical mix contained about 6 percent "bituminous cement" and graded aggregate proportioned for low air voids. Essentially, the maximum aggregate size was 75 mm ranging down to dust. The concept was to produce a mix which could use a

more "fluid" binder than used for sheet asphalt. This material became known as "Bitulithic."

More specifically, Warren was issued eight U.S. patents in 1903 which were [1.29]:

Patent 727,505	Pavement
Patent 727,506	Asphaltum and its Manufacture
Patent 727,507	Asphalt Composition and its Manufacture
Patent 727,508	Pavement
Patent 727,509	Method of Constructing and Laying Street Pavements or Roadways
Patent 727,510	Street Sheet Pavement or Roadway
Patent 727,511	Street Sheet Pavement or Roadway
Patent 727,512	Renewal of Old Street — Pavements

All of these patents were filed between May 16, 1901, and April 14, 1902. A review of the associated patent claims reveals that Warren, in effect, patented asphalt concrete, the asphalt binder, the construction of asphalt concrete surfaced streets and roads, and the overlayment of "old" streets. It seems that he covered "all the bases" with these patents. Some might argue that Patent 727,504 issued to Edward Walker of Warren, Pennsylvania, was actually just as important — the ice cream freezer! Warren's actual patent claim for 727,505 follows [1.29]:

> "A street-pavement mixture composed of mineral ingredients ranging in grades from three inches down to impalpable powder, from fifty to eight per cent, of such mineral ingredients lying between one-fourth inch and three inches in diameter, in combination with a bituminous binder."

In 1910 in Topeka, Kansas, a court ruling stated that asphalt concrete mixes containing 12.5 mm maximum size aggregate did not infringe on Warren's patent (727,505) [1.31]. Thus, asphalt concrete mixes thereafter were more oriented to the smaller maximum aggregate sizes. (A "fine aggregate" or "modified Topeka asphaltic concrete" is mentioned in a 1926 Standard Oil Co. of California publication [1.71]. The mixture consisted of 30 percent graded crushed rock or gravel (<u>all</u> passing 12.5 mm sieve, about 58 to 62 percent sand (material passing 2.0 mm and retained on 75 µm), 8 to 12 percent filler (material passing 75 µm sieve). This mixture required 7.5 to 9.5 percent asphalt cement.)

Warrenite-Bitulithic was invented in 1910 by a retired employee of Warren Brothers. It consisted of a thin, approximately 25 mm thick layer of sheet asphalt placed on top of the hot, uncompacted Bitulithic (Crawford [1.33]). The advantage of this system is that

the large aggregate of the Bitulithic mixes were not exposed directly to heavy, steel rimmed wheels which cracked the aggregate, eventually resulting in mix degradation.

By 1920, Warren's original patents had expired in the U.S. [1.34].

3.5.4 U.S. WEST COAST

A Standard Oil Co. of California publication [1.71] noted that the first asphalt concrete pavement on the West Coast was placed in 1894. Further between 1914 and 1926, 95 percent of such pavements were 125 mm or less in total thickness (75 to 87.5 mm of asphalt base and 37.5 to 50 mm of asphalt concrete surfacing). In 1921 for the states of Washington, Oregon, California, Nevada, and Arizona, approximately 770 lane-kilometers (3.6 m lanes) of asphalt concrete pavement were constructed. By 1925, the annual constructed lane-kilometers had increased to 1530.

A interesting quote from the 1926 Standard Oil Co. of California publication [1.71] seems to be repeated quite often today:

> "These pavements are today [1926] giving excellent service … notwithstanding the fact that these pavements were not originally designed for the severe conditions imposed by the heavy and fast modern motor traffic."

3.5.5 CITY OF SEATTLE

In the 1910 City of Seattle Standard Specifications, it was stated that an "asphalt pavement shall consist of:

- Wearing course 50 mm thick,
- Binder course 25 mm thick, and
- Concrete base.

Further, the asphalt cement in the asphalt paving mixture

> "shall be a solid, natural bitumen obtained from some natural deposit that has been in use in the paving industry for at least five (5) years."

By 1919, the Seattle Standard Plans and Specifications contained the same definition for asphalt pavement (AC over PCC) but the asphalt binder could be

> "either a solid natural bitumen or a California oil asphalt that has been in use in the paving industry for at least five (5) years."

3.5.6 EARLY ASPHALT CEMENTS

It should be noted at this point that an early source of asphalt binder in the U.S. was from Trinidad (near the coast of Venezuela). Trinidad supplied about 90 percent of all asphalt (worldwide) from 1875 to 1900 [1.24]. The asphalt was produced from a "lake" with a surface area of 465 000 m^2 (46.5 ha or 115 acres) and a depth of about 24 m. It was estimated by Tillson in 1900 [1.32] that this "lake" contained about 8 000 000 metric tons of "asphalt" (compare this against 1990 consumption in Europe and the U.S. of approximately 40 000 000 metric tons of asphalt binder). This asphalt, once free of water, was too "hard" to use in paving [1.25]. In fact, Trinidad lake asphalt, when loaded bulk into a ship, would fuse to the point that removal required chopping. It appears that the earliest use of asphalt binder in the U.S. was about 1874 for a project built in Washington, D.C. This binder was a combination of Trinidad lake asphalt and a flux oil distilled from crude oil. Without question, these early asphalt binders were quite variable making structural design somewhat challenging. By the 1880s, asphalt binders were regularly produced in California and by 1902 in Texas as well. It was not until 1907 that crude oil-based asphalt surpassed "natural" asphalt production [1.25].

An early Standard Oil Co. of California asphalt cement specification contained four original penetration ranges (at 25 °C) of 31-40, 41-50, 51-60, and 61-70. Thus, it appears that some of the early asphalt cements were a bit "harder" than generally used today.

3.6 EARLY PORTLAND CEMENT CONCRETE PAVEMENTS

At the turn of the century (1900), cements were categorized as "natural" or "artificial." Natural cements were made directly from specific rock. Artificial cement was made from proportioned ingredients and became known as "Portlands" (named after the natural limestone rock found on the Portland Bill, which is a small projection of land into the English Channel near Weymouth on the southern U.K. coast). The first true portland cement was produced in the U.K. about 1824 (actually portland cement was patented in 1824 by Joseph Aspdin, a bricklayer in Leeds, U.K.) and in the U.S. about 1865 (from Tillson [1.32]).

Interestingly, portland cement concrete (PCC) was not used as a pavement wearing course much until after about 1910 (Agg [1.35]); however, it was regularly used as a "stiff" base to support other wearing courses such as wooden blocks, bricks, cobble stones, etc. One likely reason for this was the lack of a consistent specification for the early cements. Tillson [1.32] in 1900 summarized over 109 separate specifications on portland cement fineness. Add to this confusion the fact that natural cements were widely used as well (about 60 percent of total cement consumption in 1898). Further, PCC hand mixing was still common in 1900 which undoubtedly restricted productivity and accurate proportioning. By 1900 (as reported by Tillson [1.32]), it was common to volumetrically proportion PCC as a 1:2:4 or 1:2:5 (cement : sand : coarse aggregate). A quote by Baker [1.37] in 1903 is of interest:

> "This form of road surface is not likely to come in general use owing to its cost and slipperiness when laid as are sidewalks, and its cost and lack of durability when laid like the foundation of a pavement."

Further light is shed on the use of PCC as a surfacing layer by Hubbard [1.19] in 1910:

> "If motor traffic alone were to be considered, a road built entirely of cement concrete might prove the most satisfactory and economical form for the future. For mixed traffic (horses and motor vehicles), however, such a road is by no means ideal and as, in spite of the increase in motor vehicles, the number of horse drawn vehicles does not seem to be decreasing..."

A 1916 report by Agg and McCullough [1.58] to the Iowa State Highway Commission further illustrates some of the issues which held back the use of PCC as a wearing course. These include

(a) Low compressive strengths.

(b) Lack of understanding about the need for longitudinal and transverse joints. A quote from the Agg and McCullough report is helpful:

> "Expansion and contraction may cause longitudinal cracks but generally, for the ordinary width pavement, placed on a flat subgrade, the liability of cracking from this cause, is remote."

(c) Poor inspection.

(d) Poorly prepared subgrade.

(e) Inadequate mix design, mixing, consolidation and curing of the PCC.

(f) Failure to achieve adequate strength gain prior to opening streets to traffic.

A survey reported by Agg and McCullough [1.58] of 50 states and municipalities presumably is representative of early 1900s practice. Some of the survey results include:

(a) Transverse joint spacing: About 50 percent of the agencies used spacings of either 7.6 m (18 percent) or 9.1 m (33 percent). The shortest spacing was 6.1 m (7 percent) and the maximum 30.5 m (2 percent).

(b) Angle of transverse joint with the centerline: Most agencies used a 90° joint but 4 percent used a skewed joint of 60°.

(c) On protection and testing of cement. These data illustrate the issue of evolving standards for tests and specifications

• American Society of Civil Engineers	22%
• American Society for Testing Materials	40%
• Above slightly modified	20%
• U.S. Bureau of Standards	7%
• Association for Standardizing Paving Specifications	4%

PCC was first used as a base for other wearing courses in London in 1872 (after Tillson [1.32]) and in New York in 1888 (base for stone surfacing). According to Collins and Hart [1.17], the first use of PCC as a wearing course was in Edinburgh, U.K., in 1872 and Grenoble, France, in 1876; however, one source stated that the first PCC pavement was placed in Inverness, Scotland, in 1865. The first PCC pavement in the U.S. was constructed in 1891 in Bellefontaine, Ohio. This pavement was only 3.0 m wide and 67.1 m long (probably what we would call a "test section" today). In 1909, in Wayne County, Michigan, a PCC highway system was constructed.

The Standard Specifications of the City of Seattle for 1910 and 1911 only mention the use of PCC as a base for other pavement surfaces (such as brick, brick block, asphalt, and stone). The specified mix proportions were set at 1:4:7 (cement : sand : gravel). The Standard Plans and Specifications for the City of Seattle dated May 9, 1919 did provide for PCC as a wearing course. The mix proportions were set as 1 : 2 : 3-1/2 (portland cement : sand : gravel). Standard thicknesses ranged from 125 to 230 mm with transverse expansion joints every 9.1 m (and at the end of a day's work).

The "lean" PCC base (for example, a 1:4:7 mix as used by Seattle) actually was beneficial for brick surfaced pavements (after Jaster et al. [1.60]):

(a) less affected by temperature changes,

(b) due to a lower tensile strength, the cracks which do form are more numerous but narrower,

(c) expansion joints are not needed (or recommended).

In the book *Highway Practice of the USA* [1.27], the following PCC mileages (centerline presumably) were listed for the U.S.:

Year	Total Mileage, km
1909	8
1910	40
1911	104
1912	504
⋮	⋮
1924	49 834

Clearly, not until after 1910 did PCC begin to receive widespread use.

By the 1930s, several PCC pavement design features began to evolve. First, typical slab thicknesses were about 200 mm with several states using a thickened edge design (maximum of about 225 mm). Second, it became clear that longitudinal joints should be used every 3.0 to 3.7 m and transverse contraction joints the same (Agg [1.35] and Bradbury [1.36]).

3.7 STRUCTURAL DESIGN

As stated earlier, pavement structural design was achieved by standards or catalogs from the 1800s well into the 1900s The focus of this review has been on those pavement types which led to asphalt pavement design. It should be noted that portland cement concrete (PCC), up until 1909, was largely used as a base or "foundation" layer for surface course materials such as bricks, wood blocks, sheet asphalt, etc. The year 1909 is noteworthy in the U.S. since this is the time which is generally used to mark the beginning of PCC as a structural wearing course (paving projects in Wayne County, Michigan [1.21]).

By 1949 [1.27], the following characteristics of PCC pavement design can be summarized:

(a) Thickness: The thickened edge cross section was less used since a better understanding of temperature induced stresses was made. Almost all slabs ranged in thickness from 150 to 250 mm.

(b) Joints: Expansion joints were not needed if contraction joints were frequently spaced (short panels). Before, they were provided every 9.1 to 30.5 m. Expansion joints were expensive to construct properly and difficult to maintain.

Contraction joints in plain PCC slabs were recommended every 4.6 to 6.1 m. Load transfer devices (dowel bars) were used by about one-half of the states.

(c) Reinforcement: The usual practice in 1949 was to place the reinforcement (if used) about 50 to 75 mm below the PCC surface. Generally, transverse joint spacings were increased to 18.3 to 30.5 m. Experiments were under way in New Jersey and Illinois examining the potential for continuously reinforced PCC (0.3 to 1.0 percent of the cross section area for the longitudinal reinforcement).

Up to the 1920s and 1930s, flexible pavement structural design was based mostly on experience based standards. Three reasons have been given for this [1.27]:

1. The methods appeared to provide "satisfactory" results for the then prevailing traffic.

2. Basic scientific knowledge was lacking.

3. The general use of stage construction did not readily lend itself to the evolving scientific methods of design.

In a 1939 paper, Gray [1.28] overviewed flexible pavement practice for the U.S. states. For hot-mix construction, the typical surface thickness was about 50 mm; however, this ranged from as thin as 19 mm to as thick as 125 mm. The base course types ranged from PCC to asphalt-treated base to gravel. The stabilized bases were generally 150 to 200 mm thick. A typical state such as Kansas applied 50 mm of asphalt concrete over a 175 mm thick PCC slab (urban areas). Thus, by about 50 years ago, pavement thicknesses were beginning to appear a bit closer to what is used today.

Table 1.4 provides an overview of a few of the flexible pavement design procedures available in 1949 [1.27]. One can quickly observe that these are mostly empirical relationships, many of which were influenced by the Boussinesq theory of load distribution (such as North Dakota [1.49]).

3.8 TRAFFIC

The kinds of loads pavements have been subjected to have varied significantly over the last 200 years. Typical early wagon wheel loads in the U.K. in 1809 are shown in Table 1.5. This information reveals loads per unit width exceeding that existing on heavy highway vehicles today. In the U.K. during the 1600 and 1700s, restrictive legislation was passed to adapt vehicles to the available pavements (which apparently were in very poor condition) [1.17]. An Act of 1751 prohibited wagons on turnpike roads with wheel rims less than 225 mm wide (a bit narrower than today's heavy truck tires). To suggest the speed of travel during that era, in the mid-1700s, the average speed between London and Bristol was about 13 kph (a distance of 187 km) (after Reference 1.55).

TABLE 1.4. A 1949 OVERVIEW OF FLEXIBLE PAVEMENT THICKNESS DESIGN PROCEDURES [modified after Reference 1.27]

Country/State	Basis for Thickness Design	Subgrade Tests	Traffic
New South Wales, Australia	Soil Character	• Grading • LL • PL • LS	Wheel Load • Normal: 9000 lb • Heavy: 13500 lb
California	Design Curves	CBR	Wheel Load
Canada	$t = 65 \log \frac{P}{s}$	• Plate Bearing • CBR • Cone Bearing • Triaxial Shear	Wheel Load
Colorado	Design Curves	• CBR • Grading • LL • PL	Volume
Kansas [25]	See Note 1	Triaxial (Modulus of Deformation)	Volume and Wheel Load
Michigan	Soil Survey Data	-	Volume
Minnesota	Design Curves	CBR	-
New Mexico	$t = 1.2 \sqrt{\frac{p}{s}} - 0.27 \sqrt{A}$	CBR	Wheel Load
North Carolina	$t = 1.2\, a \sqrt{\frac{p}{s} - 1}$	Plate Bearing	Wheel Load
North Dakota [28]	$t = \frac{65.7}{B^{0.388}}$	Cone Bearing	Wheel Load
Wyoming	Design Curves	CBR	Volume and Wheel Load

Notes 1. $t = \left[\sqrt{\left(\frac{3P}{2\pi CS}\right)^2 - a^2}\right]\left[\sqrt[3]{\frac{C}{C_p}}\right]$

2. Legend

t	=	pavement thickness
P	=	wheel load
s	=	subgrade bearing
S	=	assumed settlement of the pavement
C	=	subgrade modulus of deformation
C_p	=	pavement modulus of deformation
a	=	load contact radius
A	=	load contact area
p	=	unit capacity pressure
B	=	cone bearing value (psi)

TABLE 1.5. WEIGHT, HORSEPOWER, AND WHEEL CHARACTERISTICS OF VEHICLES ON UK ROADS - 1809 (after Collins and Hart [1.17])

Gross Weight (tons)	Number of Horses	Wheel Width (inches)	Weight per Wheel (pounds)	Wheel Weight per Unit Width (pounds/inches)
8	10	16	4480	280
6	8	9	3360	373
4.5	6	6	2520	420
3.5	4	3	1960	653
4	4	2	2240	1120

Wheels continued to evolve. In the early 1800s, Goodyear discovered the hot vulcanization of rubber which made it possible to manufacture solid rubber tires [1.17]. The impact of rubber tires can be illustrated by English law. Specifically, the Roads Act of 1920 increased the maximum empty weight of a "heavy traction engine" from 14 to 15.5 tons and, if equipped with rubber types instead of steel wheels, could travel at legal speeds of 19 kph instead of 8 kph [1.17]. The pneumatic tire was patented in the U.K. in 1845, but did not come into widespread use until about 1925 (80 years later). The earliest pneumatic tires had major shortcomings in strength and durability [1.71]. John Dunlap, U.K., greatly improved the design of the pneumatic tire in 1888 although only used on cycles until about 1900. The first pneumatic tire used on a motor vehicle was in 1895 in a race from Paris to Bordeaux [1.71].

From a Goodyear publication, the following quote may be of interest [1.71]:

> "Actually, many historians have given Goodyear credit for launching the interstate trucking industry in 1917. Of course, there were trucks before then, but virtually all trucking was confined to intercity hauling because of the solid tires of the day which gave a bone-rattling ride and consequent slow speeds.
>
> "In 1917, Goodyear was convinced that trucks would be tremendously more efficient if they rolled on pneumatic tires. When we couldn't find a trucking company willing to take the gamble, we launched our own line — the Wingfoot Express — with an initial route from Akron to Boston and back ….
>
> "Prophetically, Frank Seiberling, who was Goodyear's first president, told the U.S. Chamber of Commerce meeting in Chicago just five days after

> that first 23-day Wingfoot Express run to Boston that, 'The introduction of the motor truck into our commercial life sounds the death knell for the short line railroad.'
>
> "While the Wingfoot Express was in operation during the ensuing 12 years [1917-1929], Goodyear pioneered in the development of several important trucking concepts including tandem axles, dual wheel assemblies, and the fifth wheel system for trailering. All of this came about because Goodyear was fighting to prove the value of pneumatic tires for trucks.
>
> "In 1926 the sale of pneumatic truck tires topped solids for the first time."

A quote from Gray [1.28] in 1939 is helpful:

> "The development of the balloon tire has so reduced load concentrations that even though traffic volumes have greatly increased, the damaging effects of impact formerly experienced have practically disappeared."

An early effort to examine tire contact pressures was provided by Eckels [1.48] in a 1928 paper. Measurements were made on solid rubber, cushion and pneumatic tires. The solid rubber tires had measured contact pressures up to 1050 kPa. The pneumatic tires had contact pressures of about 700 to 800 kPa for a tire inflation pressure of 620 kPa. Data summarized by Baron [1.51] for aircraft in the early 1940s suggests inflation pressures of no more than 350 to 590 kPa.

As is illustrated by the Public Roads Administration in 1949 [1.27], the traffic input into the evolving pavement design procedures of the day varied. This included the use of wheel load magnitudes ranging to the early concept of equivalent wheel loads.

Hveem and Carmany [1.38] in a 1948 paper showed that the thickness of surfacing and base was proportional to the tire pressure and the number of load applications. Traffic was characterized in the proposed California thickness design procedure by the following term:

$$p\sqrt{a}\ \log r$$

where:
- p = effective tire pressure (psi),
- a = effective tire area (in^2), and
- r = number of load repetitions.

This concept evolved into equivalent wheel loads. This work was based on the relative destructive effects of wheel load groups ranging from 20 to 42 kN (4500 to 9500 lb). Equivalent Wheel Load (EWL) constants were developed to more easily summarize the available truck traffic (classified by the number of axles). An example of these 22 kN (5000 lb) EWL constants are shown in Table 1.6. Hveem and Carmany [1.38] found the term 6 log EWL could replace $p\sqrt{a}\ \log r$ in their thickness design equation.

TABLE 1.6. ILLUSTRATION OF EQUIVALENT WHEEL LOADS (from Hveem and Carmany [1.38])

No. of Axles	EWL Constants	Current Average Daily Commercial Vehicles	Product of Columns 1 and 2
2	300	774	232,000
3	700	212	148,400
4	1400	68	95,200
5	2100	118	247,800
6	1600	112	179,200
		Total Annual Design EWLs =	902,800

Eventually, in the late 1950s and early 1960s, the AASHO Road Test produced the equivalent single axle load concept. Axle load equivalency has been one of the most widely adopted results of the AASHO Road Test. A variety of equivalency factors can be used depending on the pavement section which is defined by a Structural Number and the terminal serviceability index (for flexible pavements). The basic equation used for calculating AASHTO equivalency factors [after References 1.39 and 1.54]:

$$\frac{W_x}{W_{18}} = \left[\frac{L_{18} + L_2}{L_x + L_2}\right]^{4.79} \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] [L_2]^{4.33}$$

where W = axle applications

$\frac{W_x}{W_{18}}$ = inverse of equivalency factors (where W_{18} = number of 18,000 lb single axle loads

L_x = axle load

L_2 = code for axle configuration

1 = single axle
2 = tandem axle
3 = tridem axle (added in 1986 Guide)

$G = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right)$ a function of the ratio of loss in serviceability at time t to the potential loss taken at a point where $p_t = 1.5$

p_t = "Terminal" serviceability index

$$\beta = 0.4 + \frac{0.081(L_x + L_2)^{3.23}}{(SN+1)^{5.19}\, L_2^{3.23}}$$ = function which determines the general shape of serviceability trend with increasing axle load applications

SN = structural number (formerly called a "thickness index")

3.9 MATERIALS

Some of the earliest recorded information about materials which could be (and were) used in pavements concern hydraulic cement and the Romans; however, in fairness, the earliest known use of hydraulic lime was in Syria about 6,500 B.C. (over 6,000 years before the Romans — give or take a few hundred years) (Brown [1.59]). The Romans "discovered" that grinding volcanic tuff with powdered hydraulic lime produced a hydraulic cement ("hydraulic" in that it hardened in the presence of water). The hydraulic lime was produced by heating limestone above 850 °C thereby driving off CO_2 and converting the limestone to CaO. The first known use of hydraulic cement by the Romans occurred at about 120 B.C. (in Rome oddly enough). The "best" variety of volcanic tuff was found near the town of Pozzuoli (near Naples on the southwestern coast of Italy) and the material acquired the name of pozzolana. Further, the Romans learned a bit about the use of other additives such as blood, lard and milk. Apparently, blood (hemoglobin actually) is an effective air-entraining agent and plasticizer (given the mild Mediterranean climate the primary use was likely for workability).

With regard to material testing, the year 1898 was significant: the American Society for Testing and Materials (ASTM) was established.

Some of the early paving materials have been previously discussed; however, it might be interesting to compare a few prices. For example, refined Trinidad Lake asphalt in 1893 cost about $27 to $36 per metric ton at the Port of New York. To adjust these costs to contemporary prices one must multiply by at least a factor of ten. Thus, the same cost today would be about $270 to $360 per metric ton (or about twice as much as an asphalt cement binder today) without the product refinements. (Note: original asphalt prices from Byrne [1.56] in 1896.)

With regard to PCC compressive strength, Byrne [1.56] reported in 1896 that PCC air cured for six months for a St. Louis bridge had a compressive strength of 8.3 MPa. Other compressive strengths he reported (presumably pre-1890s) ranged from 1.4 MPa (1 month age) to 9.7 MPa (1 year age). Compare these results to contemporary compressive strengths of 34.5 to 137.9 MPa. Thus, in 100 years, PCC strength has increased about 100 times (ratio of extreme low to extreme high).

Byrne also provided some cost data on wood block pavements in London. During the late 1800s, the average cost was estimated to be about $0.70 per m^2 per year. In terms

of today's costs, this would be about $7.00 per m^2 per year or $26,000 per lane-kilometer per year. If all of the WSDOT highway network (say, 28,800 lane-km) was maintained in a similar cost manner, it would cost about $0.75 billion (over five times as much as today). Add to this the absolutely unacceptable requirement for wood blocks (about 1 billion board-feet per year — assuming an average life of seven years — probably very optimistic with today's traffic).

It was estimated by Boorman [1.57] in his 1908 book that about 33 000 000 m^2 of "standard asphalt pavement" existed in the U.S. and 85 percent of this used Trinidad Lake asphalt as the binder. This roughly translates to about 2 700 000 metric tons of equivalent hot mix (or about three times the annual amount of hot mix purchased by WSDOT).

Material characterization up to the 1930s tended to focus on basic material parameters such as liquid limit, plasticity index and gradation. The strength or bearing capacity of unstabilized materials was usually discussed in terms of cohesion and friction angle (as illustrated by Collins and Hart [1.17]). Goldbeck [1.40], in his proposed design method, characterized the subgrade soil in terms of "subgrade supporting value" in units of pounds per square inch (ranging from 0 to 90 psi). The recommended test was to use a bearing block directly on a compacted sample. The test result was a plot of bearing stress versus indentation of the block into the sample. An indentation of 12.5 mm was used as a target value for obtaining the "bearing capacity" or "subgrade supporting value."

The papers by Burggraf [1.41, 1.42, 1.43] in 1938, 1939, and 1940 summarized the results of extensive field shear tests made on flexible pavement layers. These shear tests were conducted with a small, portable apparatus which used a parabolic plate in contact with the soil being tested [1.41]. As an illustration of the results from this test, the measured shear strength of cohesive subgrade soils were evaluated beneath stabilized base courses. The following classes were reported [1.42]:

Shear Strength, kPa (psi)	Service Behavior
Over 200 (Over 30)	Excellent
140-200 (20-30)	Good
100-140 (15-20)	Good to Fair
70-100 (10-15)	Fair to Poor
Below 70 (Below 10)	Very Poor

Work by Hubbard and Field [1.43] for The Asphalt Institute tended to downplay the importance of subgrade bearing values, and instead, emphasized the importance of surface deflection testing as a way to characterize the overall pavement structure.

The one material characterization test which had a tremendous impact on design and to some extent still does, was the development of the California Bearing Ratio (CBR).

Porter of the California Division of Highways stated in a 1942 Highway Research Board paper [1.50]:

> "The bearing ratio test was devised in 1929 in an attempt to eliminate some of the objections to field loading tests and to provide a quick method for comparing local base and subbase materials."

Some of the positive aspects of the CBR laboratory test as listed by Porter:

1. Improvement over field static load tests which must overcome "consolidation deformation" since the CBR specimen is compacted to a density expected in the field. California's attempts to use static field bearing tests were judged as unsuccessful; hence, the emphasis on correlating pavement performance with a straightforward laboratory test.

2. The soaking of the laboratory specimens with a surcharge (which represents the weight of the pavement) permits the material to swell and reach the adverse state of moisture which can exist in the field.

3. The penetration test determines the material's resistance to lateral displacement resulting in a combined measure of the influence of cohesion and internal friction.

4. The test provides a quick method of comparing base and subgrade materials.

5. By investigating and testing the associated materials, an empirical relationship can be established between CBR values, pavement thickness, and performance.

Figure 1.10 (from Porter's paper [1.50]) shows typical bearing values (psi) versus penetration (in.) for various materials ranging from "very poor subgrade" (CBR up to 5) to "good crushed rock bases" (CBR of 100). The CBRs are in terms of percentages since the bearing value is divided by 1,000 psi (0.1 penetration) or 1,500 psi (0.2 in. penetration) which represents the bearing value of a crushed rock material (refer to "standard curve 100%" in Figure 1.10).

Figure 1.11 (also from Porter [1.50]) shows the thickness design curves developed from 12 years of CBR tests associated with both failed and good performing pavements on the California highway system. Curves "A" and "B" show the minimum pavement thicknesses for light and medium-heavy traffic. The additional curves were added by U.S. Army Corps of Engineers for the design of flexible airfield pavements. The tire inflation pressure for the traffic was 400 kPa. The design curves also embody the

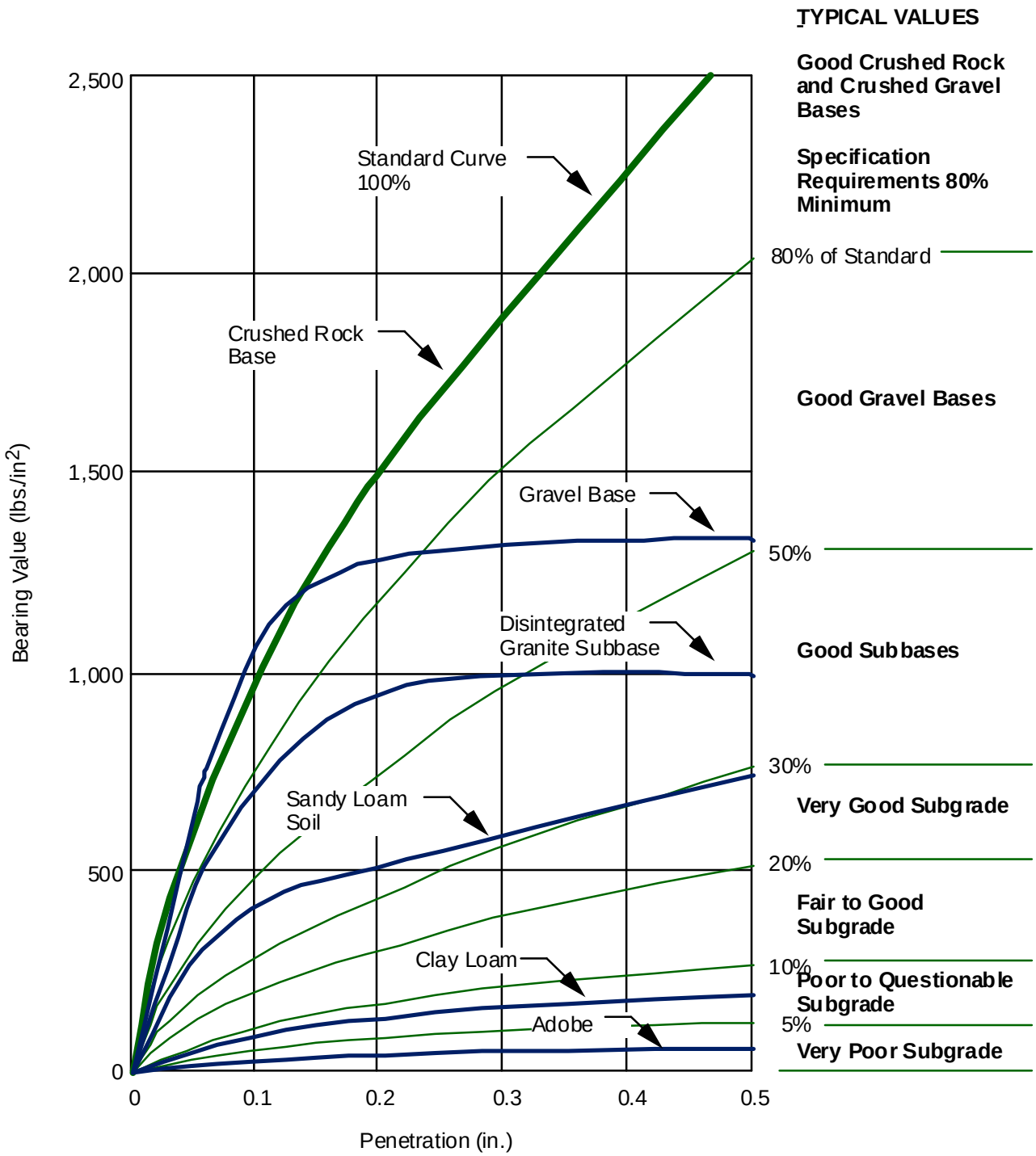


Figure 1.10. Relationship between CBR and Bearing Value (after Porter [1.50])

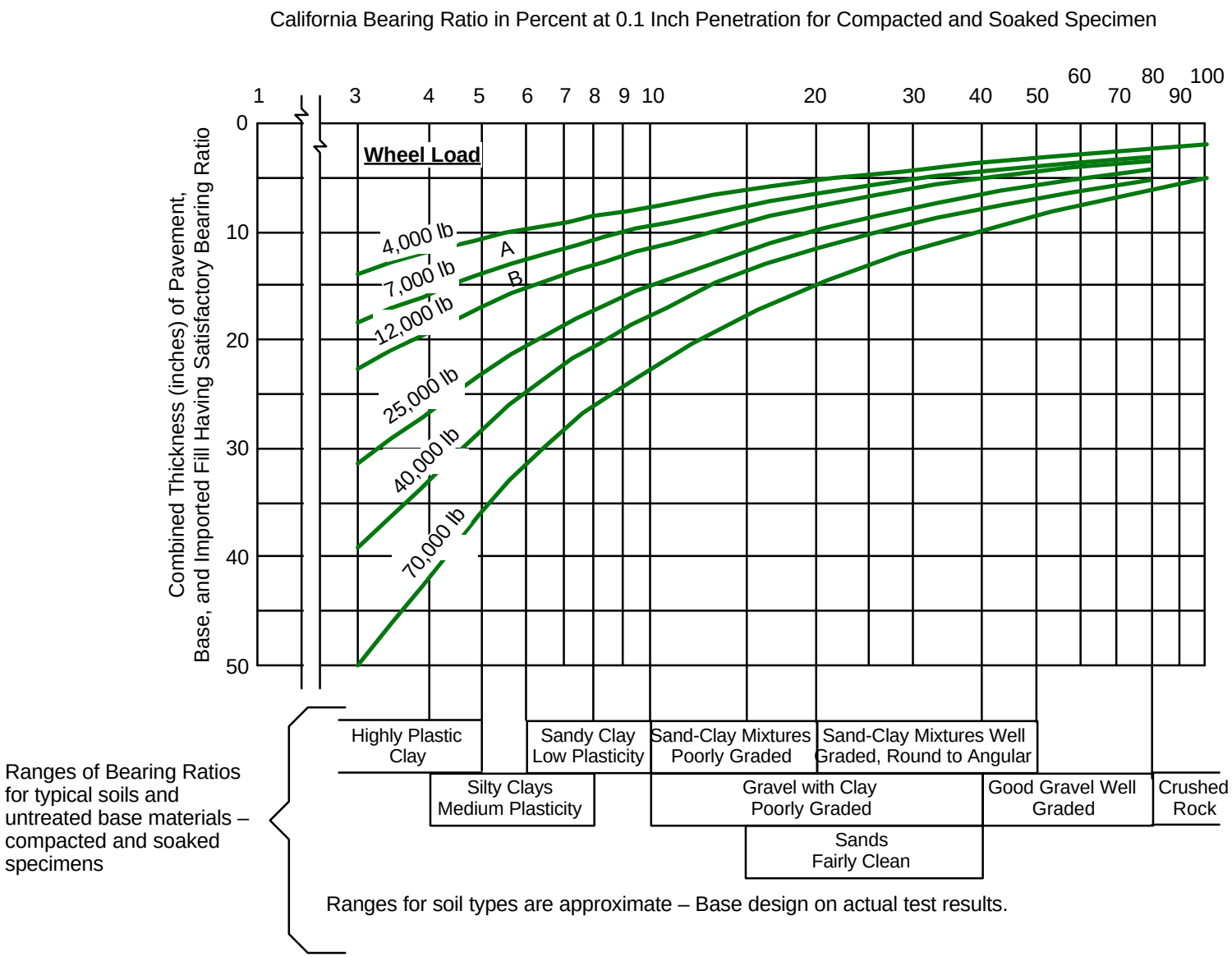


Figure 1.11. Sketch of Design Curves for Flexible Pavement Design (after Porter [1.50])

assumption of the pavement structure lying on compacted soils (at least 300 mm of compacted subgrade). Porter's paper and a discussion of it by A.C. Benkelman further "reinforced" the concept of limiting pavement deflections for design purposes.

The use of the triaxial test during the 1930s and 1940s was twofold:

(1) determination of shear strength (cohesion and ϕ angle), and

(2) estimation of moduli similar to a modulus of elasticity [1.45, 1.46, 1.47].

Worley [1.46] in 1943 reported on the use of triaxial testing and modulus of deformation results for Kansas flexible pavement design. In the Kansas tests, samples either 70 by 200 mm or 125 by 350 mm were used depending on the maximum aggregate size. The stress-strain curve from the triaxial test was used to estimate the modulus of deformation. (Worley realized that the slope of the stress-strain was not constant for most pavement materials, hence the term modulus of deformation was used in lieu of modulus of elasticity.) These moduli were used in the formula shown as Note 1 in Table 1.4.

Nijboer [1.52] of the then Shell Oil Laboratories and Seed et al [1.53] at the University of California at Berkeley helped to introduce the pavement community to the need for accounting for repeating loadings in the pavement system. Nijboer reported on fatigue of a "sandsheet" wearing course and Seed et al. on repeated loading effects on the strength and deformation of compacted clay.

In summary, the structural design of asphalt and PCC pavements up to the 1950s (where this section stops) was a function of an evolutionary process beginning with the Romans then Telford and Macadam. This evolution accelerated greatly in the 1930s and 1940s as the prior information, hopefully, suggests.

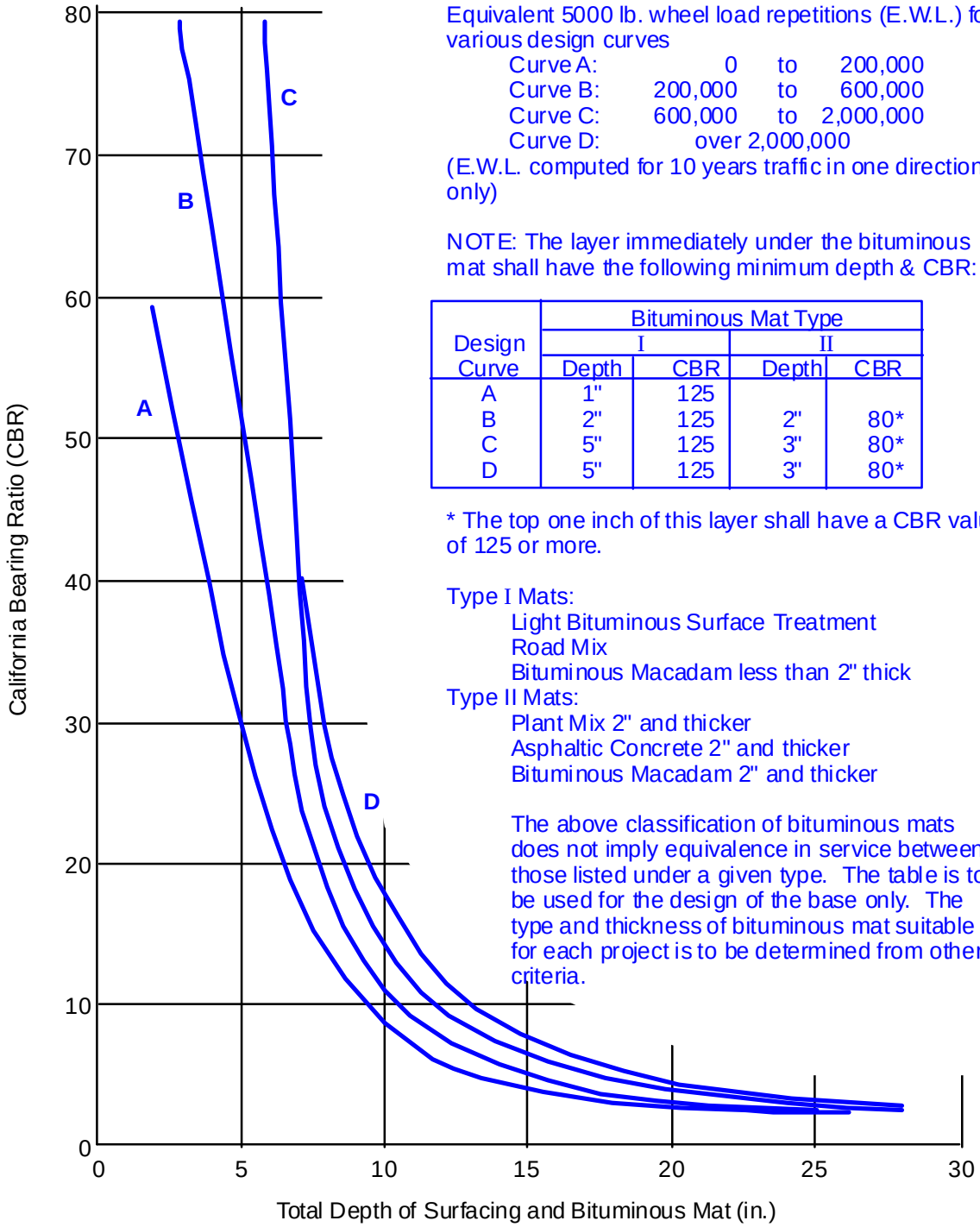
3.10 EVOLUTION OF WSDOT PAVEMENT DESIGN

3.10.1 FLEXIBLE PAVEMENTS

As of 1948, WSDOT (actually, the Washington State Highway Commission — Department of Highways) used a flexible pavement design procedure which related CBR to "thickness" with two levels of traffic. By July 1949, two additional traffic levels were added which resulted in the design chart shown in Figure 1.12.

Due to problems with some clean sands and clayey gravels along the Washington coast, the switch was made from using CBR to R value (LeClerc [1.66]). The revised design chart dated March 1951 (Figure 1.13) reflects this change. (Note: the term "surfacing and mat" requires explanation. "Surfacing" implies "surfacing aggregate" (i.e., base course) and "mat" implies "surfacing mat" (i.e., surface course)).

SURFACING DESIGN CURVES



Equivalent 5000 lb. wheel load repetitions (E.W.L.) for various design curves

Curve A:	0	to	200,000
Curve B:	200,000	to	600,000
Curve C:	600,000	to	2,000,000
Curve D:	over 2,000,000		

(E.W.L. computed for 10 years traffic in one direction only)

NOTE: The layer immediately under the bituminous mat shall have the following minimum depth & CBR:

Design Curve	Bituminous Mat Type			
	I		II	
	Depth	CBR	Depth	CBR
A	1"	125		
B	2"	125	2"	80*
C	5"	125	3"	80*
D	5"	125	3"	80*

* The top one inch of this layer shall have a CBR value of 125 or more.

Type I Mats:
- Light Bituminous Surface Treatment
- Road Mix
- Bituminous Macadam less than 2" thick

Type II Mats:
- Plant Mix 2" and thicker
- Asphaltic Concrete 2" and thicker
- Bituminous Macadam 2" and thicker

The above classification of bituminous mats does not imply equivalence in service between those listed under a given type. The table is to be used for the design of the base only. The type and thickness of bituminous mat suitable for each project is to be determined from other criteria.

Figure 1.12. Sketch of WSDOT Flexible Pavement Design Chart – July 1949

SURFACING DESIGN CURVES

Equivalent 5000 lb. wheel load repetitions (E.W.L.) for various design curves

Curve A:	0	to	200,000
Curve B:	200,000	to	600,000
Curve C:	600,000	to	2,000,000
Curve D:	2,000,000	to	5,000,000

Note: E.W.L. over 5,000,000 will require special design for surfacing depth.

Design Curve	Bituminous Mat Type I	Bituminous Mat Type II
A	1"	
B	2"	1"
C	2-1/2"	1-1/2"
D	3"	2"

Minimum Depth of Crushed Stone Surfacing Under Bituminous Mat

Type I Mats:
- Light Bituminous Surface Treatment
- Road Mix
- Bituminous Macadam less than 2-1/2" thick

Type II Mats:
- Plant Mix 2" and thicker
- Asphaltic Concrete 2" and thicker
- Bituminous Macadam 2-1/2" and thicker

The above classification of bituminous mats does not imply equivalence in service between those listed under a given type. The table is to be used for the design of the base only. The type and thickness of bituminous mat suitable for each project is to be determined from other criteria.

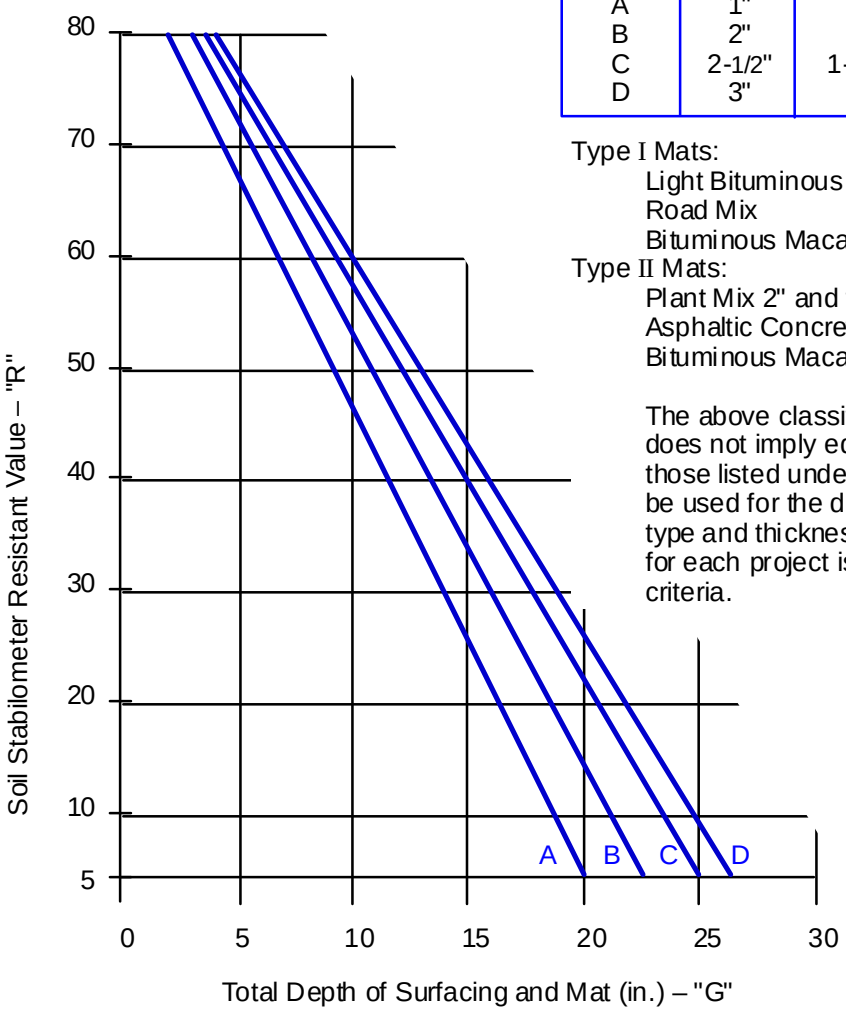


Figure 1.13. Sketch of WSDOT Flexible Pavement Design Chart – March 1951

The subsequent design chart came just two months later (Figure 1.14) and suggests the changing traffic conditions (i.e., compare Figures 1.13 and 1.14). The design chart shown in Figure 1.14 was presented in LeClerc's 1956 HRB paper [1.66]. He noted that the design procedure was based on the work of Hveem and Carmany [1.38] of the then California Division of Highways; however, a few modifications were incorporated which resulted in more conservative designs (resulting in a better match with actual pavement performance). Further, the R value test, as it evolved in Washington state, was conducted a bit differently than in California. Though it may not be obvious by examining Figure 1.14, no recognition was given in reducing the total depth of pavement due to the thickness of the asphalt concrete surfacing. This and other deficiencies resulted in further revision to the design process (LeClerc [1.67]).

The LeClerc report [1.67] of October 1956 laid out the rationale for converting mixed truck repetitions to 5,000 lb equivalent wheel loads to Traffic Index (TI). Since the TI was used by WSDOT for about 35 years (1956 to 1991), the basis for it will be summarized.

3.10.1.1 Derivation of Traffic Index

LeClerc started with the following equation (originally developed by Seed et al. [1.53] which related the number of load repetitions to axial stress at 1 percent strain for a compacted clay soil):

$$\log r = 1.04 - 0.145\,(Sa) \qquad \text{Eq. 1.1}$$

where r = number of load repetitions, and

Sa = axial stress.

From the above, a second equation was developed for the relationship between the number of repetitions for two stress levels:

$$\log r_1 = \log r_2 + 0.145\,(Sa_2 - Sa_1) \qquad \text{Eq. 1.2}$$

where r_1 = number of allowable load repetitions for stress level Sa_1, and

r_2 = number of allowable load repetitions for stress level Sa_2.

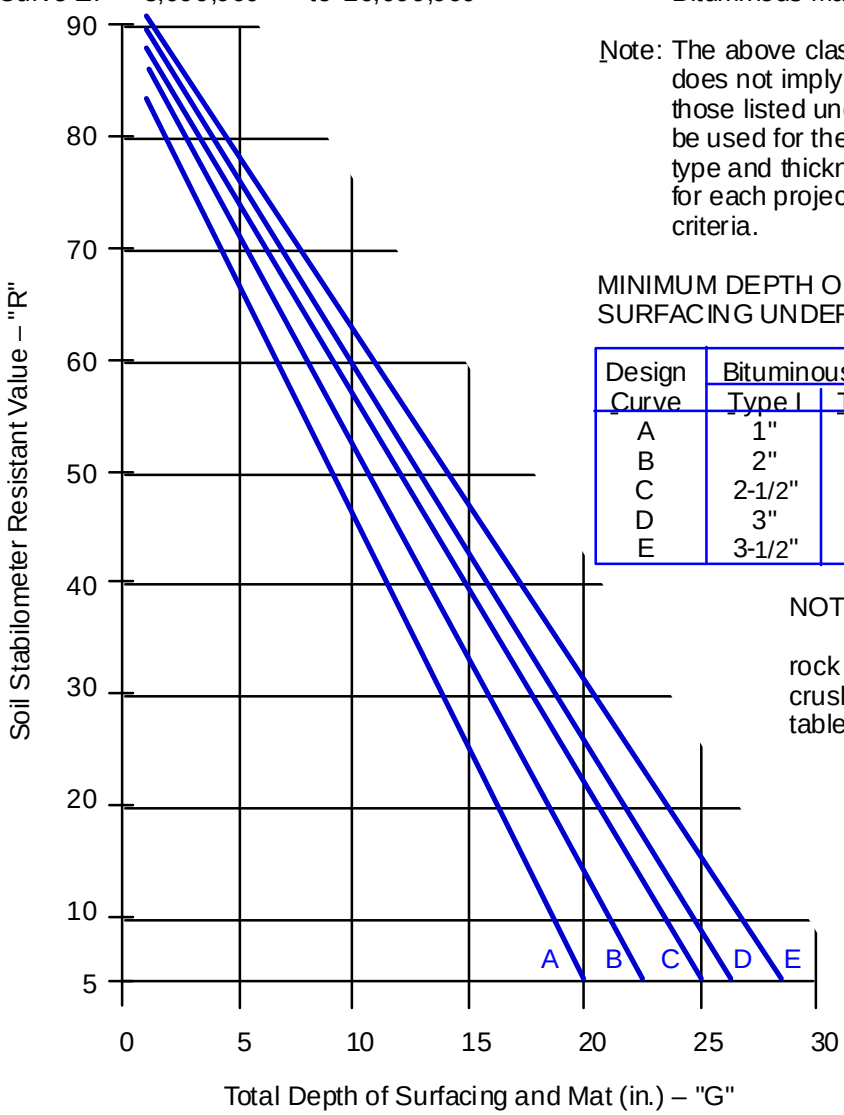
Next, LeClerc modified Equation 1.2 to account for the vertical stress applied to a subgrade by dual tires through 250 to 300 mm (10 to 12 in.) of gravel. The stress in psi is approximately 1.38 times the wheel load in kips. Substitution results in

SURFACING DESIGN CURVES

CURVE DETERMINATION:

Equivalent 5000 lb. wheel load repetitions (E.W.L.) for various design curves: (EWL computed for 10 years traffic in one direction only.)

Curve A:	0	to	200,000
Curve B:	200,000	to	600,000
Curve C:	600,000	to	2,000,000
Curve D:	2,000,000	to	5,000,000
Curve E:	5,000,000	to	20,000,000



BITUMINOUS MATS:

Type I Mats:
- Light Bituminous Surface Treatment
- Road Mix
- Bituminous Macadam less than 2-1/2" thick

Type II Mats:
- Plant Mix 2" and thicker
- Asphaltic Concrete 2" and thicker
- Bituminous Macadam 2-1/2" and thicker

Note: The above classification of bituminous mats does not imply equivalence in service between those listed under a given type. The table is to be used for the design of the base only. The type and thickness of bituminous mat suitable for each project is to be determined from other criteria.

MINIMUM DEPTH OF CRUSHED STONE SURFACING UNDER BITUMINOUS MAT

Design Curve	Bituminous Mat	
	Type I	Type II
A	1"	––
B	2"	1"
C	2-1/2"	1-1/2"
D	3"	2"
E	3-1/2"	2-1/2"

NOTE:
If ballast made from ledge rock is specified, the required depth of crushed stone surfacing shown in this table may be reduced by 1 inch.

Figure 1.14. Sketch on WSDOT Flexible Pavement Design Chart – May 1951

$$\log r_1 = \log r_2 + 0.2\,(WL_2 - WL_1) \qquad \text{Eq. 1.3}$$

where r_1, r_2 as in Eq. 1.2,

WL_1 = 5,000 lb wheel load (in kips),
WL_2 = dual wheel load in question (kips).

Thus, if we want to relate how TI (actually log r_1) relates to ESALs, substitute the following into Equation 1.3:

WL_1 = 5 kips
WL_2 = 9 kips (or 1/2 of 18,000 lb single axle).

Thus,

$$\log r_1 = \log r_2 + 0.2\,(9-5)$$

or $$\log r_1 = \log r_2 + 0.8$$

or $$TI = \log ESALs + 0.8$$

or $$\log ESALs = TI - 0.8.$$

If ESALs = 1,000,000, then the

$$TI = 6.0 + 0.8 = 6.8 \text{ and}$$

$$\frac{TI}{\log ESALs} = \frac{6.8}{6.0} \simeq 1.13$$

or $$TI \simeq 1.13\,(\log ESALs).$$

A multiplicative relationship between TI and log ESALs varies a bit as a function of the ESAL or TI level assumed.

Subsequent revisions have resulted in the following TI-ESAL relationships (developed by regression):

$$TI = 0.93\,(1 + \log ESALs)$$

LeClerc in 1956 [1.67] went on to note that:

(a) EWL values for shoulders can be taken as 1 percent of the adjacent roadway.

(b) The TI should be increased by 1.0 for bus stops, acceleration and deceleration lanes and arterial intersections requiring stops.

(c) On four-lane highways, the heaviest traveled lane is assumed to carry 75 percent of the unidirectional traffic.

In the next section, the derivation of the governing equation for all flexible pavement design charts up to 1991 (refer to Figure 520-1a in the *WSDOT Design Manual* issued March 1991) will be presented.

3.10.1.2 Derivation of Pavement Design Equation

Again, based on LeClerc's October 1956 report [1.67], the "design equation" used up to March 1991 was developed. He started with

$$S = a + b(R) \quad \text{Eq. 1.4}$$

where S = total surfacing depth (in.),

R = stabilometer "R" value, and

a, b = regression constants.

LeClerc derived equations for estimating "a" and "b" in terms of log of equivalent wheel load (or TI). By substituting these into Eq. 1.4, the following equation was developed:

$$S + 9.0 = (\log EWL + 0.10)\,(5.90 - 0.047\,(R)) \quad \text{Eq. 1.5}$$

Thus, if we let

$R = 25$

$\log EWL = TI = 6.0$, then

$S \cong 19.82$ in. $\simeq$ 1.65 ft.

This total surfacing depth includes a "light" bituminous wearing course.

The modification of the total surfacing depth to account for "stronger" materials such as asphalt concrete or cement treated base was based on the following equation:

$$S_M = \frac{S_T}{(C)^{0.2}} \quad \text{Eq. 1.6}$$

where S_M = modified surfacing depth (in.),

S_T = total surfacing depth (in.), and

C = a value based on the cohesiometer test (cohesiometer value divided by 100).

Initially, the following cohesiometer values were used:

- cement treated base = 650
- asphalt concrete = 250
- crushed stone, ballast = 100

To account for more than one "surfacing" material, a "mean" cohesiometer value was determined according to the following:

$$C_m = C_1 + \left(\frac{t_2}{t_1 + t_2}\right)^2 (C_2 - C_1)$$

where C_m = mean cohesiometer value,
C_1 = cohesiometer value of top layer,
C_2 = cohesiometer value of the next (or second) layer,
t_1 = thickness of top layer,
t_2 = thickness of second layer.

To illustrate how this process worked, the design chart of March 22, 1957 (shown as Figure 1.15), and an asphalt concrete surface course of 4.0 in. will be used. Inspection of Figure 1.15 shows a modified surfacing depth of about 18.2 in. for a TI = 6.0, R = 25, and a 4.0 in. AC layer (total surfacing depth of 19.8 in. unmodified for the AC layer). To calculate the reduction in the total surfacing depth due to the AC, first calculate C_m:

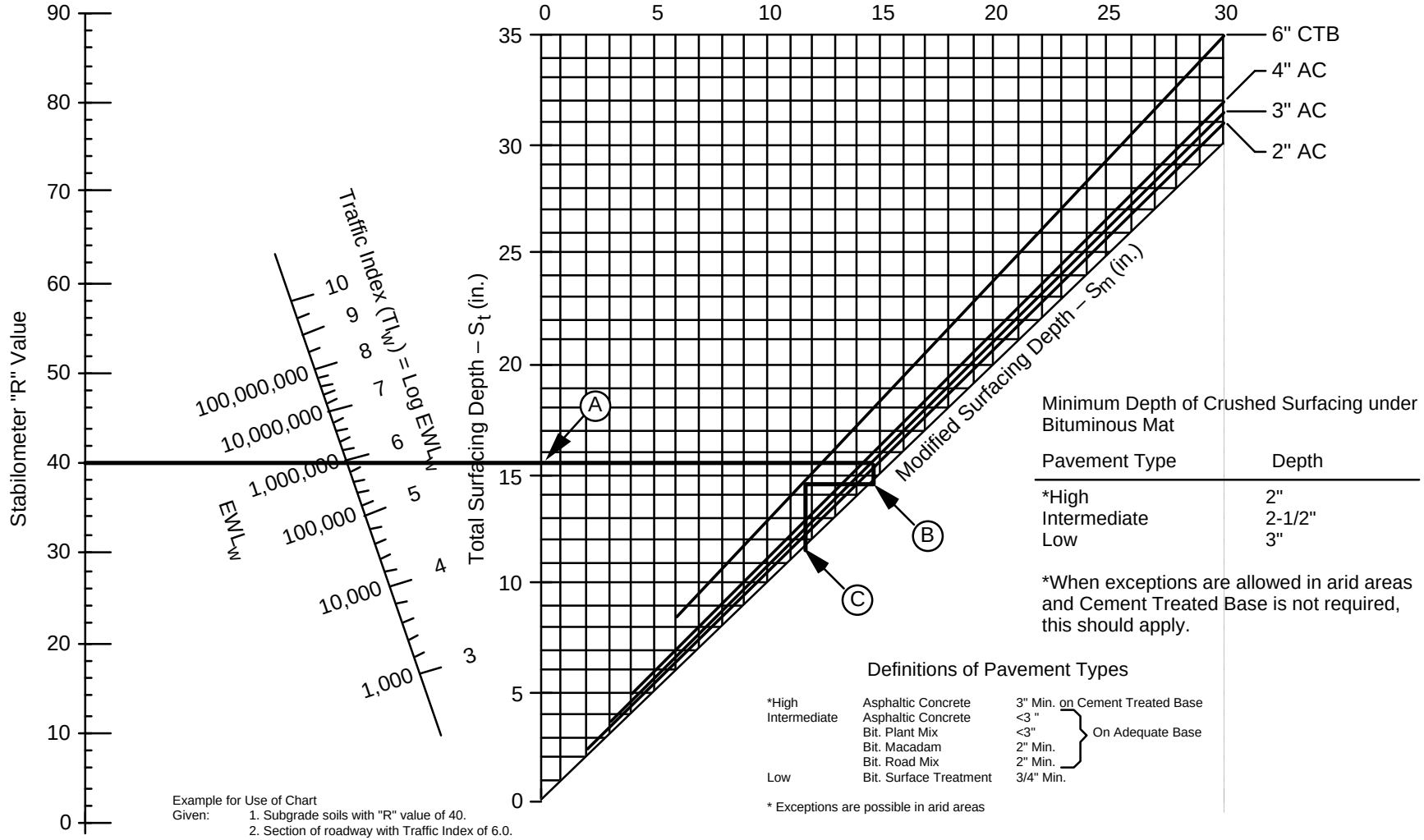
$$C_m = 250 + \left(\frac{15.8}{19.8}\right)^2 (100 - 250)$$
$$\simeq 154$$

Next, calculate S_M:

$$S_M = \frac{19.8}{\left(\frac{154}{100}\right)^{0.2}} \cong 18.2 \text{ in.} \qquad \text{(check)}$$

Thus, a pavement so designed would have a 4.0 in. AC layer over 14.2 in. of base course (or a total of 18.2 in.).

Subsequently, the process of reducing the total surfacing depth to account for ACP, ACB, and CTB was further modified. The WSDOT design chart contained in the WSDOT Design Manual is shown as Figure 1.16. The total surfacing depth is calculated by Equation 1.5 as shown previously. However, more specifically,



Example for Use of Chart
Given: 1. Subgrade soils with "R" value of 40.
2. Section of roadway with Traffic Index of 6.0.

The "R" value and the Traffic Index will be furnished with the Soils Report.

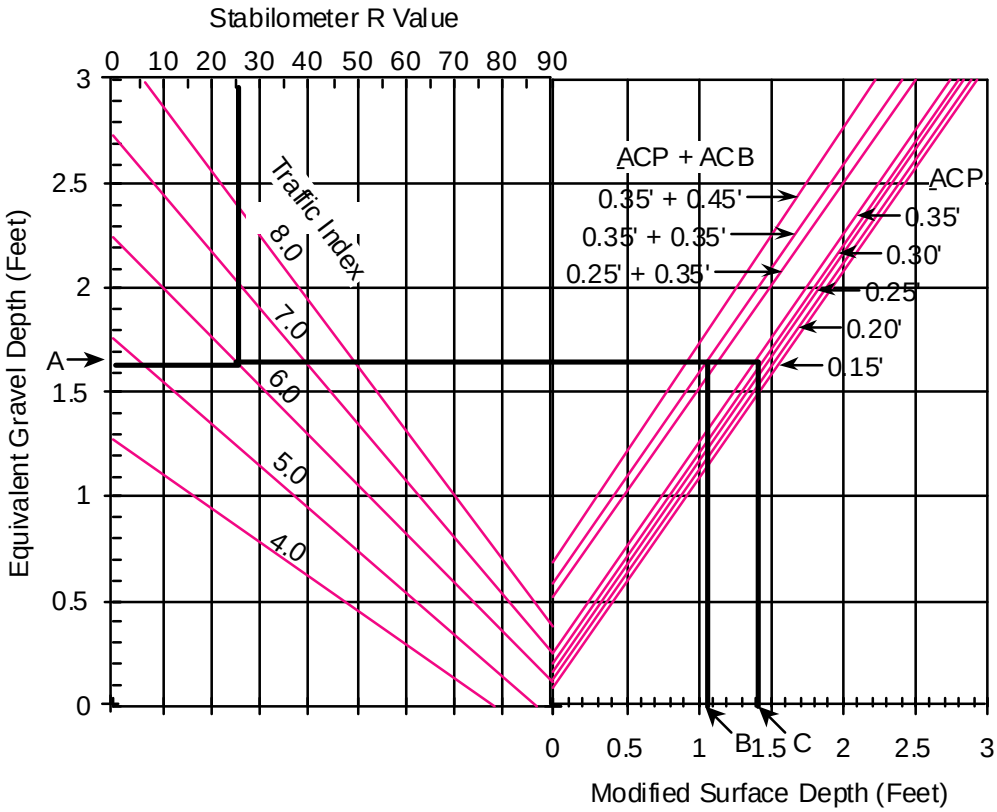
To find the total surfacing depth required, connect a line from 40 on the "R" Value scale through 6.0 on the Traffic Index scale and find its intersection with the Total Surfacing Depth scale, Point A. This is indicated to be 15-1/2 inches and is the total depth required if a light bituminous surface treatment is to be used for pavement.

If a heavier pavement, for example 3 inches of asphaltic concrete, is to be used, a reduction in the total surfacing depth may be made. To determine this reduced or modified depth, find the intersection of the 15-1/2 inch line with the diagonal 3 inch AC line and read 14-1/2 inches on the Modified Surfacing Depth scale, point B. If in addition, 6 inches of CTB is planned, the 14-1/2 inch depth may be further reduced by moving horizontally to the left from point B to the intersection with the diagonal 6 inch CTB line and then down to 11-1/2 inches on the Modified Surfacing Depth scale, point C.

The reductions in total surfacing depth are additive, and each course, CTB or AC, contributes to the total reduction. The reductions may be determined in any order.

The "thickened edge" asphaltic concrete mat (2-1/2" at centerline and 4" at edge) will be considered as a uniform 3" mat for design purposes.

Figure 1.15. Sketch of WSDOT Flexible Pavement Design Chart – March 1957



Example:

Given an R value of 25 and a traffic index of 6.0, cover thickness requirements can be determined as follows:

An equivalent gravel depth of 1.65' (round to 1.7) at point A.

A modified surfacing depth of 1.05 feet at point B for a pavement of 0.35 foot ACP + 0.35 foot ACB.

A modified surfacing depth of 1.41 foot (round to 1.45) at point C for pavement of 0.35 foot ACP.

Figure 1.16. Sketch of WSDOT Flexible Pavement Design Chart – March 1991

S = S_t = total surfacing depth (gravel equivalent), and

R = "R" value at a 400 psi exudation pressure.

(Note: This is the estimated pressure at which water is exuded from compacted R value test specimens and is comparable to the condition of the soil in the field during compaction.)

The relationship between gravel equivalent and asphalt concrete (AC), asphalt treated base (ATB), and cement treated base (CTB) is as follows (from LeClerc and Sandahl [1.72]):

$$\text{Gravel Eq. for AC} = t_{AC}\sqrt[5]{(3)(t_{AC})} \qquad \text{Eq. 1.7}$$

$$\text{Gravel Eq. for ATB} = t_{ATB}\sqrt[5]{(2)(t_{ATB})} \qquad \text{Eq. 1.8}$$

$$\text{Gravel Eq. for CTB} = t_{CTB}\sqrt[4]{(1.4)(t_{CTB})\left(\frac{1 + 2\ \text{Gr. Eq.}^*}{1 + \text{Gr. Eq.}^*}\right)} \qquad \text{Eq. 1.9}$$

where t_{AC} = thickness of AC layer,

t_{ATB} = thickness of ATB layer,

t_{CTB} = thickness of CTB layer, and

Gr. Eq.* = sum of gravel equivalent values of all structural components overlying the CTB (generally taken to be AC only but could be a combination of AC and crushed surfacing).

As noted by LeClerc and Sandahl [1.72] Equations 1.7, 1.8, and 1.9 were empirically derived from test track data and Washington State in-service pavements.

Figure 1.17 shows the relationship between the thickness of untreated base material (crushed gravel for example) and thickness of ATB. This chart was also contained in the WSDOT Design Manual through 1993. The relationship shown uses Equation 1.8 and which can be illustrated as follows:

If ATB = 0.60 ft (7.2 in.), determine the thickness of untreated base material. From Eq. 1.8:

$$\text{Gravel Eq. for ATB} = (7.2)\sqrt[5]{(2)(7.2)} \cong 12.3 \cong 1.02\ \text{ft}$$

Thus, the equivalent depth of untreated base is 1.02 ft and this checks with Figure 1.17.

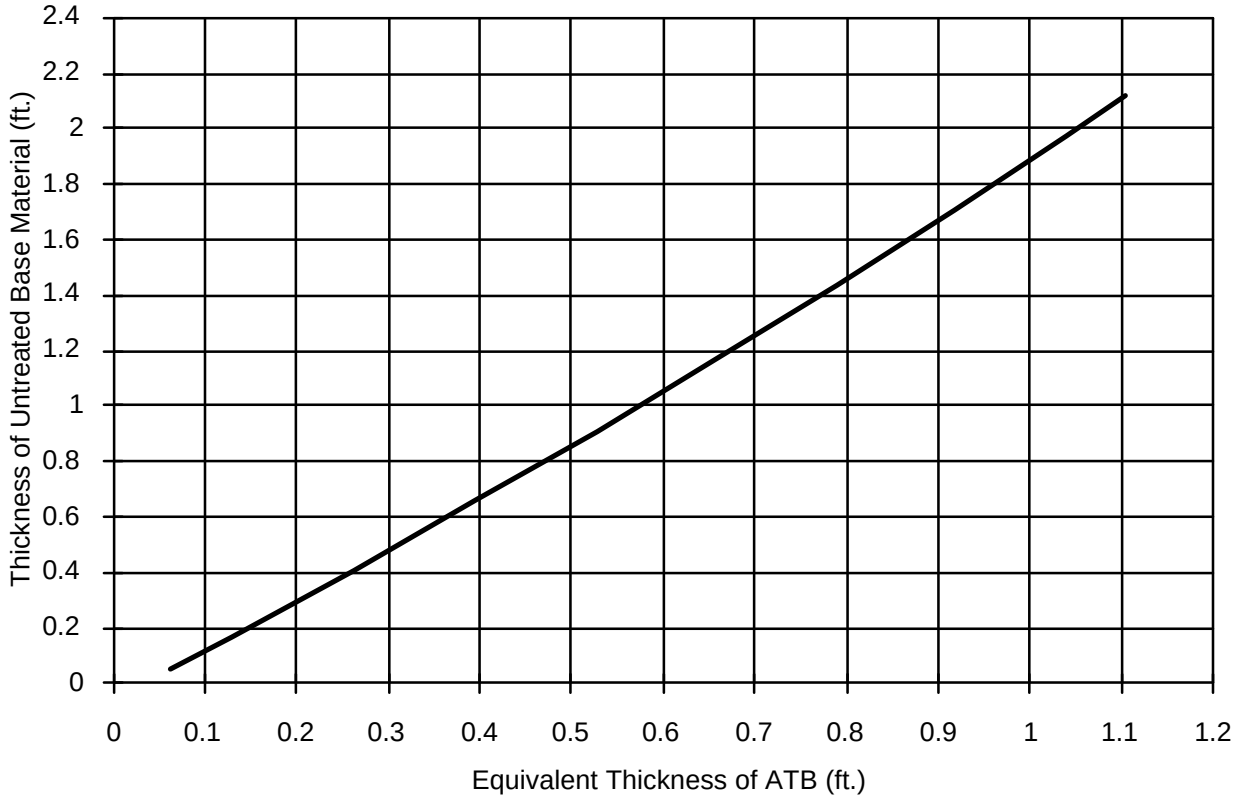


Figure 1.17. Sketch of WSDOT Conversion Chart for ATB and Untreated Base

As an illustration of this design process, calculate the layer thicknesses for a flexible pavement with a 0.35 ft (4.2 in.) thick AC surface. First, calculate the gravel equivalent for the "known" AC layer (use Eq. 1.7):

$$\text{Gravel Eq. for AC} = 4.2\sqrt[5]{(3)(4.2)} = 6.97 \simeq 7.0 \text{ in.} = 0.58 \text{ ft}$$

If R = 25 and TI = 6.0, then S_t = 1.65 ft (as previously shown)

Thus, the thicknesses are:

AC	=	0.35 ft (4.2 in.)
Gravel	=	1.65 ft - 0.58 ft = 1.07 ft (12.8 in.)
Total	=	0.35 ft + 1.07 ft = 1.42 ft

or from the design chart (Figure 1.16):

Modified Surfacing Depth = 1.42 ft for AC = 0.35 ft (checks)

Note that gravel equivalent implies, at a minimum, <u>crushed gravel</u>.

3.10.2 RIGID PAVEMENTS

3.10.2.1 Thickness Design

The earliest reference to the WSDOT approach for the design of rigid pavements is contained in a report prepared at the WSDOT Materials Laboratory about 1953 (no author noted in the document).

It was noted at that time (presumably 1953) that PCC pavement would be considered only when the 10-year design traffic exceeded 3,000,000 EWLs (or a TI ≅ 6.5 or ESALs ≃ 475,000). Thus, PCC was considered for ESALs per year levels of about 50,000 (compare this to contemporary ESALs per year of greater than 1,000,000 on I-5 in the Seattle area).

The 1953 information also noted that PCC slabs must be supported by at least 100 mm of "clean crushed material" in order to prevent pumping and provide a "uniform, stable foundation under the full width of the slab." Further, in addition to requiring a TI greater than 6.5, the report stated that PCC should not be used if the R value of the subgrade was greater than 40. The justification given for this was based, in part, on Figure 1.14. For R = 40 and traffic Curve D (2,000,000 to 5,000,000 EWLs), the total surfacing depth required was 400 mm (16 in.). Further, it was noted that a 75 mm (3 in.) AC over 150 mm (6.0 in.) CTB over 75 mm (3 in.) of granular base (300 mm (12 in.) total) was considered equivalent to 400 mm (16.0 in.) of total surfacing (assumes only a "light" bituminous surface treatment wearing course). Apparently, a 200 mm (8.0 in.) PCC slab over 100 mm (4.0 in.) of base was considered equivalent to the AC/CTB structure; hence, the R value 40 requirement.

In a 1958 Materials Laboratory report prepared by LeClerc [1.68], it was noted that about 100 mm (4.0 in.) of clean granular material is required under PCC slabs to provide a "stable base and prevent pumping." To ensure that this in fact occurs, LeClerc noted that a 150 mm (6.0 in.) minimum depth of base was considered the "practical minimum requirement." He further noted that the minimum rigid section was 345 mm (1.15 ft) in depth:

- 195 mm (0.65 ft) PCC slab
- 150 mm (0.50 ft) clean granular base

For "large volume roadways" in <u>wetter climate areas</u> the minimum PCC section was:

- 225 mm (0.75 ft) PCC slab
- 150 mm (0.50 ft) clean granular base

LeClerc further noted that base course depths greater than 150 mm (0.5 ft) may be required on the basis of R value of the subgrade/embankment (the assessment apparently based on prior pavement performance).

A paper by Miller (Paving Engineer, Portland Cement Association) published apparently in 1971 [1.69] noted that the then Washington Highway Department designed the PCC

slab thickness based on a fatigue concept similar to that developed by the PCA. The process provided for a 20 year traffic projection. The design process required:

- conversion of R value to k value
- stress calculations which were a function of wheel load (including load safety factors which ranged from 1.1 to 1.3) and k value (single and tandem axle stresses were calculated separately)
- fatigue evaluation whereby the stress ratio for each axle load was calculated (applied stress divided by the modulus of rupture of the PCC)

Since the 1986 *AASHTO Guide for Design of Pavement Structures* became available, WSDOT Materials Laboratory personnel have used this procedure for rigid pavement design.

3.10.2.2 Joint Design

WSDOT contraction joint practice has evolved over time. For example, SR11 south of Bellingham (for those portions not overlaid) have joints roughly every 9.0 m. This pavement was constructed in 1921. PCC placed on SR 2 near Spokane ("Sunset Highway") in 1919 had contraction joints generally less than 6 m apart. In general, WSDOT joint design can be summarized as follows:

(a) 1940s-1950s Straight (non-skewed) joints spaced (4.6 m) apart.

(b) 1966 Random spacing was adopted which ranged from 4.3 to 5.6 m

(c) 1967-1992 Random spacing was reduced to a range of 2.7 to 4.3 m for an <u>average</u> of 3.5 m. The actual spacing pattern is 2.7, 3.0, 4.3, and 4.0 m.

(d) 1992
- Undoweled joints

  Same design as noted for 1967-1992 above.
- Doweled joints

  Straight (non-skewed) joints spaced (3.7 to 4.6 m) apart.

3.10.3 FROST DESIGN

The earliest reference as to how frost effects were incorporated into the pavement design process was provided by LeClerc in a 1958 report [1.68] (additionally refer to Part II, SECTION 2.0, Paragraph 3.2.4). He noted that subgrade or borrow soils that contain 10 percent or greater particles passing the 75 µm sieve <u>were</u> susceptible to frost

effects and should be protected by an "adequate" depth of presumably "frost free" cover. Further, he wrote

> "… cover depth used in our state to protect against frost damage is based on the widely accepted formula of providing cover equal to at least one-half the maximum depth of frost penetration."

3.11 WASHINGTON STATE HIGHWAY MILESTONES

A few notes obtained mostly from References 1.61, 1.73 and 1.75 will be presented relative to the origins of WSDOT (by year):

1852 The Legislature of Oregon Territory established the first "official" road—Washington State Historical Road No. 1 (Byrd's Mill Road) which connected Puyallup to Tacoma to Steilacoom.

1853 Seattle needed an east-west route (other than water) — Portland and Oregon were growing and Seattle did not want to miss out on development. In 1853, some references (such as Reference 1.62) note that Gov. Stevens had the first survey done to locate a Snoqualmie Pass route, which is true. However, Speidel [1.63] notes that in fact Doc Maynard (who actually founded Seattle) first located the Pass in the early 1850s. In 1865, A.A. Denny, H.L. Yesler, and J.E. Clark (citizens of Seattle) collected $2,500 to improve a 40 km section through the pass. In 1867, the county appropriated $2,500 for continuing maintenance and improvement work. During the summer of 1868, the Snoqualmie Pass wagon road was finished. The road was mainly used by freight wagons and settlers until the 1909 Alaska - Yukon - Pacific Exposition. The A-Y-P created a need to improve the road since an auto race was planned from New York to Seattle via Snoqualmie. Funds were raised and the road improved so that 150 cars could get through — thus staring auto traffic over the pass.

1893 Legislative action was started on the Cascade Wagon Road. The route roughly follows the North Cascades Highway—SR 20 (SR 20 completed in September 1972).

1899 The Washington Good Roads Association (WGRA) was founded in Spokane by a group of 14 which included Samuel Hill, R.H. Thomson, and former Gov. Roland H. Hartley. Samuel Hill was president of the WGRA from 1899 to 1908. In general, the WGRA pushed for state (or central) control of road building and maintenance as opposed to county control.

1905 The legislature established the Washington Highway Department along with a State Highway Fund and a three member state highway board (to consist of the state auditor, state treasurer, and the state highway commissioner). Operation under the 1905 legislation was difficult.

> "Maps and plans under the law were to be prepared by the counties. The county commissioners were to advertise for bids and, with the approval of the highway commissioner, were to award contracts. That this division of authority led to much friction is evident from the first biennial report of the highway department which records many instances of counties failing to cooperate with the highway board."

There were 1 740 kilometers of state roads of which 200 km were classified as "improved."

1907 New legislation

> "provided that plans and specifications for state road construction were to be prepared by the highway department and that bids were to be called for and contracts were to be awarded by the state highway board."

1909 Until 1909, the maintenance of state roads was the responsibility of the counties. The legislature now required that maintenance of these roads be via state funds (however, only $10,000 was appropriated for this purpose).

1910 Highway commissioner Henry L. Bowby noted in his biennial report to the legislature

> "The advent of the automobile brought new problems to the road engineer."

He went on to say that waterbound macadam is adequate for horse drawn vehicles, or tar or asphalt macadam for automobile traffic — but no solution was available for the combined problem of horsedrawn and automobile traffic. He recommended that $100,000 be appropriated for field and laboratory experiments to try to solve this problem. Ultimately, the decision was made to design roads for the automobile rather than horse-drawn vehicles.

1911 The Permanent Highway Act was passed by the legislature. Permanent highways were to be graded to a width of not less than 4.9 m and surfaced with macadam, stone, gravel, or some other durable material for a width of not less than 3.6 m. Grades were not to exceed 5 percent wherever possible and in no case greater than 10 percent.

1912 According to the WGRA [1.64], a PCC pavement was built near Toledo, Washington, in Lewis County. Reference 1.61 noted that the project was 4.5 km long and the pavement section consisted of a base 114 mm thick (1:3:5 mix) with a 38 mm thick wearing course (1:1:1 mix). Expansion joints were placed every 15.2 m (cost: $1.30/yd^2).

In Lincoln County, a PCC pavement was constructed with a 100 mm thick base (1:3.5:7 mix) and a 38 mm thick wearing course (2:3:6 mix) (cost: $0.90/yd^2).

In Franklin County a PCC base was constructed with an asphalt wearing course. The base was 125 mm thick (1:2:4 mix) with a "top course of one-third of a gallon of asphaltic oil per square yard with screenings to take up the oil." Expansion joints were provided every 15.2 m (cost: $1.25/yd^2).

In King County, two types of "hard surfacings" were used:

- Warrenite: a bituminous wearing course on a PCC or crushed rock base.
- Brick on a PCC base (most of the brick was placed on a sand cushion over the PCC base).

1916 Federal Aid Act was approved by Congress and signed by President Wilson on July 11, 1916. The first allotment of funds became available to the state for the fiscal year ending June 30, 1917.

1920 There were 186,827 motor vehicles in the state of Washington.

1921 "The Department of Highways became a division of the Department of Public Works."

"Prior to July 1921, all testing of highway construction materials was done in the Testing Laboratory of the City of Seattle. The 1921 legislature provided funds for equipping and operating a Division of Highways' laboratory. The abandoned boiler and coal rooms of the Temple of Justice were equipped for a laboratory. Testing machines were purchased and other machines were assigned to the Division of Highways by the federal government from surplus war supplies."

The state gasoline tax was first assessed at $0.01 per gallon.

1923 The legislature abolished the Division of Highways within Department of Public Works and authorized the governor to appoint a state highway engineer.

Legislation placed the maintenance of primary state highways under the control of the state highway department.

1927-1929 The first extensive oiling program by the state highway department was done:

> "This resulted from a 1927 legislative appropriation of $200,000 to be used for experimental work in dust prevention. Light oil treatment was tried in various sections throughout the state, but heavy oil was used at that time only on highways in arid and irrigated sections east of the mountains. A considerable amount of calcium chloride was also used as a dust preventative in the western part of the state."

> "The division of authority between the state highway engineer and the non-technical state highway committee was a far from satisfactory arrangement … friction developed between the state highway engineer (Samuel J. Humes) and the other members of the state highway committee. The 1929 legislature … abolished the state highway committee, created the position of director of highways, and made the Department of Highways a separate code department."

1933-1935 Lacey V. Murrow was appointed to succeed S.J. Humes as Director of Highways on March 20, 1933. (Lacey Murrow was the older brother of Edward R. Murrow. Lacey V. graduated from WSU in the 1920s, and while there, was a class officer and honor cadet [1.70].)

Large amounts of federal appropriations were made available during 1933-1935 for highway construction:

> "As the primary object of these federal appropriations was to relieve unemployment, Department of Highways' contracts carried provisions calling for use of as much hand labor as possible."

> "Because of the need for more adequate control of heavily loaded vehicles on state highways, a traffic control organization was established. Twenty-six stationary platform truck scales were installed at vantage points along state highways and for use at points remote from these weighing stations each of the six traffic officers was provided with a drive-on loadometer to be carried in his car."

1936

> "At the request of the United States Bureau of Public Roads, the Department of Highways in 1936, started a statewide highway planning survey. This consisted of making a comprehensive study of the entire road system of the state as a preliminary step to the formulation of a sound, longterm highway program."

The survey consisted of:

- Road Inventory,
- Traffic Survey, and
- Financial survey.

1937 The legislature created the Commission on Highway Transportation. Composed of

- Director of Highways
- Director of Public Service
- Director of Licenses
- Chair of Tax Commission
- Transportation engineer appointed by the Governor

1940 "L.V. Murrow resigned September 15, 1940 to enter military service. Many other division heads, assistant division heads and subordinate employees followed in quick succession."

On November 7, 1940, the Tacoma-Narrows Bridge was overcome by gravity and a bit of wind.

1941 The Highway Advisory Commission was created by the legislature to be composed of five members appointed by the Governor.

> "The commission was to act in an advisory capacity in designating where appropriations not specifically tied down by law were to be expended, to pass on routes of highways and on plans and specifications, and to submit recommendations to the governor, upon request, concerning all matters affecting Department of Highways administration."

1943-1945 The traffic activities of the Department were reduced whereby the Washington State Patrol assumed responsibility for regulating operations of "heavy trucks."

In 1944, the Federal Aid Highway Act created a national system of interstate highways. The resulting interstate system in Washington was 960 km (599 mi.) long and included State Highway 1 (US 99) and State Highways 2, 7, 18 and 11 (old US 10).

1951 Legislature created the Highway Commission which assumed the duties previously performed by the Director of Highways.

"The results (of establishing the Commission) were dramatic, giving the Highway Department a continuity that it formerly lacked. State highway policy was given a non-political outlook. Not more than one member of the Highway Commission may be appointed from any one of the congressional districts. Not more than three can be from either side of the state's Cascade range, nor could more than three be members of either of the major political parties. Each member is appointed by the Governor to a six-year term."

"At its first meeting, the Commission appointed Director of Highways W.A. Bugge to succeed himself." (Note: William Adair Bugge, Sr. served as Director of Highways for 14 years. He also served as President of AASHTO in 1957 and project director for Engineering Consultants for the design and construction management of the Bay Area Rapid Transit System. He died in Olympia on November 14, 1992.)

A bond issue was used to obtain funds for improvements to US 99, Snoqualmie Pass four laning and numerous other projects.

1951-Current There have been quite a few changes for WSDOT since the early 1950s, but that is where this short historical view will stop.

4. WHAT IS AASHTO?

4.1 THE BEGINNING [1.7]

The American Association of State Highway Officials (AASHO) was created in November 1914 at a meeting held in Atlanta, Georgia. At this meeting 18 state highway engineers met (including William R. Roy from Washington State) and decided that the first official AASHO meeting would be held in Washington, D.C. on December 12, 1914 with the purpose of "to assist in drafting a bill to be presented to Congress, which will embody a plan of Federal cooperation in road construction." The resulting legislation became known as the Federal Highway Act of 1916.

The AASHO Washington, D.C. meeting of 1914 was important for another reason. Two alternatives were proposed for the federal-aid program:

- a limited system of federal highways constructed by the federal government, or
- state highways with supplemental federal-aid monies.

The latter program won.

4.2 AASHTO MILESTONES [1.7]

4.2.1 Development of standards and specifications for highway design, construction, maintenance and safety (accomplished through technical committees).

4.2.2 Nationwide route numbering of U.S. primary and Interstate systems

4.2.3 HIGHWAY RESEARCH

(a) AASHO Road Test (started in 1955). Established principles for cost allocation and guided pavement design

(b) National Cooperative Highway Research Program (NCHRP) (created in 1962)

(c) Strategic Highway Research Program (SHRP)

4.2.4 AASHTO MATERIALS REFERENCE LABORATORY

Quality control program for materials laboratories for Member Departments. Created in 1955.

4.2.5 AASHO changed to AASHTO in 1973

4.3 AASHTO COMMITTEE STRUCTURE

The current AASHTO Committee structure is shown in Figure 1.18. The primary work of AASHTO is through the Standing Committees.

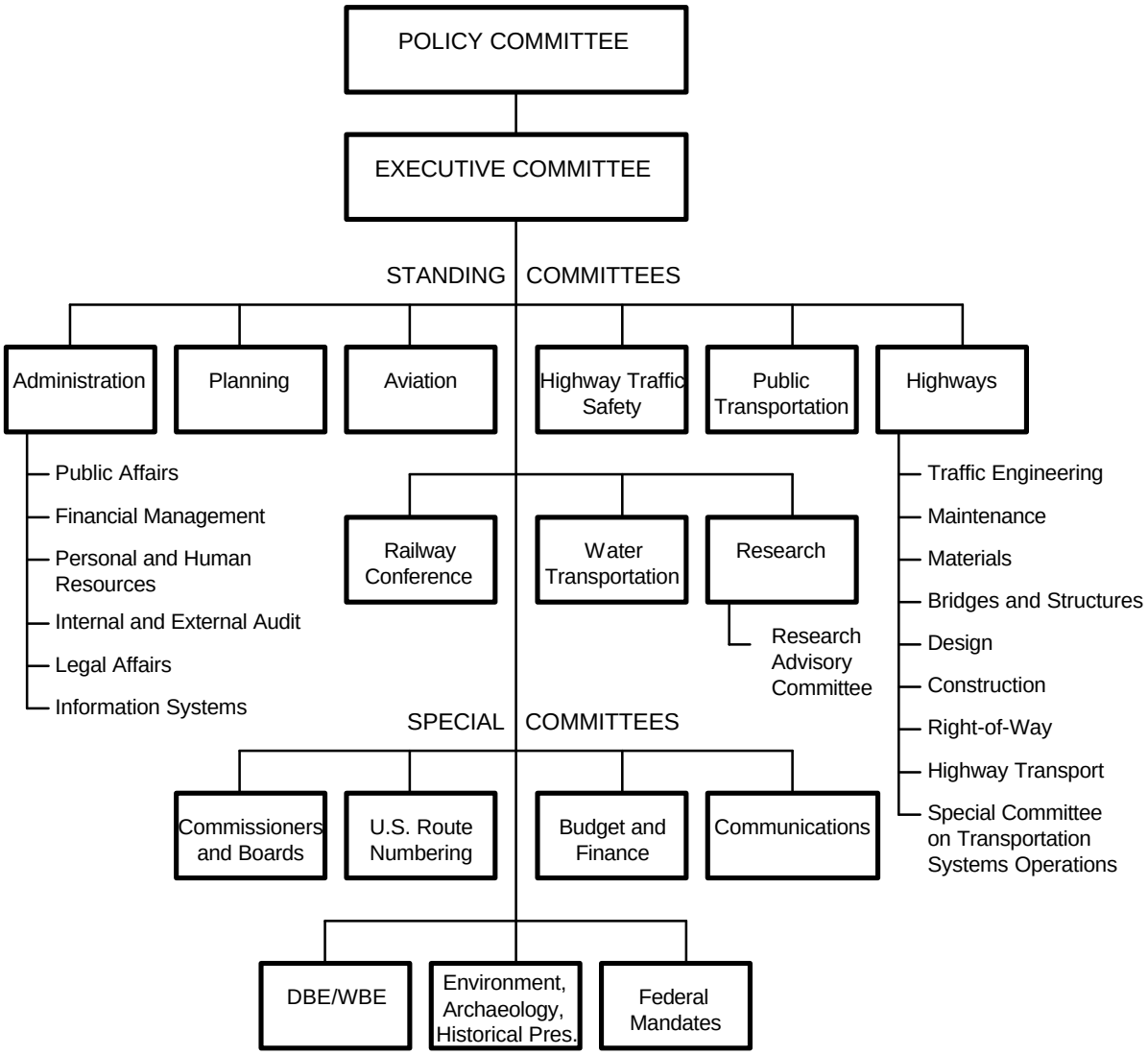


Figure 1.18. The AASHTO Committee Structure (1986) [from Ref. 1.7]

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SECTION 2.0

FUNDAMENTAL DESIGN PARAMETERS

This SECTION is used to describe how information relating to traffic loads, materials, and environmental factors can be used by WSDOT in designing and managing pavements.

1. LOADS

1.1 INTRODUCTION

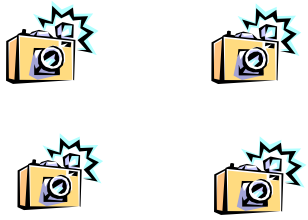
In pavement design and rehabilitation the following must be considered:

(a) Wheel load
(b) Tire inflation (contact) pressure
(c) Repetitions of wheel loads
(d) Axle and tire configurations
(e) Distribution of traffic across the pavement
(f) Vehicle speed

1.2 COMPARISON OF HIGHWAY AND AIRFIELD PAVEMENTS

(a) For airfield pavement, the most severe distress generally occurs where traffic (aircraft) follow a designated line (e.g., taxiways).

(b) For a given wheel load and tire pressure, highway pavements are thicker than airfield pavements because load repetitions are much higher for highways. However, in general, airfield pavements are thicker due to higher wheel loads.



1.3 TYPICAL HIGHWAY TIRE-AXLE LOADS AND TERMINOLOGY

1.3.1 DESCRIPTIONS

Tire-axle combinations are typically described as:

(a) Single axle — single tire (truck steering axles, etc.)
(b) Single axle — dual tires
(c) Tandem axle — single tires
(d) Tandem axle — dual tires

Refer to Figure 2.1 for illustration of these descriptions.

A number of truck related definitions from AASHTO [2.22] are contained in Appendix 2.1.

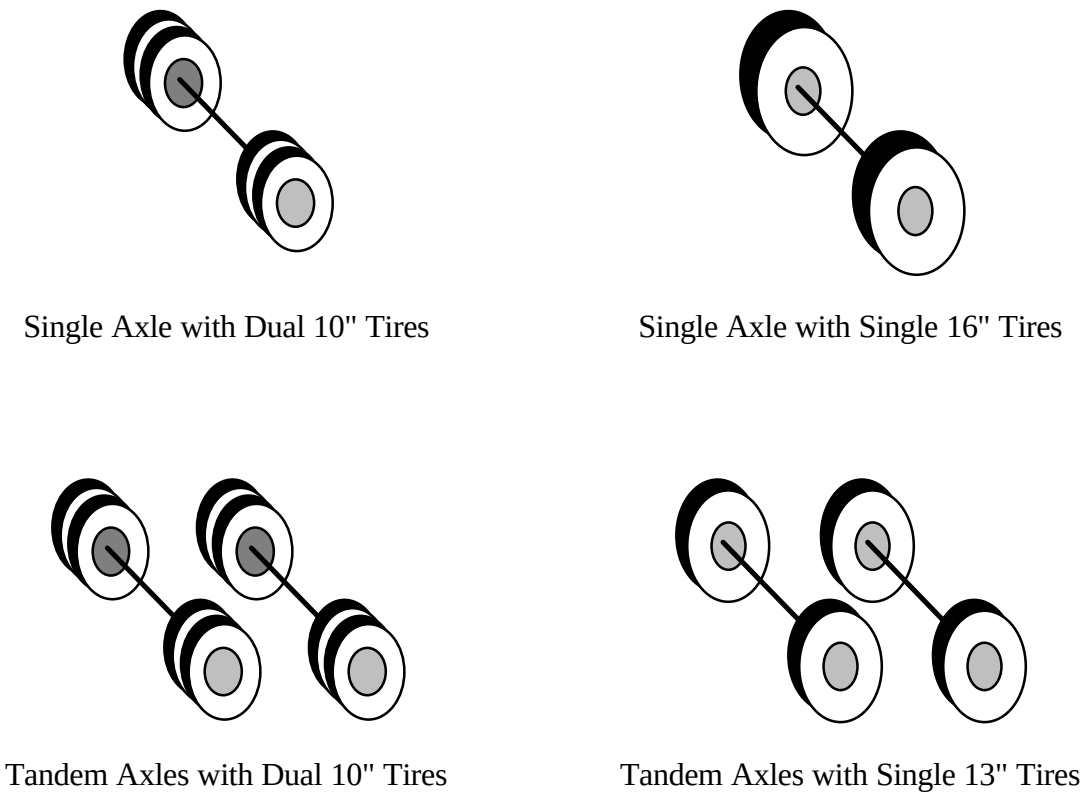


Figure 2.1. Tire-Axle Combinations [2.1]

1.3.2 FEDERAL AND TYPICAL STATE AXLE LOAD LIMITS

1.3.2.1 Axle Limits

(a) Single axles: 20,000 lb (89 kN)
(b) Tandem axles: 34,000 lb (151 kN)
(c) Total truck gross weight: 80,000 lb (356 kN)

The range of weight limits for SHAs in the U.S. (including the District of Columbia, hopefully such statistics do not help qualify D.C. as a state) vary a bit. As an illustration, the maximum and minimum limits will follow (data current as of January 1989 — source the American Trucking Association).

- Single axle
  - Maximum allowable = 22,400 lb (100 kN) (Connecticut, Massachusetts, New Jersey, Rhode Island, Vermont)
  - Minimum allowable = 20,000 lb (89 kN) (43 SHAs)
  - WSDOT = 20,000 lb (89 kN)
- Tandem axle
  - Maximum allowable = 44,000 lb (196 kN) (Florida, Rhode Island)
  - Minimum allowable = 34,000 lb (151 kN) (40 SHAs)
  - WSDOT = 34,000 lb (151 kN)
- Tridum axle
  - Maximum allowable = 66,000 lb (294 kN) (Florida)
  - Minimum allowable = 34,000 lb (151 kN) (California, Indiana, Mississippi, Missouri, New Hampshire, Tennessee)
  - WSDOT = 42,000 lb (187 kN)
- Tire load
  - Maximum allowable = 800 lb/inch width (Indiana, Massachusetts, New Jersey, New York, Pennsylvania)
  - Minimum allowable = 550 lb/inch width (Alaska, Mississippi, N. Dakota)
  - WSDOT (maximum) = 600 lb/inch (105 N/mm) width (steer axle); otherwise 500 lb/inch (88 N/mm)

- Maximum allowable gross weight
  - Interstate
    - 50 SHAs = 80,000 lb (356 kN)
    - Michigan = 164,000 lb (730 kN)
    - WSDOT = 80,000 lb (356 kN)
  - Other Roads
    - Maximum = 164,000 lb (730 kN) (Michigan)
    - Minimum = 73,280 lb (326 kN) (Illinois, Minnesota, Missouri)
    - WSDOT = 80,000 lb (356 kN)

1.3.2.2 FHWA Bridge Formula [after Ref. 2.6, 2.22]

(a) Introduction

The total gross weight in pounds imposed on the pavement by any group of two or more consecutive axles on a vehicle or combination of vehicles shall not exceed that weight calculated by use of Equation 2.1 below. The bridge formula is needed since an individual set of bridge design computations cannot be done for every type of truck that may use highways.

Bridge designers use a standard design vehicle for estimating critical stresses, strains, or deflections in a bridge structure. This vehicle is designated HS-20-44 and has been referred to as an umbrella loading as illustrated in Figure 2.2 [2.6]. This truck was adopted for design in 1944 with the specific axle weights and spacings shown in Figure 2.2. Federal law requires its use in bridge design for the Interstate system.

In effect, the bridge formula helps to ensure bridges are not "overstressed" due to almost infinite number of truck-axle configurations and weights.

(b) Formula

$$W = 500(NL/(N-1) + 12N + 36) \qquad \text{Eq. 2.1}$$

Where W = maximum weight on any group of two or more consecutive axles to the nearest 500 lb,

L = distance between the extremes of any group of two or more consecutive axles, ft,

N = number of axles in the group under consideration.

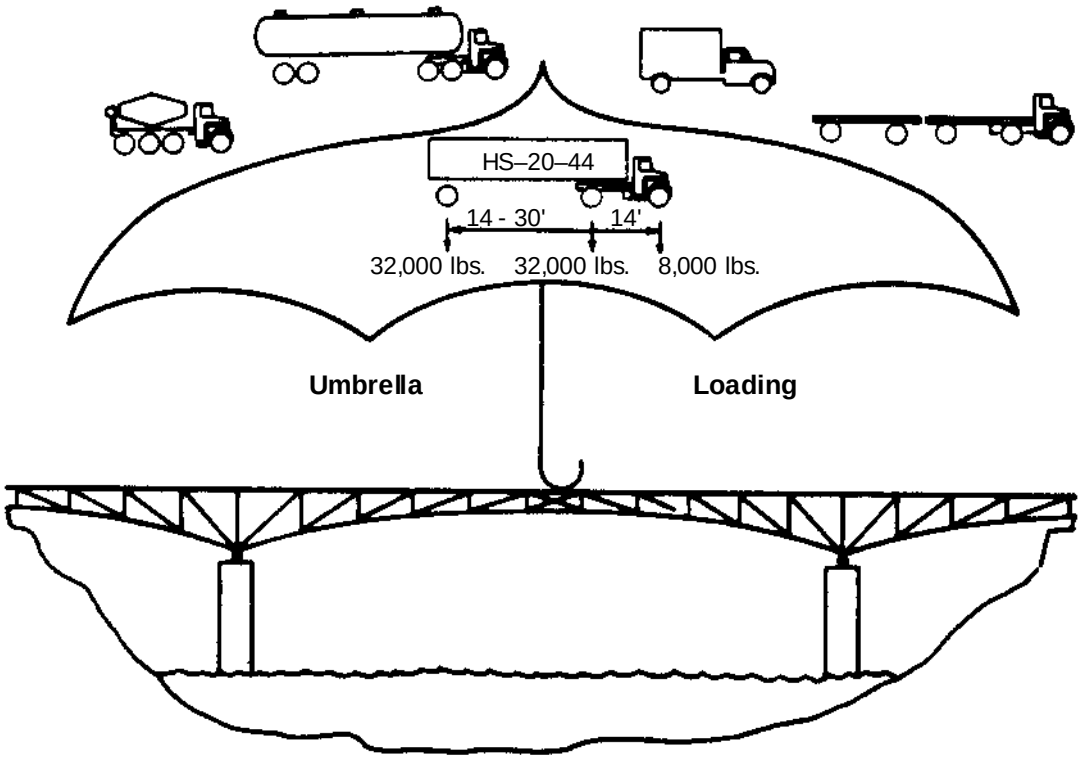


Figure 2.2. Standard Bridge Design Vehicle [after Ref. 2.6]

(c) Example 1

- Five axle truck
  - One single axle (steering axle) - 51 ft separation from steering axle to rear portion of back tandem
  - Two tandem axles (34 ft separation for tandems)
    - Each set centered 4 ft apart
- W = 500(5(51)/(5-1) + 12(5) + 36)
- = 80,000 lb for the group of axles - steering to rear tandem

(d) Example 2

A partial definition of a tandem axle (see Appendix 2.1) is any two axles whose centers are more than 40 in. but not more than 96 in. apart. We know the maximum allowable load on a tandem is 34,000 lb. Calculate the allowable load based on the bridge formula (initially use 96 in. or 8 ft).

- W = 500(2(8)/(2-1) + 12(2) + 36)
- = 500(76) = 38,000 lb.

Actually, a tandem axle spacing of 48 in. or 4 ft calculates to 34,000 lb.

- W = 500(2(4)/(2-1) + 12(2) + 36)

What the above illustrates, if two axles are spaced 97 in. apart for example, then the allowable load on those two axles is 38,000 lb.

(e) Example 3

A partial definition of a tridum axle (again, see Appendix 2.1) is three axles whose extreme centers are not more than 144 in. (12 ft) apart. Calculate the allowable loading according to the bridge formula:

- = 500(68) = 34,000 lb.

- W = 500(3(12)/(3-1) + 12(3) + 36)
- = 500(90) = 45,000 lb.

1.3.3 TIRE LOADS AND TERMINOLOGY

1.3.3.1 Tire Loads

States generally have regulations limiting allowable load per inch width of tire. The States of Washington, Oregon and Idaho have adopted a standard of 600 lb per inch (105 N/mm) width. Based on a slightly dated survey [2.2], this tire load limitation varies from a high of 800 lbs/inch (140 N/mm) (New York, etc.) to a low of 450 lbs/inch (79 N/mm) (Louisiana). In 1993, the state of Washington further restricted the allowable lb per inch width to 500 lb (88 N/mm) per inch width, with the exception that the steering axle tires can be 600 lb per inch (105 N/mm) width.

1.3.3.2 Truck tire definitions [after Reference 2.3]

(a) Tire Terminology [after Refs. 2.3, 2.38]

- Section Width

  Width of the tire at its widest point (outer sidewall to other sidewall) including "growth" due to inflation.

- Minimum Dual Spacing

  Minimum distance from the center of one tire assembly to another on a dual tire axle. Minimum spacing precludes tire rubbing, excessive heat generation, and stone damage (those caught between tires). This spacing has implications in pavement analyses since the center-to-center tire spacing must be modeled.

- Nominal Rim Diameter

  Used, along with Section Width, to describe tire sizes.

(b) Tire Sizes

Refer to Figures 2.3, 2.4 and 2.5.

For example, if a bias ply truck tire has a designated tire size of 12.00 - 24, then the section width is approximately 12 in. and the nominal rim diameter is 24 in. For truck tires, the design rim width is about 2.5 to 5.0 in. less than the section width (difference depends on tire size).

Data published in Land Line Magazine (January/February 1992) based on a survey of members in the Owner-Operator Independent Drivers Association (OOIDA) revealed the following:

- 58 percent stated that they use 11 x 24.5 tires.
- 19 percent use 11 x 22.5 tires.
- Rim diameters of 24.5 are preferred by certain flatbed and tanker fleets especially in urban delivery situations (turning and curb hopping).
- Rim diameters of 22.5 enhance high cube operations (recall 13.5 ft (4.1 m) is the maximum truck height). Additionally, they lower the vehicle center of gravity a bit.

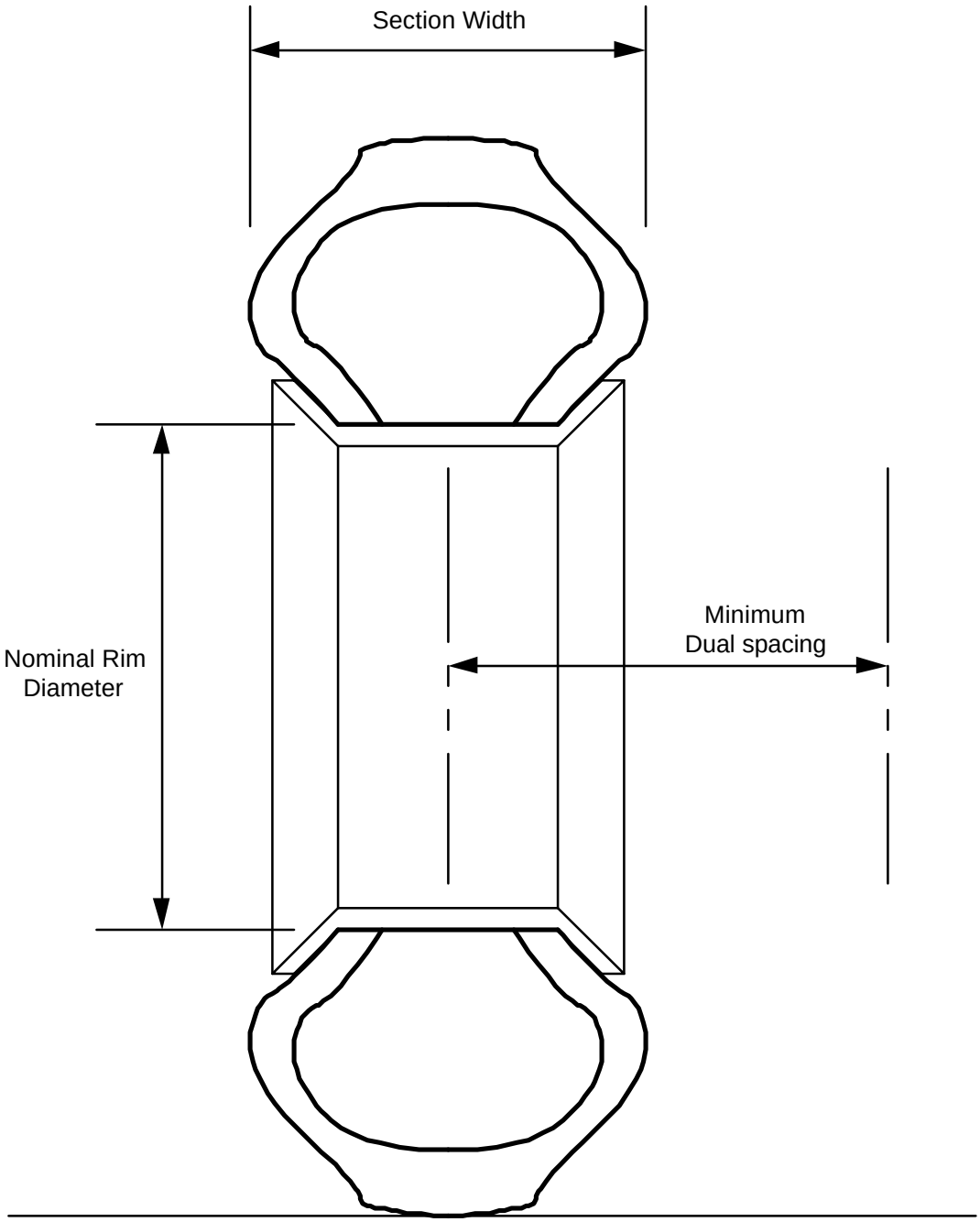


Figure 2.3. Principal Tire Dimensions

Typical Minimum Dual Spacing (from Ref 2.3)

Tire Size			Minimum Center-to-Center Spacing (in. (mm))	
10	R	22.5	11.4	(289.6)
11	R	22.5	12.6	(320.0)
11	R	24.5	12.6	(320.0)
12	R	22.5	13.5	(342.9)
12	R	24.5	13.5	(342.9)
12.5	R	22.5	13.6	(345.4)
12.5	R	24.5	13.6	(345.4)

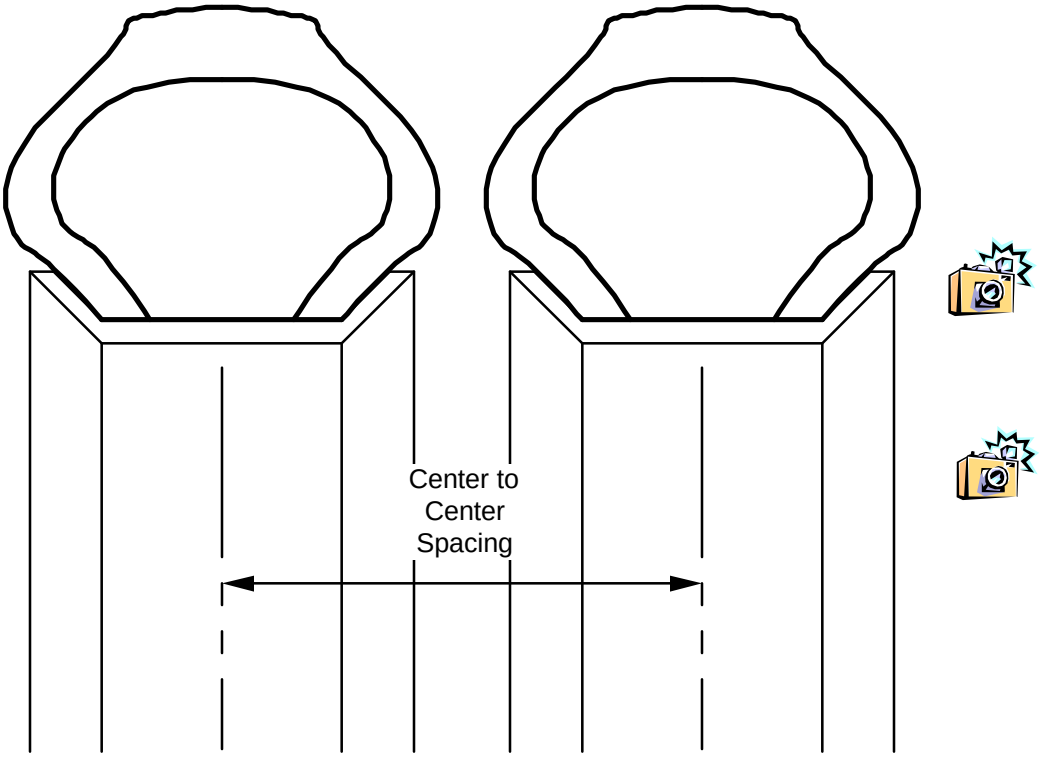


Figure 2.4. Dual Tire Spacing

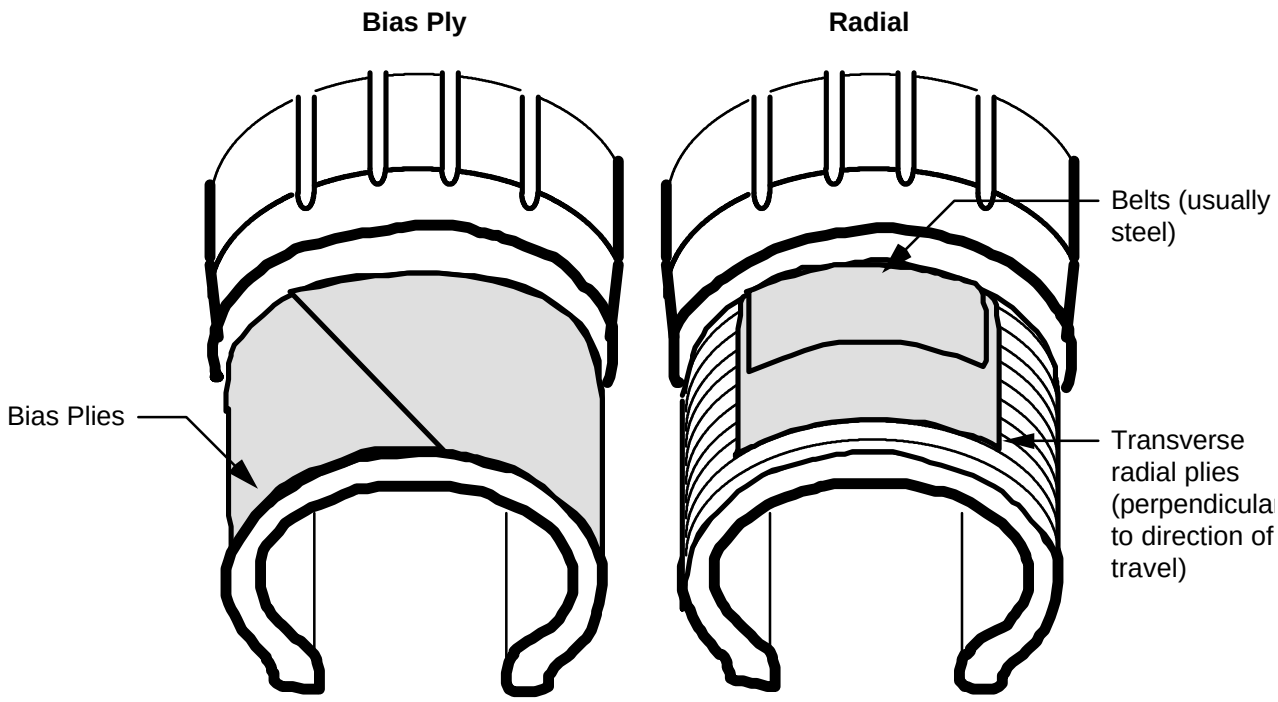


Figure 2.5. Sketches of Tire Construction – Bias Ply and Radial (sketched after material contained in Ref. 2.38)

- Wide-base tires ("super singles") have peaked in popularity for "over-the-roaders."
    - They provide lowest tare weights for bulk haulers (in lieu of dual tires).
    - Can provide less traction on slick pavements as compared to duals.
    - Little margin of safety on "spread tandems" whereby each axle is loaded to 20,000 lb (89 kN).
    - OOIDA aware of concern about potential for increased rutting with some wide-base tires.

(c) Bias and radial ply tires

Refer to Figure 2.5

- Bias ply tires: plies (cords) laid diagonally during manufacture.
- Radial ply tires: plies (cords) laid radially during manufacture which results in a more flexible tire wall.
- About 20 percent of truck fuel consumption is due to rolling resistance. Thus, a 5 percent reduction in rolling resistance will produce a 1 percent fuel saving (after Fitch [2.37]). It is reasonable to expect a reduction in rolling resistance of about 30 to 40 percent, thus a 6 to 8 percent fuel savings.
- As bias ply tires rotate and deflect, the bias (or crossed) plies interact and generate heat [2.5] which accelerates tire "aging." The cross plies stiffen the tire sidewalls which provide less "give" under load (as compared to radial tire construction).
- There is some evidence to suggest that radial and bias ply tires track somewhat differently. Interviews with truck drivers and truck tire manufacturers suggest that
    - Radials tend to track more consistently in a wheeltrack, particularly if the wheeltrack has an existing depression or rut.
    - These tracking differences have been attributed to the ply layout (the bias (cross) plies have greater tendency to "crawl" out of a wheeltrack depression than a radial ply constructed tire).
    - The net effect is that radial tires, due to ply layout, likely contribute to some of the wheeltrack rutting observed mostly on WSDOT Interstate highways.

1.3.4 TRUCKS AND BUSES [AFTER REFERENCE 2.4]

1.3.4.1 Truck and bus population in U.S.

(a)	Light trucks	30,900,000	(87%)
(b)	Medium trucks and buses	3,100,000	(9%)
(c)	Heavy trucks and buses	1,500,000	(4%)
		35,500,000	(100%)

1.3.4.2 Common practice in classifying trucks is by gross vehicle weight rating. Three broad categories:

(a) Light: pickup trucks and vans
(b) Medium: city delivery trucks (for example)
(c) Heavy: over-the-road tractor-trailer combinations.

1.3.4.3 Vehicle manufacturers use eight classes according to gross vehicle weight rating (GVWR):

(a) Classes 1, 2, 3 (light trucks)

GVWR: up to 14,000 lb (62 kN) (types: pickup trucks, vans, ambulances, parcel delivery)

(b) Classes 4, 5, 6, 7 (medium trucks and buses)

GVWR: 14,001 to 33,000 lb (62 to 147 kN) (types: city cargo van, beverage delivery truck, wrecker, school bus)

(c) Class 8 (heavy trucks and buses)

GVWR: 33,001 lb (147 kN) and over (types: truck tractor, concrete mixer, dump truck, fire truck, city transit bus)

Note: The above classes are not the same as used by the FHWA (see Paragraph 1.6.1).

(d) Other definitions

(i) Gross Vehicle Weight Rating (GVWR): weight specified by manufacturer as the maximum loaded weight (truck plus cargo) of a single vehicle.

(ii) Gross Combination Weight Rating (GCWR): maximum weight specified by the manufacturer as the loaded weight of a combination vehicle (two or more units).

1.4 TIRE AND CONTACT PRESSURE

1.4.1 GENERAL

For most pavement analyses it is assumed that the tire load is uniformly applied over a circular area. It is generally assumed that tire inflation and contact pressures are the same (not true but adequate for approximations).

1.4.2 TIRE INFLATION PRESSURES

1.4.2.1 Calculation

Load = (pressure)(area)

and $a = \sqrt{\frac{P}{p\pi}}$

where a = radius of tire contact,
P = total load on tire, and
p = tire inflation pressure.

1.4.2.2 Typical values for highway truck tires

(a) A survey conducted in 1982 [2.2] revealed average truck tire inflation pressures of about 95 psi.

(b) Unpublished data (owner-operator survey of national practice) provided by the Owner Operator - Independent Drivers Association of America, Oak Grove, Missouri (March 1987) resulted in the following statistics:

(i) Steering axle tires

- 90% use radial ply tires
- 10% use bias ply tires
- Average inflation pressure = 101 psi (693 kPa)
- Minimum inflation = 80 psi (552 kPa) (1% of survey)
- Maximum inflation = 120 psi (827 kPa) (2% of survey)

(ii) Drive axle tires

- 92% use radial ply tires
- 8% use bias ply tires
- Average inflation pressure = 99 psi (683 kPa)
- Minimum inflation = 80 psi (552 kPa) (2% of survey)
- Maximum inflation = 120 psi (827 kPa) (1% of survey)

(iii) Trailer axle tires

- 87% use radial ply tires
- 13% use bias ply tires
- Average inflation pressure = 97 psi (669 kPa)
- Minimum inflation = 80 psi (552 kPa) (3% of survey)
- Maximum inflation = 120 psi (827 kPa) (1% of survey)

1.5 REPETITIONS OF WHEEL LOADS

1.5.1 GENERAL

We must be able to convert wheel loads of various magnitudes and repetitions ("mixed traffic") to an equivalent number of "standard" or "equivalent" loads for design purposes. The most commonly used equivalent load is 18,000 lb (80 kN) equivalent single axle loads (normally designated ESAL). The ESAL standard axle load is used in the AASHTO "Guide for Design of Pavement Structures".

1.5.2 AASHTO LOAD EQUIVALENCIES (18,000 LB. (80 KN) ESALS)

Wheel load equivalency has been one of the most widely adopted results of the AASHO Road Test (1958 to 1960), i.e., to relate relative damage attributed to axles of different type (single and tandem) and weight. A variety of equivalency factors can be used depending on the pavement section which is defined by a Structural Number (SN) and the terminal serviceability index (p_t).

Highway design in most states is based on the ESAL traffic input anticipated over a future 10 to 40 year period.

The relationship between repetitions is not arithmetically proportional to the axle loading. Instead, a 10,000 lb (44.4 kN) single axle needs to be applied to a pavement structure many more than 1.8 times the number of repetitions of an 18,000 lb (80 kN) single axle to have the same effect — in fact, more than 12 times. Similarly, a 22,000 lb (97.8 kN) single axle needs to be repeated less than half the number of times of an 18,000 lb (80 kN) single axle to have an equivalent effect.

1.5.3 EXAMPLE

(a) Sample of ESALs (typical values)

Axle Type (lbs)	Axle Load (lbs)	ESAL Equivalency Factor [from Ref. 2.7]
Single axle	2,000	0.0003
	10,000	0.118
	14,000	0.399
	18,000	1.000
	20,000	1.4
	30,000	7.9
Tandem axle	2,000	0.0001
	10,000	0.011
	14,000	0.042
	18,000	0.109
	20,000	0.162
	30,000	0.703
	34,000	1.11
	40,000	2.06
	50,000	5.03

1.5.4 GENERAL OBSERVATIONS BASED ON EQUIVALENCY FACTORS

(a) A 18,000 lb (80 kN) single axle does over 3,000 times more damage to a pavement than a 2,000 lb (8.9 kN) single axle (1.000/0.0003 ≅ 3,333).

(b) A 30,000 lb (133.3 kN) single axle does about 67 times more damage than a 10,000 lb (44.4 kN) single axle (7.9/0.118 ≅ 67).

(c) A 30,000 lb (133.3 kN) single axle does about 11 times more damage than a 30,000 lb (133.3 kN) tandem axle (7.9/0.703 ≅ 11).

1.5.5 EXAMPLE — LOG TRUCK EQUIVALENT DAMAGE

Assume a log truck has three axles:

(a) Truck tractor

- Steering axle (single axle) = 14,000 lb (62.2 kN)
- Drive axle (tandem axle) = 34,000 lb (151.1 kN)

(b) Trailer

- Pole trailer axle (tandem axle) = 30,000 lb (133.3 kN)

(c) The total equivalent damage by this truck is (p_t = 3.0, SN = 3):

• Steering axle @ 14,000 lb	=	0.47 ESAL
• Drive axle @ 34,000 lb	=	1.15 ESAL
• Pole axle @ 30,000 lb	=	0.79 ESAL
Total	=	2.41 ESAL

(d) If a pavement was subjected to 100 of these trucks each day (in one direction) for 20 years (5 days per week), the total ESAL for this truck would be

(5 day/7 day)(365 days/year)(20 years)(100 tk/day)(2.41 ESAL/tk)
= 1,256,643 ESAL ≅ 1,300,000 ESALs

1.5.6 HOW TO CALCULATE ESAL FACTORS

1.5.6.1 Flexible Pavements

(a) Again, ESALs indicate the relative damage to a pavement structure due to various axle loads (i.e., mixed traffic).

(b) The basic equation derived from the results of the AASHO Road Test (more on this later) for flexible pavement follows:

$$\frac{W_x}{W_{18}} = \left[\frac{L_{18} + L_{2s}}{L_x + L_{2x}}\right]^{4.79} \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] \left[L_{2x} \right]^{4.33}$$

where W = axle applications

$\frac{W_x}{W_{18}}$ = inverse of equivalency factors (where W_{18} = number of 18,000 lb (80 kN) single axle loads)

L_x = axle load being evaluated (kips)

L_{18} = 18 (standard axle load in kips)

L_2 = code for axle configuration

1 = single axle
2 = tandem axle
3 = triple axle (added in the 1986 *AASHTO Guide*)
x = axle load equivalency factor being evaluated
s = code for standard axle = 1 (single axle)

$$G = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right) =$$ a function of the ratio of loss in serviceability at time t to the potential loss taken at a point where $p_t = 1.5$

p_t = "terminal" serviceability index

$$\beta = 0.4 + \frac{0.081\,(L_x + L_{2x})^{3.23}}{(SN + 1)^{5.19}\,L_{2x}^{3.23}} =$$ function which determines the general shape of serviceability trend with increasing axle load applications

SN = structural number (formerly called a "thickness index")

"Standard Axle"

L_{18} = 18
L_{2s} = 1

(c) Example Calculation of Equivalency Factor—Single Axle

- Assumptions: Single axle, 30,000 lb (133 kN), SN = 3, $p_t = 2.5$
- Answer: (Table D.4, p. D-6, *AASHTO Guide*) = 7.9
- Calculations

$$\frac{W_{30}}{W_{18}} = \left[\frac{18 + 1}{L_{30} + L_{2x}}\right]^{4.79} \left[\frac{10^{G/\beta_{30}}}{10^{G/\beta_{18}}}\right] \left[\,L_{2x}\,\right]^{4.33}$$

where W_{18} = predicted number of 18,000 lb (80 kN) <u>single</u> axle load applications,

W_{30} = predicted number of 30,000 lb (133 kN) <u>single</u> axle load applications,

$L_x = L_{30} = 30$

$L_{2x} = 1$

G = serviceability loss factor

$$= \log\left(\frac{4.2 - 2.5}{4.2 - 1.5}\right) = -0.2009$$

β_{30} = curve slope factor

$$= 0.4 + \frac{0.081\,(30 + 1)^{3.23}}{(3 + 1)^{5.19}\,(1)^{3.23}} = 4.388$$

and $G/\beta_{30} = -0.2009/4.388 = -0.04578$

$$\beta_{18} = 0.4 + \frac{0.081\,(18 + 1)^{3.23}}{(3 + 1)^{5.19}\,(1)^{3.23}} = 1.2204$$

and $G/\beta_{18} = -0.2009/1.2204 = -0.1646$

Thus, $$\frac{W_{30}}{W_{18}} = \left[\frac{18 + 1}{30 + 1}\right]^{4.79} \left[\frac{10^{-0.04578}}{10^{-0.1646}}\right] \left[1\right]^{4.33} = 0.1260$$

and $\frac{W_{30}}{W_{18}} \cong 12.6\%$ of W_{18} loads allowable with a 30,000 lb single axle

Finally, Load Equivalency Factor $= \frac{1}{0.1260} = 7.9365 \cong 7.9$
(same as contained in *AASHTO Guide* — Appendix D)

(d) Example Calculation of Equivalency Factor—Tandem Axle

- Assumptions: Tandem axle, 40,000 lb (133 kN), SN = 5, $p_t = 2.5$
- Answer: (Table D.5, p. D-7, *AASHTO Guide*) = 2.08
- Calculations

$$\frac{W_{40}}{W_{18s}} = \left[\frac{L_{18} + L_{2s}}{L_{40} + L_{2x}}\right]^{4.79} \left[\frac{10^{G/\beta_{40}}}{10^{G/\beta_{18}}}\right] \left[L_{2x}\right]^{4.33}$$

where L_{40} = 40 (tandem axle)

L_{18} = 18 (single axle)

L_{2x} = 2 (tandem axle)

L_{2s} = 1 (single axle)

$$G = \log\left(\frac{4.2 - 2.5}{4.2 - 1.5}\right) = -0.2009$$

$$\beta_{40} = 0.4 + \frac{0.081\,(40 + 2)^{3.23}}{(5 + 1)^{5.19}\,(2)^{3.23}} = 0.53824$$

$$\beta_{18} = 0.4 + \frac{0.081\,(18 + 1)^{3.23}}{(5 + 1)^{5.19}\,(1)^{3.23}} = 0.50006$$

$$G/\beta_{40} = -0.2009/0.53824 = -0.37325$$

$$G/\beta_{18} = -0.2009/0.50006 = -0.40175$$

$$\frac{W_{40}}{W_{18}} = \left[\frac{18 + 1}{40 + 2}\right]^{4.79} \left[\frac{10^{-0.37325}}{10^{-0.40175}}\right] \left[2\right]^{4.33} = 0.48064$$

Finally, Load Equivalency Factor = $\frac{1}{0.48064} = 2.08$

(same as contained in *AASHTO Guide* — Appendix D)

1.5.6.2 Rigid Pavements

(a) The basic equation for calculating load equivalency factors for rigid pavements follows:

$$\frac{W_x}{W_{18}} = \left[\frac{L_{18} + L_{2s}}{L_x + L_{2x}}\right]^{4.62} \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] \left[L_{2x}\right]^{3.28}$$

where $\frac{W_x}{W_{18}}$, L_x, L_2, p_t as for flexible pavements

$$G = \log\left(\frac{4.5 - p_t}{4.5 - 1.5}\right)$$

$$\beta = 1.00 + \frac{3.63\,(L_x + L_{2x})^{5.20}}{(D + 1)^{8.46}\,L_{2x}^{3.52}}$$

(b) Example Calculation of Equivalency Factor

- Assumptions: Single axle, 30,000 lb (133 kN), D = 7 in., p_t = 2.5
- Answer: (Table D.13, p. D-15, *AASHTO Guide*) = 7.7
- Calculations

$$\frac{W_{30}}{W_{18}} = \left[\frac{18 + 1}{L_{30} + L_2}\right]^{4.62} \left[\frac{10^{G/\beta_{30}}}{10^{G/\beta_{18}}}\right] \left[\,L_2\,\right]^{3.28}$$

where W_{18} = predicted number of 18,000 lb (80 kN) single axle load applications,

W_{30} = predicted number of 30,000 lb (133 kN) single axle load applications,

L_x = L_{30} = 30

L_{2x} = 1 (single axle)

$$G = \log\left(\frac{4.5 - 2.5}{4.5 - 1.5}\right) = -0.1761$$

$$\beta_{30} = 1.00 + \frac{3.63\,(30 + 1)^{5.20}}{(7 + 1)^{8.46}\,(1)^{3.52}} = 5.7298$$

and G/β_{30} = -0.1761/5.7298 = -0.03073

$$\beta_{18} = 1.00 + \frac{3.63\,(18 + 1)^{5.20}}{(7 + 1)^{8.46}\,(1)^{3.52}} = 1.3709$$

and G/β_{18} = -0.1761/1.3709 = -0.12845

Thus, $$\frac{W_{30}}{W_{18}} = \left[\frac{18 + 1}{30 + 1}\right]^{4.62} \left[\frac{10^{-0.03073}}{10^{-0.12845}}\right] \left[\,1\,\right]^{3.28} = 0.1305$$

and $\frac{W_{30}}{W_{18}} \simeq$ 13.05% of W_{18} loads allowable with a 30,000 lb single axle

Load Equivalency Factor = $\frac{1}{0.1305}$ = 7.67≃ 7.7

1.5.7 GENERALIZED FOURTH POWER LAW

(a) Approximate load equivalency factor $\cong \left(\frac{\text{axle load in question}}{\text{18,000 lb (80 kN) single axle load}}\right)^4$
(i.e., the 4th power law)

(b) Example 1

Use prior example (from Paragraph 1.5.5.3)

A 18,000lb (80 kN) single axle, LEF =1.0
A 30,000lb (133kN) single axle, LEF = 7.9

and $\frac{LEF_{30}}{LEF_{18}} = \frac{7.9}{1.0} = 7.9$

and $\left(\frac{30{,}000\,\text{lb}}{18{,}000\,\text{lb}}\right)^4 = 7.7$

Thus, the two estimates are approximately equal.

(c) Example 2

Using a SN = 5 and p_t = 2.5 (and Appendix D of the *AASHTO Guide*),

A 10,000lb (44.4kN) single axle, LEF =0.09
A 20,000lb (88.9kN) single axle, LEF = 1.51

and $\frac{LEF_{20}}{LEF_{10}} = \frac{1.51}{0.09} = 16.8$

and $\left(\frac{20{,}000\,\text{lb}}{10{,}000\,\text{lb}}\right)^4 = 16$

Now, try comparing 10,000 lb (44.4 kN) and 40,000 lb (177.8 kN) single axles.

A 40,000lb (177.8kN) single axle, LEF =21.1

and $\frac{LEF_{40}}{LEF_{10}} = \frac{21.1}{0.09} = 234$

and $$\left(\frac{40{,}000\text{ lb}}{10{,}000\text{ lb}}\right)^4 = 256$$

1.6 WSDOT METHODS FOR ESTIMATING ESALs

1.6.1 FORECASTING TRUCK AND BUS TRAFFIC

A basic element in estimating the future ESALs for a specific project is to forecast the truck and bus volumes for the design (and analysis) period. Once this is done, LEFs in various forms can be applied to the forecast volumes and summed. The *WSDOT Traffic Forecasting Guide* [2.20] can be used for this purpose. Generally, the Regional Materials Engineer or Materials Laboratory personnel will not be responsible for the forecast volumes.

A complete forecast will include the 13 FHWA vehicle classes (which are not the same vehicle classes as used by vehicle manufacturers). These classes are as follows [from Ref. 2.20]:

Class 1 *Motorcycles* (Optional) — All two- or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handle bars rather than wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles. This vehicle type may be reported at the option of the State.

Class 2 *Passenger Cars* — All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.

Class 3 *Other Two-Axle, Four-Tire Single Unit Vehicles* — All two-axle, four tire, vehicles, other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, and carryalls. Other two-axle, four-tire single unit vehicles pulling recreational or other light trailers are included in this classification.

Class 4 *Buses* — All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. All two-axle, four-tire single unit vehicles. Modified buses should be considered to be a truck and be appropriately classified.

Class 5 *Two-Axle, Six-Tire, Single Unit Trucks* — All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., having two axles and dual rear wheels.

Class 6 *Three-Axle Single Unit Trucks* — All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., having three axles.

Class 7 *Four or More Axle Single Unit Trucks* — All trucks on a single frame with four or more axles.

Class 8 *Four or Less Axle Single Trailer Trucks* — All vehicles with four or less axles consisting of two units, one of which is a tractor or straight truck power unit.

Class 9 *Five-Axle Single Trailer Trucks* — All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.

Class 10 *Six or More Axle Single Trailer Trucks* — All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.

Class 11 *Five or Less Axle Multi-Trailer Trucks* — All vehicles with five or less axles consisting of three or more units, one of which is a tractor or straight truck power unit.

Class 12 *Six-Axle Multi-Trailer Trucks* — All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.

Class 13 *Seven or More Axle Multi-Trailer Trucks* — All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

Note: In reporting information on trucks the following criteria should used:

a. Truck tractor units traveling without a trailer will be considered single unit trucks.

b. A truck tractor unit pulling other such units in a "saddle mount" configuration will be considered as one single unit truck and will be defined only by the axles on the pulling unit.

c. Vehicles shall be defined by the number of axles in contact with the roadway. Therefore, "floating" axles are counted only when in the down position.

d. The term "trailer" includes both semi- and full trailers.

The WSDOT PMS uses a TRIPS summary for three basic truck designations (excludes FHWA Classes 1, 2, 3):

(a)	Single Units which includes	(i)	Buses (FHWA Class 4)
		(ii)	2 axle, 6 tire single units (FHWA Class 5)
		(iii)	3 axle single units (FHWA Class 6)
		(iv)	4+ axle single units (FHWA Class 7)
(b)	Single Trailers ("Double Units)" which includes	(i)	4 axle single trailer (FHWA Class 8)
		(ii)	5 axle single trailer (FHWA Class 9)
		(iii)	6+ axle single trailer (FHWA Class 10)
(c)	Multi-Trailers ("Multiple Units" or "Trains") which includes	(i)	5 axle multi-trailer (FHWA Class 11)
		(ii)	6 axle multi-trailer (FHWA Class 12)
		(iii)	7+ axle multi-trailer (FHWA Class 13)

1.6.2 LOAD EQUIVALENCY FACTOR (LEF) APPROXIMATIONS

To translate truck and bus volumes to ESALs, it is of value to have LEF approximations in terms of ESALs/axle or ESAL/vehicle. To do this, static scale loadometer data summarized in W-4 Tables from 1960 to 1983 were examined for Washington state. This includes weigh stations for highway systems classified as

- Interstate-Rural (Final IR)
- Other Rural (Other MR)
- All Rural (all MR)
- All Urban (Other U)
- All Systems/Classes (All MR and U)

The various ESALs breakouts are shown in Table 2.1. All panel trucks and pickups were excluded from the calculations if they had two axles with four tires (i.e., two axle, six tire trucks or larger were used). The data in Table 2.1 is summarized according to single units, combinations (includes single trailer and multi-trailer), all trucks, and by axles. The ESAL calculations are for flexible pavements (LEFs from Appendix D, *AASHTO Guide*, SN = 5, p_t = 2.5) only.

The basic W-4 Table results are based on weight data from a limited number of weigh stations (typically 5 to 15) which operated for a maximum period of 24 hours for no more than five days per year. Thus, the samples and hence the summary in Table 2.1 may be biased (either high or low). With the advent of extensive WIM data, a substantially better estimate of ESAL factors can be made.

The above ESAL factors may appear to be "low"; however, about one-half of the trucks weighed at weigh stations were empty. Thus, an ESAL/axle factor = 0.25 corresponds to a single axle load of about 12,700 lb (56.5 kN) (assumes SN = 5, p_t = 2.5).

In urban areas, the loadometer data does not reflect weights of transit buses. Data provided by Metro (Municipality of Metropolitan Seattle) in 1987 [2.21] and DeBoldt [2.28] in 1993 provides some insight. This information is summarized in Table 2.2. (Metro as of March 1993 had a total of 1,214 buses in operation: 101 AM General Diesel, 138 AM General Trolley, 224 Flyer (40'), 35 Flyer (35'), 331 MAN Articulated (60'), 157 MAN (40'), and 228 Breda (60').)

Table 2.1. ESAL Factors from WSDOT W-4 Tables (1960-1983)

Highway System Weigh Classification	ESAL Factors and Year							
	ESALs/SU		ESALs/Comb.		ESALs/Truck		ESALs/Axle	
	Lowest	Highest	Lowest	Highest	Lowest	Highest	Lowest	Highest
Interstate-Rural	0.06 (1960)	0.28 (1963)	0.62 (1960)	1.16 (1979)	0.41 (1960)	1.12 (1979)	0.11 (1960)	0.23 (1979)
Other Rural	0.16 (1965)	0.63 (1978)	0.99 (1960)	1.41 (1976)	0.67 (1960)	1.37 (1978)	0.18 (1960)	0.28 (1978)
All Rural	0.15 (1965)	0.31 (1983)	0.85 (1962)	1.17 (1979)	0.61 (1962)	1.13 (1979)	0.17 (1971)	0.24 (1979)
All Urban	0.10 (1975)	0.22 (1980)	0.70 (1960)	1.18 (1980)	0.30 (1960)	0.98 (1980)	0.11 (1960)	0.23 (1980)
All Systems	0.14 (1960)	0.27 (1983)	0.86 (1971)	1.17 (1979)	0.58 (1960)	1.13 (1979)	0.16 (1960)	0.24 (1979)

The total ESALs per bus were obtained from the results shown in Table 2.2:

Bus	ESALs/Bus
• AM General Diesel	
• Empty	1.14
• 50% Full	1.67
• 100% Full	2.34
• 130% Full	2.85
• AM General Trolley	
• Empty	0.80
• 50% Full	1.22
• 100% Full	1.78
• 130% Full	2.19
• Flyer	
• Empty	0.96
• 50% Full	1.45
• 100% Full	2.11
• 130% Full	2.61

Bus	ESALs/Bus
• MAN 60'	
• Empty	0.84
• 50% Full	1.42
• 100% Full	2.20
• 130% Full	2.87
• Flexible Diesel	
• Empty	0.57
• 50% Full	0.94
• 100% Full	1.50
• 130% Full	1.92
• GM Diesel	
• Empty	0.58
• 50% Full	0.95
• 100% Full	1.46
• 130% Full	1.84

Table 2.2. Metro Bus Axle Weights and Associated ESALs

Bus Type	Total Empty Weight (lb)	Tire Size[3,4]	Seating Capacity	Weight (pounds) and ESAL per Axle											
				Empty			50% Full Pax			100% Full Pax			130% Full Pax		
				1st Axle	2nd Axle	3rd Axle	1st Axle	2nd Axle	3rd Axle	1st Axle	2nd Axle	3rd Axle	1st Axle	2nd Axle	3rd Axle
AM General Diesel[1]	26,600	12.5 x 22.5	45	8,200	18,400	—	9,925	20,200	—	11,350	22,000	—	12,295	23,088	—
				0.04	1.10	—	0.09	1.58	—	0.16	2.18	—	0.21	2.64	—
AM General Trolley[1]	24,740	12.5 x 22.5	45	7,980	16,760	—	9,555	18,560	—	11,130	20,360	—	12,075	21,440	—
				0.03	0.77	—	0.08	1.14	—	0.15	1.63	—	0.20	1.99	—
Flyer[1] 40'	26,300	12.5 x 22.5	47	8,850	17,450	—	10,500	19,325	—	12,150	21,200	—	13,140	22,325	—
				0.06	0.90	—	0.11	1.34	—	0.20	1.91	—	0.29	2.32	—
MAN[1] 60' Articulated	37,300	12 x 20	70	12,900	15,100	9,280	14,716	17,226	10,587	16,281	19,226	12,272	17,295	20,464	13,675
				0.27	0.50	0.07	0.45	0.85	0.12	0.68	1.31	0.21	0.87	1.67	0.33
Flxible Diesel[1]	22,770	12.5 x 22.5	51	7,410	15,360	—	9,210	17,310	—	11,010	19,410	—	12,090	20,625	—
				0.03	0.54	—	0.07	0.87	—	0.14	1.36	—	0.20	1.72	—
GM Diesel[1] 35'	21,640	12.5 x 22.5	48	6,020	15,620	—	7,670	17,570	—	9,320	19,520	—	10,310	20,690	—
				0.01	0.57	—	0.03	0.92	—	0.07	1.39	—	0.10	1.74	—
Flyer Diesel[1] 35[1]	24,470	12.5 x 22.5	39	7,420	17,050	—	8,770	18,625	—	10,120	20,200	—	10,930	21,145	—
				0.03	0.82	—	0.05	1.16	—	0.09	1.58	—	0.13	1.89	—
MAN[2] 40'	28,240	12.5 x 22.5	45	10,000	18,240	—	11,215	20,428	—	12,431	22,617	—	13,160	23,930	—
				0.09	1.18	—	0.15	1.65	—	0.23	2.44	—	0.29	3.00	—
Breda[2] 60' Articulated	49,330	12.75 x 22.5 (steer)	67	13,257	15,546	20,527	14,207	18,043	22,117	15,156	20,540	23,707	15,726	22,038	24,661
		12.5 x 22.5		0.30	0.56	1.67	0.39	1.01	2.23	0.51	1.69	2.91	0.59	2.20	3.38

[1] From Reference 2.21 (Metro)
[2] From Reference 2.28 (DeBoldt)
[3] Tire inflation pressures range from 95 to 115 psi
[4] Tire sizes may vary
Note: LEFs based on single axle, SN = 5, p_t = 2.5

Bus	ESALs/Bus
• Flyer Diesel	
• Empty	0.85
• 50% Full	1.21
• 100% Full	1.67
• 130% Full	2.02
• MAN 40'	
• Empty	1.27
• 50% Full	1.80
• 100% Full	2.67
• 130% Full	3.29

Bus	ESALs/Bus
• Breda 60'	
• Empty	2.53
• 50% Full	3.63
• 100% Full	5.11
• 130% Full	6.17

If no other information is known about a bus route other than the volume of buses, use an ESAL/bus corresponding to 50 percent full. This results in an average ESAL/bus ≅ 1.60.

To summarize the ESAL factors, Table 2.3 was prepared.

Table 2.3. Summary of Flexible Pavement ESAL Factors

Highway System	ESAL Factors				
	Single Units	Combination Units	Buses	Individual Axle	Overall Truck*
Interstate	0.30	1.20	1.60	0.25	1.10
Non-Interstate Rural	0.50	1.40	1.60	0.25	1.40
Non-Interstate Urban	0.25	1.20	1.60	0.25	1.00

*Excludes buses

1.6.3 CONVERSION OF FLEXIBLE PAVEMENT ESALS TO RIGID PAVEMENT ESALS

All of the approximate ESAL factors shown in Paragraph 1.6.2 were developed for flexible pavements. The corresponding ESAL factors for rigid pavements can be different. The 1993 AASHTO Guide, Part III, Chapter 5, Paragraph 5.2.3 recommends the use of a multiplier of 1.5 to convert flexible ESALs to rigid ESALs (or a multiplier of 0.67 to convert rigid ESALs to flexible ESALs).

A study conducted by Irick et al. for the Trucking Research Institute [2.62] was used to reanalyze the AASHO Road Test data with regard to estimation of LEFs. In general, the TRI study showed that LEFs for both flexible and rigid pavements should be larger for lighter loaded axles and smaller for heavier loaded axles as compared to AASHTO LEFs. Further, the LEFs for rigid pavements would be reduced for single axle loads greater than 18,000 lb and tandem axle loads greater than 34,000 lb (as compared to AASHTO). The following LEFs from the TRI study and AASHTO illustrate these points:

		LEFs for $p_t = 2.5$			
		Single Axle = 12,000 lb		Single Axle = 30,000 lb	
Pavement Type	SN or D	TRI	AASHTO	TRI	AASHTO
Flexible	3.06	0.25	0.21	5.6	7.8
	4.36	0.24	0.21	6.2	6.9
Rigid	8.0"	0.36	0.19	3.6	7.8
	9.5"	0.37	0.18	3.5	8.7

		Tandem Axle = 24,000 lb		Tandem Axle = 48,000 lb	
Pavement Type	SN or D	TRI	AASHTO	TRI	AASHTO
Flexible	3.06	0.34	0.32	3.5	4.2
	4.36	0.32	0.28	3.7	4.1
Rigid	8.0"	0.65	0.45	3.8	7.3
	9.5"	0.66	0.44	3.7	7.9

Thus, in general, for the heavier axle loads, the TRI study showed that single axle flexible LEFs would be somewhat higher than rigid LEFs. Tandem axle flexible and rigid LEFs would be about the same.

To further examine this issue of flexible and rigid LEFs, the following LEFs from the 1993 AASHTO Guide, Appendix D, are shown for a Terminal Serviceability Index of 3.0 and a typical range of truck single axle loads:

Single Axle Load (lb)	LEFs					
	SN = 3	D = 7"	$\frac{LEF_R}{LEF_F}$	SN = 5	D = 11"	$\frac{LEF_R}{LEF_F}$
10,000	0.168	0.095	0.57	0.101	0.081	0.80
12,000	0.296	0.202	0.68	0.212	0.174	0.82
14,000	0.468	0.378	0.81	0.391	0.337	0.86
16,000	0.695	0.640	0.92	0.651	0.600	0.92
18,000	1.000	1.000	1.00	1.000	1.000	1.00
20,000	1.41	1.47	1.04	1.44	1.58	1.10
22,000	1.96	2.06	1.05	1.97	2.38	1.21
24,000	2.69	2.81	1.04	2.60	3.47	1.33
26,000	3.65	3.77	1.03	3.33	4.87	1.46

Within the range of typical single axle loads observed on the WSDOT route system, the differences between LEFs for flexible and rigid pavements are small (as shown above). Such differences are much more pronounced for tandem axles as illustrated below:

Tandem Axle Load (lb)	LEFs					
	SN = 3	D = 7"	$\frac{LEF_R}{LEF_F}$	SN = 5	D = 11"	$\frac{LEF_R}{LEF_F}$
26,000	0.517	0.637	1.23	0.402	0.619	1.54
28,000	0.643	0.854	1.33	0.538	0.850	1.58
30,000	0.788	1.12	1.42	0.702	1.14	1.62
32,000	0.956	1.44	1.51	0.896	1.51	1.69
34,000	1.15	1.82	1.58	1.12	1.96	1.75
36,000	1.38	2.27	1.64	1.38	2.51	1.82

These ratios for tandem axles are a fundamental reason why AASHTO offers the approximate factor of 1.5 to convert flexible pavement ESALs to rigid pavement ESALs.

Contrary to the view that flexible and rigid ESALs are different, the Ontario Ministry of Transportation has chosen to calculate ESALs based on treating all axles as singles (e.g., a tandem axle would be treated as two single axles.) They use one set of LEFs which

are independent of axle spacing and pavement type. The Ontario approach was used to present the WSDOT WIM data which is shown in Paragraph 1.6.4.

1.6.4 WEIGH-IN-MOTION INFORMATION

An early sample of weigh-in-motion (WIM) data from WSDOT sites developed by Hallenbeck [2.39] provides additional insight into WSDOT route system truck traffic (1992-1993 data). As an illustration of the type of WIM data which is becoming available, the daily number of trucks counted and averaged over a one-year period at six of the sites are:

Site	Daily Truck Range (One Way)			
	Single Units	Double Units	Multiple Units	Total
SR5 Kelso	600 - 1,000	2,300 - 4,000	1,100 - 2,000	4,000 - 7,000
SR395 Pasco	100 - 200	1,000 - 1,300	400 - 500	1,500 - 2,000
SR90 Cle Elum	200 - 400	1,800 - 2,500	500 - 900	2,500 - 3,800
SR5 Seattle (185th)	1,000 - 1,300	1,500 - 2,200	300 - 700	2,800 - 4,000
SR522 Woodinville	400 - 600	500 - 700	100 - 200	1,000 - 1,500
SR195 Spokane	100 - 150	200 - 300	100 - 150	400 - 600

Notes: [1]Single units include FHWA Classes 4, 5, 6, and 7 (refer to Paragraph 1.6.1)
[2]Double units include FHWA Classes 8, 9, and 10
[3]Multiple units include FHWA Classes 11, 12, and 13

The above data provide some critical insight into the truck classes using the state system. For example, double units (tractor-trailers) generally constitute 50 to 70 percent of the total trucks. The single units tend to dominate (in numbers) over multiple units in urban areas and vice versa in rural areas.

The overall ESAL per vehicle class for ten WIM sites averaged over a one-year period (calculated by the Ontario approach (refer to Hajek [2.63]) are shown on the right. The Ontario approach treats all pavements the same (i.e., no separate LEFs for flexible and rigid pavements) and all axles as singles. This approach produces LEFs similar to AASHTO LEFs for single axles, SN = 5, p_t = 2.5.)

FHWA Vehicle Class	ESAL/ Vehicle
4	0.57
5	0.26
6, 7	0.42
8	0.30
9	1.20
10	0.93
11	0.82
12	1.06
13	1.39
Avg of All Classes	0.84

Based on the above ESALs per vehicle, the ESAL factors shown in Table 2.3 are reasonable if not a bit conservative. The above WIM based ESALs per vehicle are preliminary.

This WIM information can be contrasted against a 1931 truck survey reported by the Bureau of Public Roads (Peabody [2.40]). For Washington State, few routes had more than 100 trucks per day with a load-carrying capacity exceeding three tons (roughly equivalent to a GVW of about 18,000 lbs (80 kN) and would be classified as a "single unit" today). Those trucks with a load-carrying capacity of less than three tons (26.7 kN) constituted 82.5 percent of the truck traffic, with 17.5 percent having a capacity of three tons (26.7 kN) or more. Trucks with a capacity of 7.5 (66.7 kN) tons or more constituted only 0.7 percent of the total truck traffic.

1.6.5 WSPMS ESAL ESTIMATES

The traffic and ESAL estimates contained in the WSPMS currently use three vehicle types as follows:

WSDOT Vehicle Descriptor	FHWA Vehicle Classes	ESAL/ Vehicle
Singles	4, 5, 6, 7	0.40
Doubles	8, 9, 10	1.00
Trains	11, 12, 13	1.75

The total ESALs shown in the WSPMS are calculated assuming a vehicle annual growth rate which is project specific. For multi-lane facilities, a lane distribution factor is used in the cumulative ESALs displayed.

The initial analysis of the WSDOT WIM data [2.39] reveals the following ESALs per vehicle:

	WIM ESAL/Vehicle
Singles	0.37
Doubles	1.02
Trains	1.22

The above WIM results agree closely with the WSPMS ESALs per vehicle except for "trains." For the initial ten WIM sites analyzed, the ESAL per vehicle for trains ranged from a low of 0.43 to a high of 1.79.

1.6.6 WSDOT ESAL GROWTH RATES

Project specific ESAL growth rates, typically ranging between 0 to 4 percent annually, are used in the WSPMS; however, this can vary substantially. For example, I-5, MP 176.35, has experienced a growth from about 200,000 ESALs per year in 1965 (original construction) to about 1,000,000 ESALs per year in 1994. Thus, over a 30 year period, the ESALs per year have increased by a factor of five or an annual growth rate of about seven percent.

1.7 MAXIMUM ESAL LEVELS

Currently, the highest ESALs per year in the design lane are about 1,600,000 per year (I-5 in Tacoma, about MP 133, vicinity of SR 509 and SR 7). The maximum ESALs for a specific lane can be estimated by use of basic highway capacity rules. To estimate the maximum ESALs based on current typical trucks:

- Maximum passenger cars per lane per hour (speed ~ 35 mph) [2.30]:

  = 2000 passenger cars/hr/lane

- Equivalent number of passenger cars per truck or bus (rolling terrain) [2.30}:

  ≅ 4

- Maximum number of trucks per hour per lane:

  $$= \frac{2000}{4} = 500$$

  = 500 x 24 = 12,000 trucks/day/lane

- ESALs per day (non-Interstate Rural, Table 2.3)

$$= 12{,}000 \text{ x } 1.4 \frac{\text{ESALs}}{\text{tk}} = 16{,}800 \text{ ESALs/day}$$

- ESALs per year

$$= 16{,}800 \text{ ESALs/day x } 365 \frac{\text{days}}{\text{yr}} = 6{,}132{,}000 \text{ ESALs/yr}$$

Thus, the approximate upper ESAL limit for a variety of trucks (characterized by 1.4 ESALs per truck)

≈ 6,000,000 ESALs/ year

For comparison, if <u>all</u> trucks had five axles loaded to legal limits (an extreme example), then the maximum ESALs would be higher than 6,000,000 ESALs/year:

- Truck (p_t = 2.5, SN = 5)

• Steering axle @ 14,000 lb (62.3 kN)	=	0.36
• Drive axle (tandem) @ 34,000 lb (151.2 kN)	=	1.09
• Trailer axle (tandem) @ 34,000 lb (151.2 kN)	=	1.09
Total ESALs/truck	=	2.54

- ESALs per day

$$= 12{,}000 \text{ x } 2.54 \text{ ESALs/tk} = 30{,}480 \text{ ESALs/day}$$

- ESALs per year

$$= 30{,}480 \text{ ESALs/day x } 365 \frac{\text{days}}{\text{yr}} = 11{,}125{,}200 \text{ ESALs/yr}$$

≈ 11,000,000 ESALs/ year

<u>None</u> of the above "maximum" ESAL levels are expected on any state route; however, these values are provided more in the spirit of using such values to check ESAL estimates for active projects. If a specific ESAL estimate exceeds say 5,000,000 ESALs/year (or 200,000,000 ESALs for a 40 year design), then the estimate had best be carefully checked.

1.8 SWEDISH AXLE AND VEHICLE WEIGHTS

1.8.1 INTRODUCTION

A summary of axle and vehicle weights was provided by Bertil Mårtensson of RST Sweden AB during April 1992. It is interesting to see how Swedish conditions (somewhat more typical of Europe) both differs and agrees with the U.S. and Washington state.

In Sweden (and Europe in general), the term "truck" is defined as a vehicle with a gross weight exceeding 3.5 metric tons (7,700 lb). This corresponds to a Class 2 vehicle in the U.S. (6,001 - 10,000 lb). Recall that the vehicle classes in the U.S. are:

Class	Weight	
1	6,000 lb or less	(26.7 kN or less)
2	6,001 - 10,000 lb	(26.7 - 44.5 kN)
3	10,001 - 14,000 lb	(44.5 - 62.3 kN)
4	14,001 - 16,000 lb	(62.3 - 71.2 kN)
5	16,001 - 19,500 lb	(71.2 - 86.7 kN)
6	19,501 - 26,000 lb	(86.7 - 115.7 kN)
7	26,001 - 33,000 lb	(115.7 - 146.8 kN)
8	33,001 or more	(146.8 kN or more)

1.8.2 VEHICLE WEIGHTS

On April 1, 1990, the maximum allowable vehicle weight increased from 51.4 metric tons (113,300 lb) to 56 metric tons (123,500 lb). This vehicle has a maximum length of 24 m (78.7 ft), width of 2.6 m (8.5 ft) with a total of seven axles (3 + 4). International truck and trailer combinations with a total of six axles (3 + 3) have maximum gross vehicle weights of 44 metric tons (97,000 lb). In Sweden, the maximum single axle load is 10 metric tons (22,000 lb), a tandem is 16 metric tons (35,300 lb), and a tridum is 24 metric tons (52,900 lb).

1.8.3 AXLE WEIGHTS

During 1989, the Swedish National Road Administration used 11 WIM stations to assess "dynamic" vehicle and axle weights. Typically, the measured average axle weight ranged from 5.5 to 5.7 metric tons (12,130 to 12,600 lb). Further, this range of axle weights is about the same as observed during the 1960s and 1970s and therefore appears to be somewhat constant (similar to Washington state experience). (Thus, the typical Swedish 18,000 lb (80 kN) ESAL per axle is also about 0.25!)

The typical <u>seven axle</u> (3 + 4) Swedish truck has an average weight of 44 metric tons (97,000 lb) or an average axle weight of 6.3 metric tons (13,900 lb). The typical

international truck and trailer with six axles (3 + 3) has an average weight of 40 metric tons (88,200 lb) or an average axle weight of 6.7 metric tons (14,700 lb).

Trucks with two axles or truck and trailer combinations with either four or five axles ((2 + 2) or (2 + 3)) have lower average weights of about, on average, 4.9 metric tons (10,800 lb).

Overall, about 1 to 2 percent of all measured axles have weights higher than 10 metric tons (22,000 lb).

2. MATERIALS

2.1 INTRODUCTION

In this subsection, the material tests for CBR, R-value, elastic modulus, and k-value will be emphasized. The CBR test was used by WSDOT up to 1951. The R-value has been used for unstabilized materials from 1951 to now. Measures of elastic modulus are used by WSDOT for both new and rehabilitation design. The modulus of subgrade reaction (k) is used in the design of rigid pavements. The R-value will continue to be used in WSDOT specifications for unstabilized materials as a measure of strength. At the end of Paragraph 2, terminology and background information on subgrade soils are presented.

A few material considerations include the following:

(a) Efficiency of stress distribution characteristics

(b) Resistance to transient deflections, stresses or strains (which can result in cracking, etc.)

(c) Resilient (or elastic) properties such as resilient modulus.

(d) Resistance to permanent deformations (rutting).

(e) Susceptibility of materials to changes with time (aging).

(f) Susceptibility of materials to environmental changes such as:

  (i) Increase in moisture content resulting in lower mechanical properties.

  (ii) Increase or decrease in moisture content in expansive clays (such as montmorillonite) resulting in volume changes.

2.2 STRENGTH TESTS — UNSTABILIZED MATERIALS

2.2.1 CALIFORNIA BEARING RATIO (CBR)

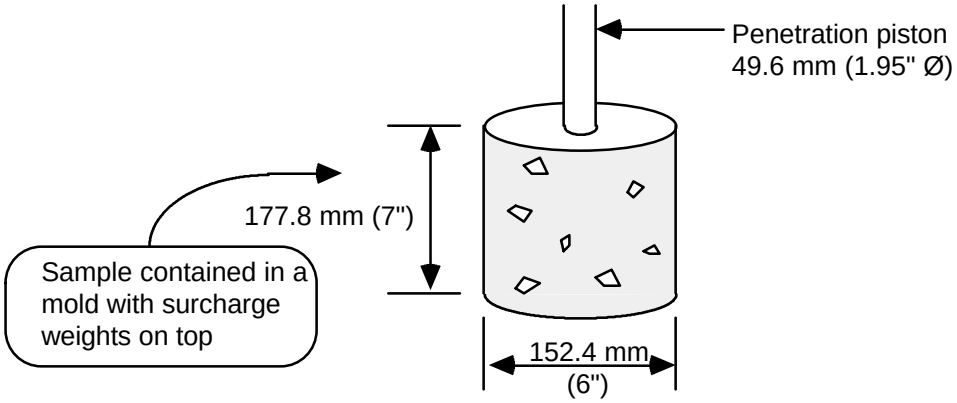
(a) Developed by the California Division of Highways around 1930

(b) Subsequently adapted by numerous states, counties and U.S. federal agencies. Adopted by the U.S. Army Corps of Engineers during early 1940's (WW II use).

(c) This test is a comparative measure of the shearing resistance of a material and is used with empirically derived curves to design flexible pavement structures.

(d) This test can be used for base, subbase, and subgrade materials.

(e) Sketch of sample



(f) Apply load to piston at a rate of 1.3 mm (0.05") per minute. Record total load readings at penetrations ranging from 0.64 mm (0.025 in.) up to 7.62 mm (0.300 in.)

$$CBR(\%) = (100)\,(x/y)$$

where x = material resistance or the unit load on the piston (pressure)—for 2.54 mm (0.1") or 5.08 mm (0.2") of penetration

y = standard unit load (pressure) for well graded crushed stone

= for 2.54 mm (0.1") penetration = 6.9 MPa (1000 psi)

= for 5.08 mm (0.2") penetration = 10.3 MPa (1500 psi)

Thus CBR compares the material being tested with the bearing of a well-graded crushed stone. Thus, a high quality crushed stone base material should have a CBR ≅ 100%.

(g) Typical CBR Ranges

	Soil Type	CBR Range
Coarse-gained soils	GW GP GM GC SW SP SM SC	40 - 80 30 - 60 20 - 60 20 - 40 20 - 40 10 - 40 10 - 40 5 - 20
Fine-grained soils	ML CL LL < 50% OL MH CH LL > 50% OH	15 or less 15 or less 5 or less 10 or less 15 or less 5 or less

(h) Standard Test Methods

- AASHTO T193: The California Bearing Ratio
- ASTM D1883: Bearing Ratio of Laboratory Compacted Soils

2.2.2 Resistance Value (R-value)

(a) A method for evaluating treated and untreated materials for bases, subbases, and subgrades for pavement thickness design.

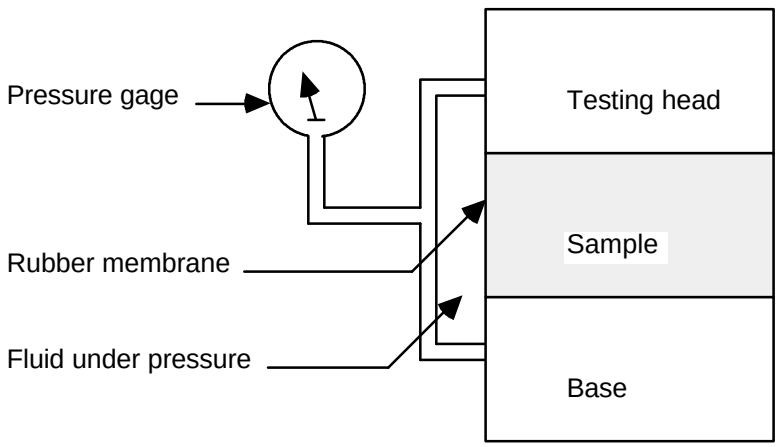
(b) Developed F.N. Hveem and R.M. Carmany of the California Division of Highways — first reported late 1940's. Test method has been used mostly by SHAs on the west coast of the U.S.

(c) Apparatus used is called the stabilometer. The test procedure is basically one where the material's resistance to deformation is expressed as a function of the ratio of the transmitted lateral pressure to that of the applied vertical pressure (which is 160 psi). The sample is 101.6 mm (4 in.) and about 63.5 mm (2.5 in.) high.

(d) In 1950-1951, WSDOT adopted the use of the "R-value" to characterize unstabilized pavement materials. The stabilometer test is essentially a modified triaxial compression test. Materials tested in the stabilometer are assigned an R-value (R stands for resistance). Water would have an R = 0 and steel would

have an R = 100. Naturally, all pavement materials will fall within this range. The R-value was developed at a time (1940s) when rutting (or shoving) in the wheel tracks was the primary concern. At that time, surface courses were not as thick as they are today (hence fatigue has become a more serious design criterion).

The test procedure to determine R-value requires that the laboratory prepared samples are fabricated to a moisture and density condition which are representative of the worst possible condition in a compacted subgrade. The R-value is calculated from the ratio of the applied vertical pressure to the developed lateral pressure and is essentially a measure of the material's resistance to plastic flow.



$$R = 100 - \frac{100}{(2.5/D)[(Pv/Ph) - 1] + 1}$$

where R = resistance value

Pv = applied vertical pressure (160 psi)

Ph = transmitted horizontal pressure at Pv = 160 psi

D = displacement of stabilometer fluid necessary to increase horizontal pressure from 5 to 100 psi.

(e) Typical R-values

- Well-graded (dense gradation) crushed stone base course : 80+
- MH silts : 15-30
- Most Washington state subgrade soils less than 60

(f) Standard test methods (not necessarily identical)

- AASHTO T190: Resistance R-Value and Expansion Pressure of Compacted Soils

• ASTM D2844:	Resistance R-Value and Expansion Pressure of Compacted Soils
• WSDOT Test Method No. 611:	Determination of the Resistance of "R" Value of Untreated Bases, Subbases and Basement Soils by the Stabilometer.

2.3 ELASTIC MODULUS

2.3.1 DEFINITIONS

2.3.1.1 Elastic Modulus

Elastic modulus is sometimes called Young's modulus since Thomas Young published the concept of elastic modulus in 1807 (not exactly a new idea). Essentially, elastic modulus can be determined for any solid material and represents a constant ratio of stress and strain.

E = stress/strain

Thus, the "flexibility" of any object (be it pavement or airplane or bridge or whatever) depends on its elastic modulus and geometrical shape; however, it is important to note that strength (stress needed to break something) is not the same thing as stiffness (as measured by elastic modulus).

A material is elastic if it is able to return to its original shape or size immediately after being stretched or squeezed. Almost all materials are elastic to some degree as long as the load placed on a material does not cause it to deform permanently.

A bit more detail will be provided for this material property (and associated tests) since it is gaining widespread use in pavement design and rehabilitation.

2.3.1.2 Different Types of Moduli

Discussions about moduli can be complicated by the numerous kinds of moduli such as:

- Modulus of elasticity
- Diametral resilient modulus
- Triaxial resilient modulus
- Bulk modulus
- Modulus of resilience
- Modulus of rigidity
- Modulus of rupture
- Modulus of roughness
- Secant modulus
- Tangent modulus
- Young's modulus
- Shear modulus
- Fineness modulus
- Etc.

We are only interested in the modulus of elasticity and resilient modulus and throughout these notes, we will use the term "modulus" to mean the same.

2.3.1.3 Difference Between Modulus of Elasticity and Resilient Modulus

(a) Introduction

What is the difference between modulus of elasticity and resilient modulus? Resilient modulus is roughly the equivalent of modulus of elasticity, but it is an estimate of the modulus of elasticity. The modulus of elasticity for a material is basically the slope of its stress-strain plot within the elastic range (as shown in Figure 2.6). Figure 2.7 shows a stress versus strain curve for steel. The initial straight-line portion of the curve is the elastic range for the steel. If the material is loaded to any value of stress in this part of the curve, it will return to its original shape. Thus, the modulus of elasticity is the slope of this part of the curve and is equal to about 207,000 MPa for steel. On the other hand, resilient modulus is based on stress and strain measurements from rapidly applied loads — more like those that pavement materials experience from wheel loads.

(b) Modulus of Elasticity — Not a Measure of Strength

It is important to remember that a measure of a material's modulus of elasticity or resilient modulus is not a measure of strength. Strength is the stress needed to break or rupture a material (as illustrated in Figure 2.6), whereas elasticity means that the material returns to its original shape and size.

2.3.1.4 Poisson's Ratio

Another important material property used in elastic analysis of pavement systems is Poisson's ratio. This is defined as the ratio of transverse to longitudinal strains of a loaded specimen. This concept is illustrated in Figure 2.8. In realistic terms, Poisson's ratio can vary from initially 0 to 0.5 (no specimen volume change after loading). Generally, "stiffer" materials will have lower Poisson's ratios than "softer" materials. You might see Poisson's ratios larger than 0.5 reported in the literature; however, this implies that the material was stressed to cracking, experimental error, etc.

2.3.1.5 Nomenclature and Symbols

The nomenclature and symbols from the 1986 AASHTO Guide [2.7] will be used in referring to pavement moduli. For example:

(a) E_{AC} = asphalt concrete elastic modulus
(b) E_{BS} = base course resilient modulus
(c) E_{SB} = subbase course resilient modulus
(d) M_R (or E_{SG}) = roadbed soil (subgrade) resilient modulus

The only exception is that M_R and E_{SG} will be used interchangeably.

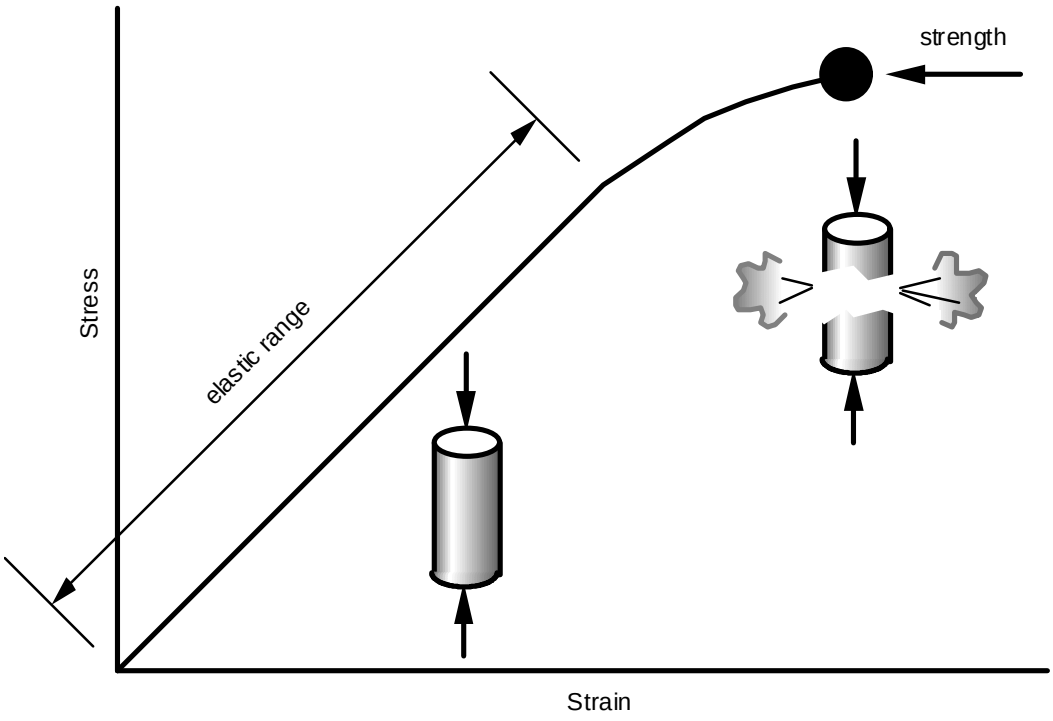


Figure 2.6. Sketch of Stress vs. Strain of a Material in Compression

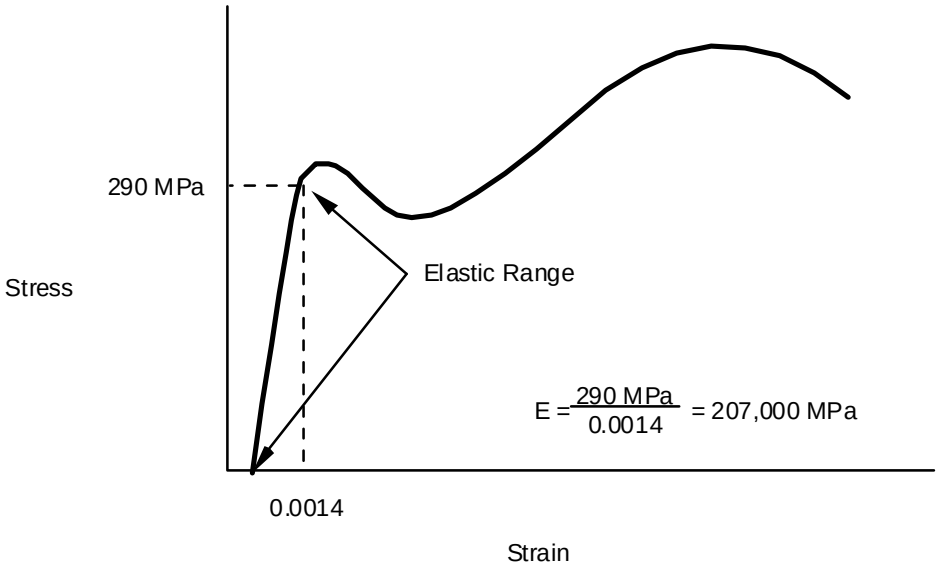
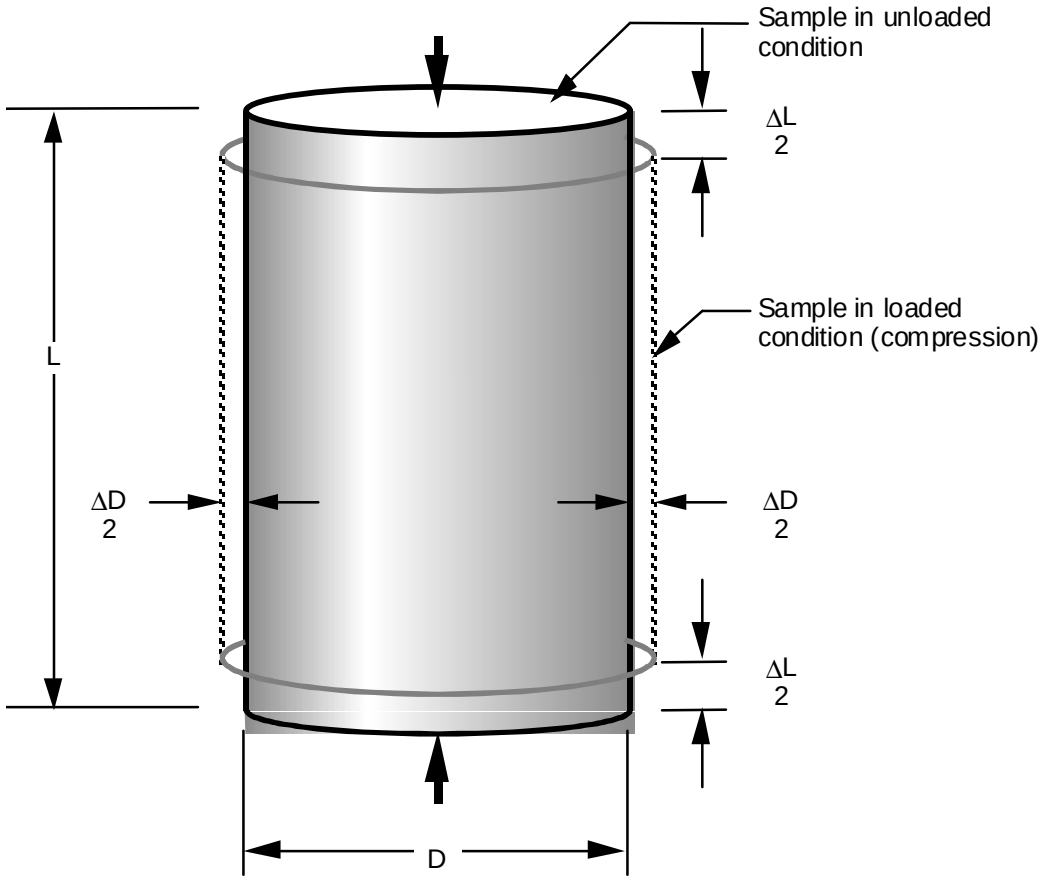


Figure 2.7. Stress-Strain Diagram for Steel



$$\mu = -\frac{\varepsilon_D}{\varepsilon_L}$$

Where

μ = Poisson's ratio

$\varepsilon_D = \frac{\Delta D}{D}$ = strain along the diametrical (horizontal) axis

$\varepsilon_L = \frac{\Delta L}{L}$ = strain along the longitudinal (vertical) axis

Figure 2.8. Illustration of Poisson's Ratio

2.3.1.6 Stress Sensitivity of Moduli

Changes in stress can have a large impact on resilient modulus. "Typical" relationships are shown in Figures 2.9 and 2.10.

2.3.1.7 Temperature Effects

Figures 2.11 and 2.12 illustrate how the stiffness of asphalt concrete is sensitive to temperature. The modulus changes by a factor of about 100 for a 56 °C (100 °F) temperature change for "typical" dense AC mixtures.

2.3.1.8 Typical Values

(a) Typical values of modulus of elasticity for various materials include

	Material	E	
		(psi)	(MPa)
	• Rubber	1,000	7
	• Wood	1-2,000,000	7-14,000
	• Portland Cement Concrete	3-6,000,000	20-40,000
	• Aluminum	10,000,000	70,000
	• Steel	30,000,000	200,000
	• Diamond	170,000,000	1,200,000

(b) Flexible pavement materials

	Material	E	
		(psi)	(MPa)
	• Asphalt Concrete (32 °F (0 °C))	2,000,000	14,000
	• Asphalt Concrete (70 °F (21 °C))	500,000	3,500
	• Asphalt Concrete (120 °F (49 °C))	20,000	150
	• Crushed Stone	20,000-40,000	150-300
	• Silty Soils	5,000-20,000	35-150
	• Clayey Soils	5,000-15,000	35-100

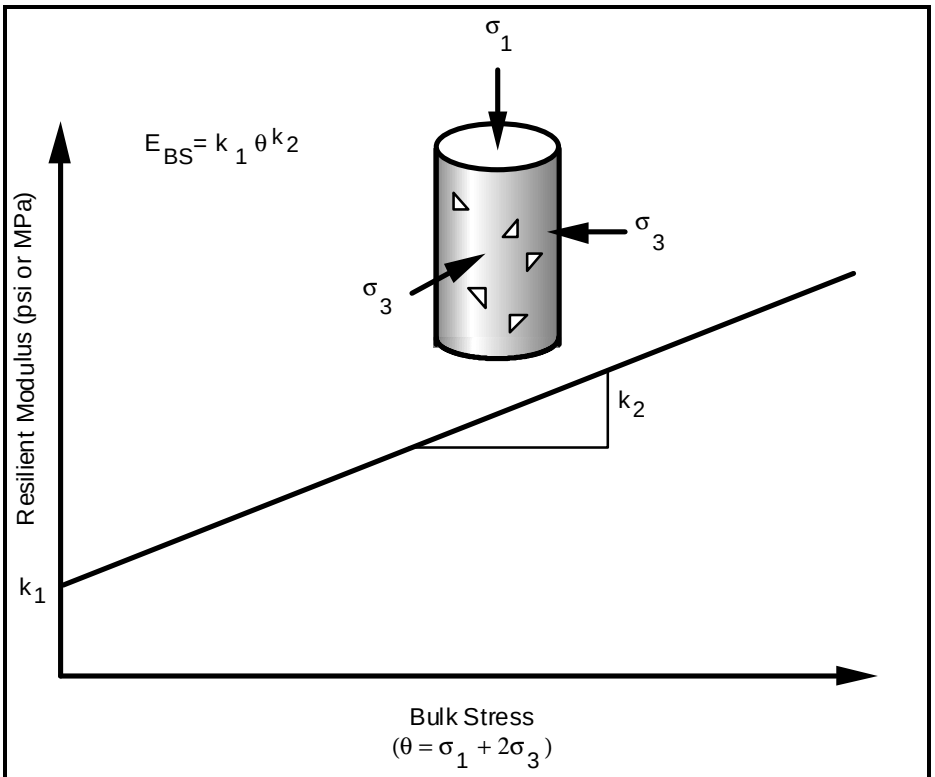


Figure 2.9. Resilient Modulus vs. Bulk Stress for Unstabilized Coarse Grained Materials

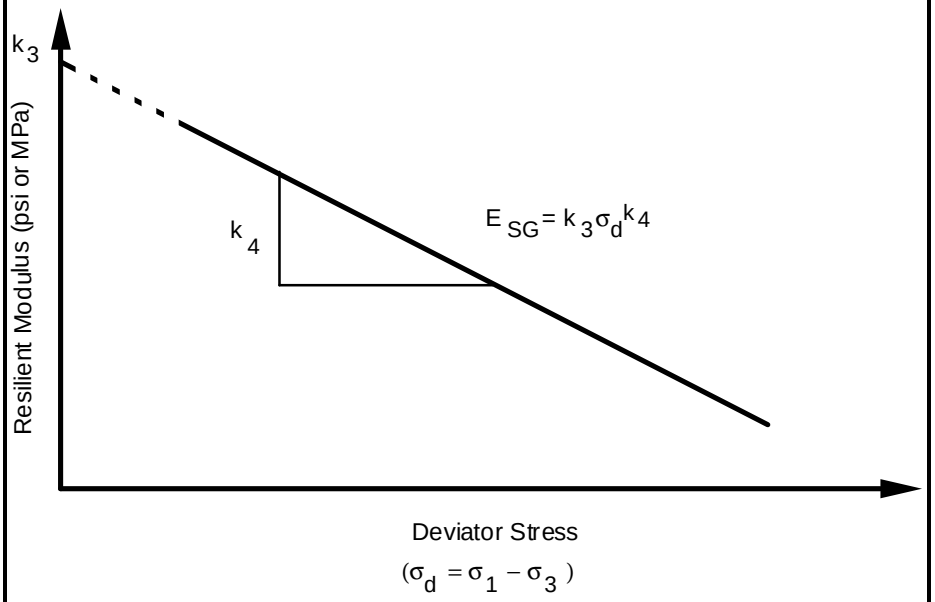


Figure 2.10. Resilient Modulus vs. Deviator Stress for Unstabilized Fine Grained Materials

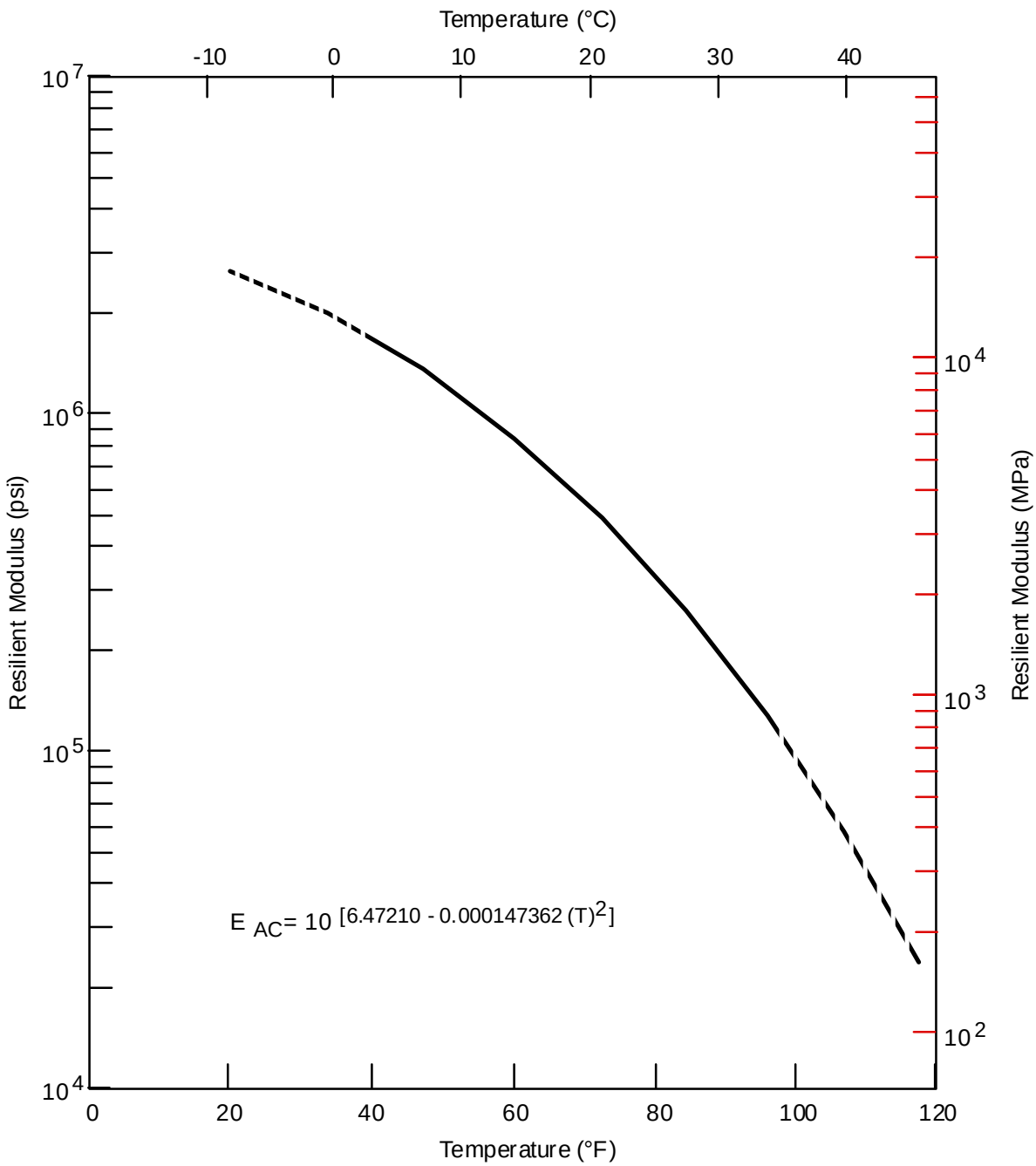


Figure 2.11. General Stiffness-Temperature Relationship for Class B (Dense Graded) Asphalt Concrete in Washington State [2.13]

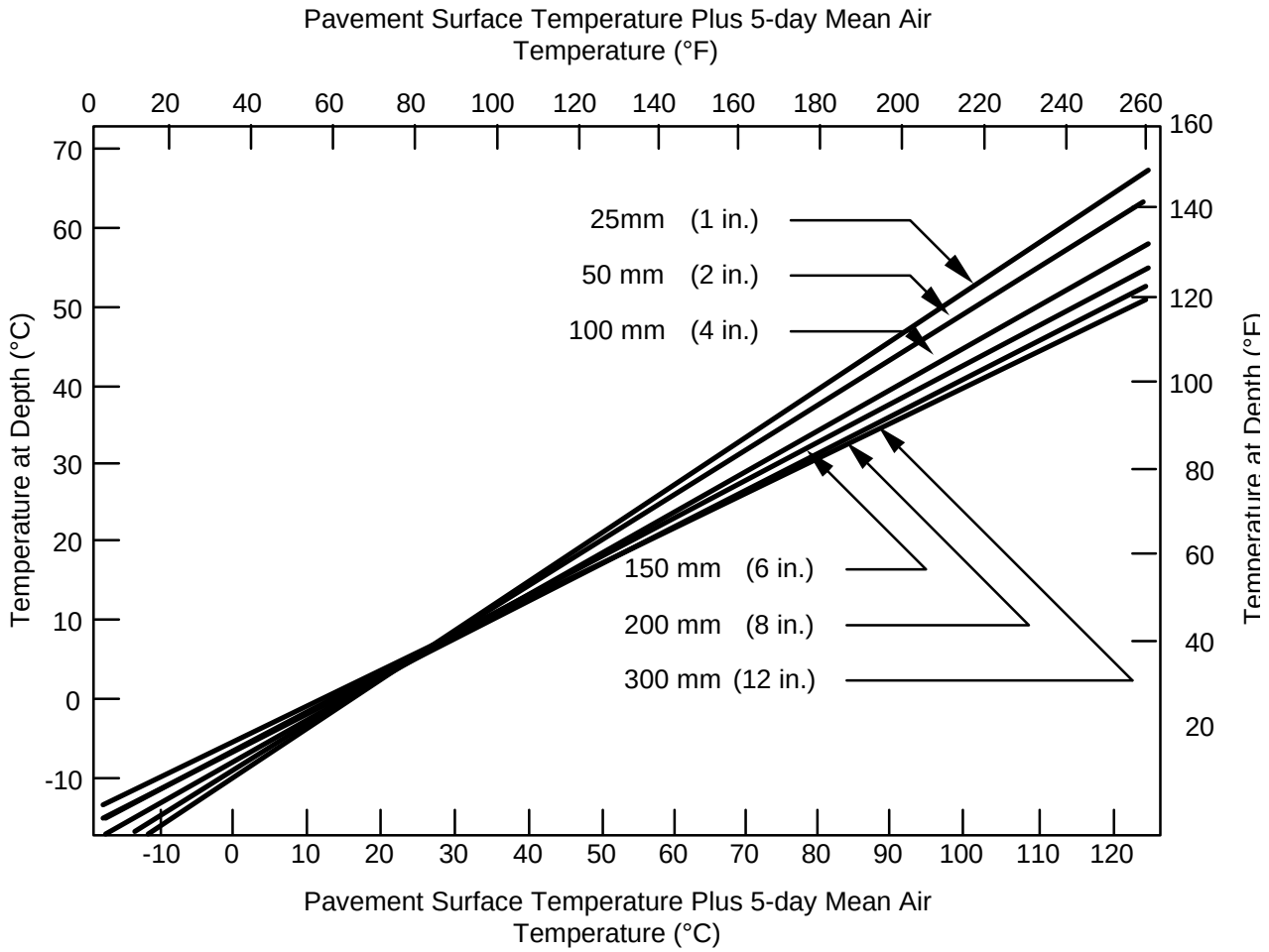


Figure 2.12. Estimation of Pavement Temperature [2.14]

2.3.1.9 Poisson's Ratio

Typical values of Poisson's ratio (μ) include:

Material	Poisson's Ratio
• Steel	0.25 - 0.30
• Aluminum	0.33
• PCC	0.15 - 0.20*
• Flexible Pavement	
• Asphalt Concrete	0.35 (±)
• Crushed Stone	0.40 (±)
• Soils (fine-grained)	0.45 (±)

*Dynamic determination of μ could approach 0.25 for PCC [Neville (2.16)]

2.3.2 HOW ARE ELASTIC MODULI DETERMINED?

2.3.2.1 Introduction

There are two fundamental approaches to estimating elastic moduli — laboratory tests and field deflection data/backcalculation. Of the laboratory tests, two deserve brief mention — diametral resilient modulus and triaxial.

2.3.2.2 Diametral Resilient Modulus

Diametral resilient modulus is the stiffness of a material subjected to a repeated, dynamic pulse-type loading. Diametral deformation is measured along the horizontal diameter (in fact, the term "diametral" simply means "diameter" — or measured across a diameter).

One standard test method for this test is ASTM D4123 Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (recommended for use in the 1993 version of the *AASHTO Guide for Design of Pavement Structures*). It generally takes about 10 minutes to test one sample. A compressive load (to produce tensile stress) is applied to an AC core or laboratory compacted sample, typically 100 mm (4 in.) in diameter and 63.5 mm (2.5 in.) thick or 150 mm (6 in.) in diameter and 75 mm (3 in.) thick. The AC sample is loaded vertically (Figure 2.13) which produces a relatively uniform tensile stress across the vertical diameter (Figure 2.14). The horizontal deformation is measured with LVDTs across the diameter of the sample as shown in Figure 2.15. The formula below can be used to calculate the resilient modulus:

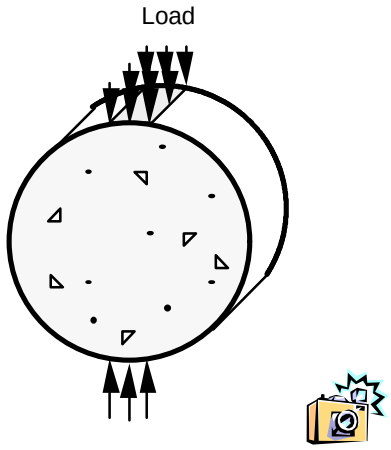


Figure 2.13. Vertical Loading of an AC Core or Laboratory Prepared Specimen for Determining Diametral Resilient Modulus

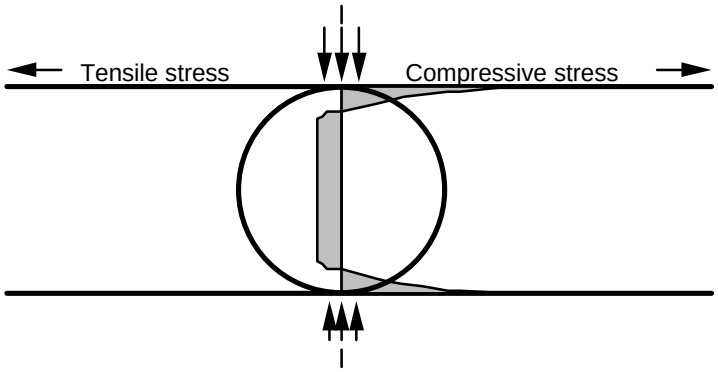


Figure 2.14. Vertical Loading Produces a Relatively Uniform Tensile Stress Across the Vertical Diameter

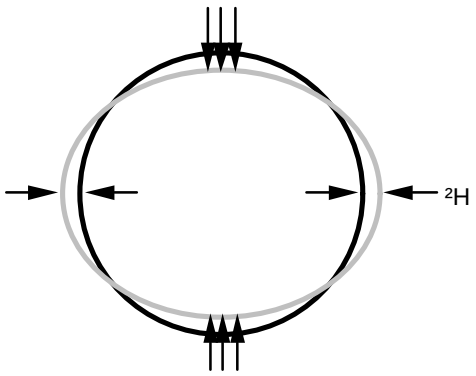


Figure 2.15. Measurement of Horizontal Deformation in the Diametral Resilient Modulus Test

$$E_{AC} = \frac{P\,(\mu + 0.27)}{(t)(\Delta H)}$$

where E_{AC} = asphalt concrete resilient modulus, psi (MPa),
P = repeated load, lb (N),
μ = Poisson's ratio (usually assumed),
t = thickness of the sample, in. (mm),
ΔH = recoverable horizontal deformation, in. (mm).

To conduct this type of test, the needed test equipment includes (after ASTM D4123):

- Testing machine capable of applying a load pulse over a range of frequencies, load durations, and load levels (typical load duration is 0.1s at 1 Hz with load ranges 4 to 35 N/mm (4 to 200 lb/in.) of specimen thickness (10 to 50 percent of the AC tensile strength).
- Temperature control system capable of controlling temperatures from 5 to 40 °C (41 to 104 °F). Typically, moduli are determined at 5, 25, 40 °C (41, 77, and 104 °F).
- Measurement and recording system. The horizontal measurements are made with linear variable differential transducers (LVDTs) capable of measuring deformations of 0.00025 mm (0.00001 in.). Loads are measured with an electronic load cell.

Due to possible creep effects at the higher temperatures, caution is warranted for such resilient moduli results.

A 1988 survey of SHAs [2.24] revealed that 18 SHAs either were using or planned to adopt ASTM D 4123. Another 8 SHAs used a similar but modified procedure.

2.3.2.3 Triaxial Resilient Modulus

(a) Introduction

One triaxial standard text method is AASHTO T274 (currently under revision but recommended for use in the 1993 version of the *AASHTO Guide for Design of Pavement Structures*). The specimen is a cylindrical sample normally 100 mm (4 in.) in diameter by 200 mm (8 in.) high (Figure 2.16). The sample is generally compacted in the laboratory; however, undisturbed samples are best if available (which is rare). The specimen is enclosed vertically by a thin "rubber" membrane and on both ends by rigid surfaces (platens) as sketched in Figure 2.17. The sample is placed in a pressure chamber and a confining pressure is applied (σ_3) as sketched in Figure 2.18. The sample then undergoes repeated pulses of an axial

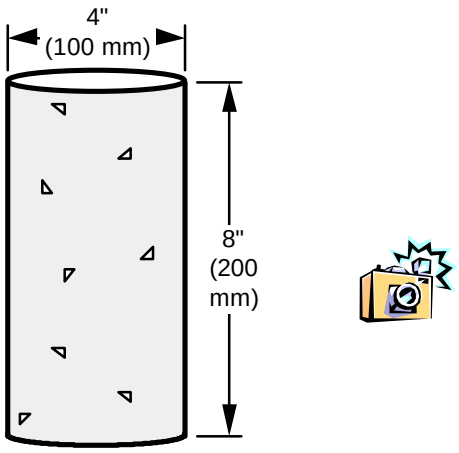


Figure 2.16. Basic Triaxial Specimen Configuration

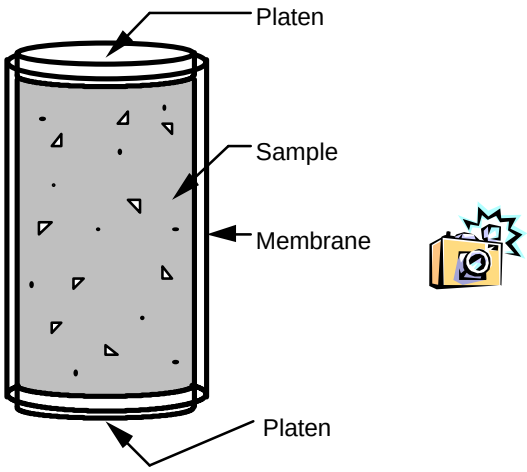


Figure 2.17. Enclosure of Triaxial Specimen

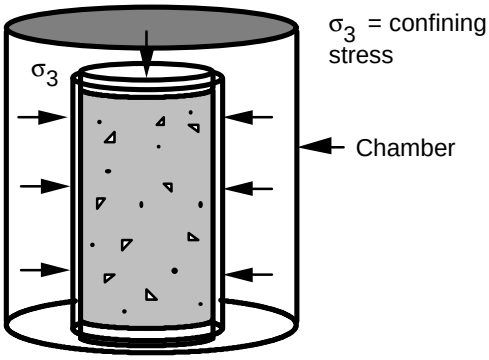


Figure 2.18. Triaxial Specimen in Pressure Chamber

stress. This deviator stress is designated σ_d and it equals the total vertical stress applied by the testing apparatus (σ_1) minus the confining stress (σ_3). In other words, the deviator stress is the repeated stress applied to the sample. These stresses are further illustrated in Figure 2.19. The resulting strains are calculated over a gauge length, which is designated by "L" (refer to Figure 2.20). As illustrated in Figure 1.15, the initial condition of the sample is unloaded (no induced stress). When the deviator stress is applied, the sample deforms, changing in length as shown in Figure 2.21. This change in sample length is directly proportional to the stiffness.

To calculate the resilient modulus, one uses the following equations:

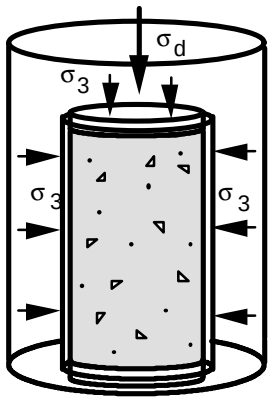
$$M_R \text{ (or } E_R) = \frac{\sigma_d}{\varepsilon_r}$$

M_R (or E_R) = resilient modulus,

where σ_d = deviator stress,
= P/A
P = repeated load,
A = cross sectional area of the sample,
ε_r = recoverable axial strain,
= $\Delta L/L$
L = gauge length over which the sample deformation is measured,
ΔL = change in sample length due to applied load.

An NCHRP project (1-28, "Laboratory Determination of Resilient Modulus for Flexible Pavement Design") underway at the Georgia Institute of Technology should result in improved test producers which, presumably, will "upgrade" ASTM D 4123 and AASHTO T274 or result in all new test methods. (This NCHRP information current as of May 1992.)

In September 1992, AASHTO issued a document entitled "Interim Specifications for Transportation Materials and Methods of Sampling and Testing" which contains AASHTO T294-92-I "Interim Method of Test for Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils — SHRP Protocol 46." This, in effect, replaces AASHTO T274 and essentially adopts the SHRP test procedure. One change from T274 to T294 is that the axial deformation is measured by an LVDT's external to testing chamber (the LVDTs are attached to the test specimen in T274). The AASHTO T294 procedure is so new that results are lacking at this time (July 1994).



σ_1 = total axial stress

σ_d = deviator stress

σ_3 = confining stress

$$\sigma_1 = \sigma_d + \sigma_3$$

or

$$\sigma_d = \sigma_1 - \sigma_3$$

Figure 2.19 Stresses Acting on Triaxial Specimen

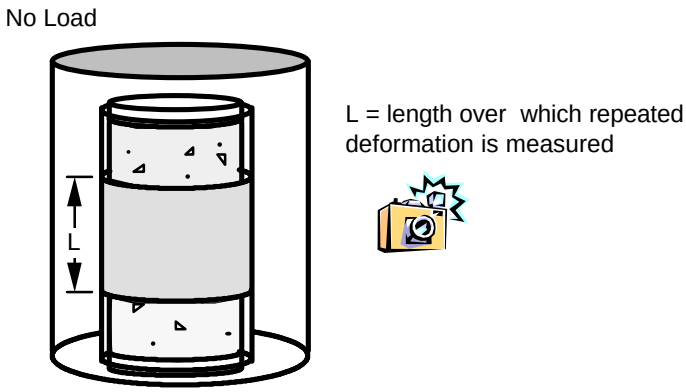


Figure 2.20. Gage Length for Measurement of Strain on Triaxial Specimen

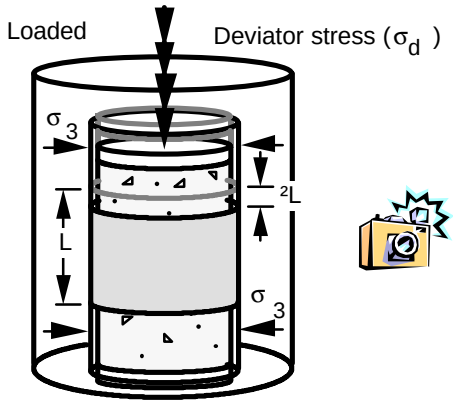


Figure 2.21. Deformation of Triaxial Specimen Under Load

(b) SHA Survey

A 1988 survey of SHAs [2.24] revealed that 24 SHAs had or intended to adopt AASHTO T274. The agencies which were running the test used a variety of test equipment. None of the agencies reported running the test before 1985 (thus, no one specific SHA has accumulated extensive experience with the test and the results). WSDOT started running a modified version of AASHTO T274 in 1987.

(c) Typical WSDOT Base and Subgrade Triaxial Resilient Modulus

Based on resilient modulus triaxial tests conducted at the WSDOT Materials Laboratory (modified version of AASHTO T274), a "default" or composite modulus of WSDOT's crushed surfacing material was developed. These triaxial tests were run in July 1988, April 1989 and May 1989 on the same, disturbed samples from 14 sites:

• SR 410	MP 9.6	• SR 20	MP 77.5
• SR 411	MP 18.0	• SR 20	MP 108.2
• SR 5	MP 35.8	• SR 20	MP 140.8
• SR 500	MP 3.2	• SR 195	MP 7.2
• SR 14	MP 18.2	• SR 195	MP 20.0
• SR 11	MP 20.8	• SR 195	MP 63.8
• SR 20	MP 53.5	• SR 90	MP 208.8

The results of the triaxial tests are summarized in Table 2.4 for both base (14 samples) and subgrade (13 samples) materials. Composite regression equations for the samples along with the average density (wet and dry) and resilient moduli are shown in the table. In most cases, the three sets of test results were used to calculate the values shown. If the material was classified as a base, both the θ and σ_d independent variables were evaluated at 172 kPa (25 psi). If classified as a subgrade, both θ and σ_d were evaluated at 69 kPa (10 psi). (Since the σ_1 principal stress dominates either bulk stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$) or deviator stress ($\sigma_d = \sigma_1 - \sigma_3$), this turns out to be a reasonable solution.)

Based on the results contained in Table 2.4, the following observations are made.

- The average modulus for the bases (14 sites) = 194 MPa (28,100 psi) with a standard deviation = 29.0 MPa (4,200 psi). If the two lowest values are removed, the average is 203 MPa (29,400 psi)with a standard deviation of 19.3 MPa (2,800 psi) (12 samples).

- The average k_1 and k_2 values for the bases were calculated for individual equations with $R^2 \geq 80\%$. This results in an average $k_1 = 8{,}900$ and average $k_2 = 0.36$. Rounding results in the following general equation:

$$E_{BS} = 8{,}900\ (\theta)^{0.36} \text{ (in psi)}$$

Table 2.4. Summary of Laboratory Resilient Moduli—Recompacted Base and Subgrade Samples

Material Type	Location		Approximate Regression Equation	Average Density (lb/ft^3)		Average Resilient Modulus
	State Route	Milepost		Wet	Dry	@ $\theta \cong \sigma_d$ = 25 psi for base @ $\theta \cong \sigma_d$ = 10 psi for subgrade
Base Materials	410	9.6	$9000\ \theta^{0.38}$	129.8	124.2	30,300
	411	18.0	$5400\ \theta^{0.40}$	117.5	112.4	19,900
	5	35.8	$6200\ \theta^{0.38}$	136.5	126.0	20,700
	500	3.2	$13000\ \theta^{0.26}$	129.8	120.8	30,300
	14	18.2	$10400\ \theta^{0.31}$	135.0	125.7	28,000
	11	20.8	$8600\ \theta^{0.35}$	128.9	123.4	26,700
	20	53.5	$8300\ \theta^{0.42}$	132.2	126.5	32,200
	20	77.5	$7800\ \theta^{0.39}$	129.5	124.1	27,200
	20	108.2	$9500\ \theta^{0.38}$	134.8	128.9	32,200
	20	140.8	$8700\ \theta^{0.36}$	140.3	133.0	27,800
	195	7.2	$8700\ \theta^{0.36}$	138.4	131.5	27,700
	195	20.0	$7400\ \theta^{0.39}$	126.5	118.8	25,400
	195	63.8	$11300\ \theta^{0.35}$	136.0	128.2	34,900
	90	208.8	$10000\ \theta^{0.34}$	129.3	122.6	30,000
Subgrade and Borrow Materials	410	9.6	$3600\ \sigma_d^{0.29}$	124.6	106.1	6,900
	5	35.8	$11600\ \theta^{0.28}$	132.0	119.9	21,600
	500	3.2	$8800\ \sigma_d^{0.28}$	142.3	130.6	17,100
	14	18.2	$16200\ \theta^{0.23}$	133.5	123.2	27,400
	11	20.8	$7300\ \theta^{0.43}$	138.1	129.8	19,300
	20	53.5	$5700\ \theta^{0.55}$	126.9	121.4	20,000
	20	77.5	$8800\ \theta^{0.41}$	131.5	125.9	22,400
	20	108.2	$11100\ \theta^{0.36}$	139.3	132.9	25,700
	20	140.8	$9200\ \theta^{0.35}$	132.9	127.9	20,600
	195	7.2	$18000\ \sigma_d^{-0.28}$	135.7	118.0	9,600
	195	20.0	$21600\ \sigma_d^{-0.23}$	131.6	112.9	12,300
	195	63.8	$8500\ \sigma_d^{-0.065}$	141.5	126.0	6,900
	90	208.8	$24900\ \sigma_d^{0.18}$	144.1	134.2	37,800

MPa = (psi) (0.00689476)

For a "standard" θ = 25 psi (172 kPa), then E_{BS} = 28,400 psi (196 MPa).

If only the 12 sites are used (remove the lowest two test results), then k_1 = 9400 and k_2 = 0.36 or $E_{BS} = 9400\ (\theta)^{0.36}$. For a "standard" θ = 25 psi, (172 kPa) then $E_{BS} \cong$ 30,000 psi (207 MPa)

- The average modulus for the subgrades (13 sites) = 19,300 psi (133 MPa) with a standard deviation = 8,600 psi (59 MPa). These values are based on all equations without regard to R^2. In some cases, the "subgrade" sample was likely borrow material (e.g., SR 5 MP 35.8, etc.). If only the subgrade soils which are fine-grained are used (say $\gamma_d \leq$ 120 pcf (1922 kg/m^3)), then the average subgrade modulus (three sites) = 9,600 psi (66 MPa) (standard deviation = 4,000 psi (28 MPa)). (This estimate excluded SR 5, MP 35.8.)

2.3.2.4 Elastic Modulus — CBR and R–Value Correlations

A widely used empirical relationship developed by Heukelom and Klomp [2.61] and used in the 1993 AASHTO Guide is:

$$E_{SG} = (1500)\ (CBR)$$

where E_{SG} = subgrade resilient modulus (psi), and

CBR = California Bearing Ratio.

This equation is restricted to fine grained materials with soaked CBR values of 10 or less. Like all such correlations, it should be used only, if appropriate, for project specific conditions.

Figure 2.21b contains a WSDOT developed relationship between the R-value and resilient modulus. This graph was developed using WSDOT samples which ranged from silty materials (A-7) to coarse aggregate (A-1). The samples were tested according to Washington Test Method 611 (Determination of the Resistance (R-Value of Untreated Bases, Subbases, and Basement Soils by the Stabilometer) and AASHTO T274 (Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils). Note that WSDOT Test Method 611 “design R-Values” are determined at an exudation pressure of 400 psi. AASHTO T190 (Resistance R-Value and Expansion Pressure of Compacted Soils) allows the use of a 300 psi exudation pressure. Thus, R-Values may differ due to the exudation pressure.

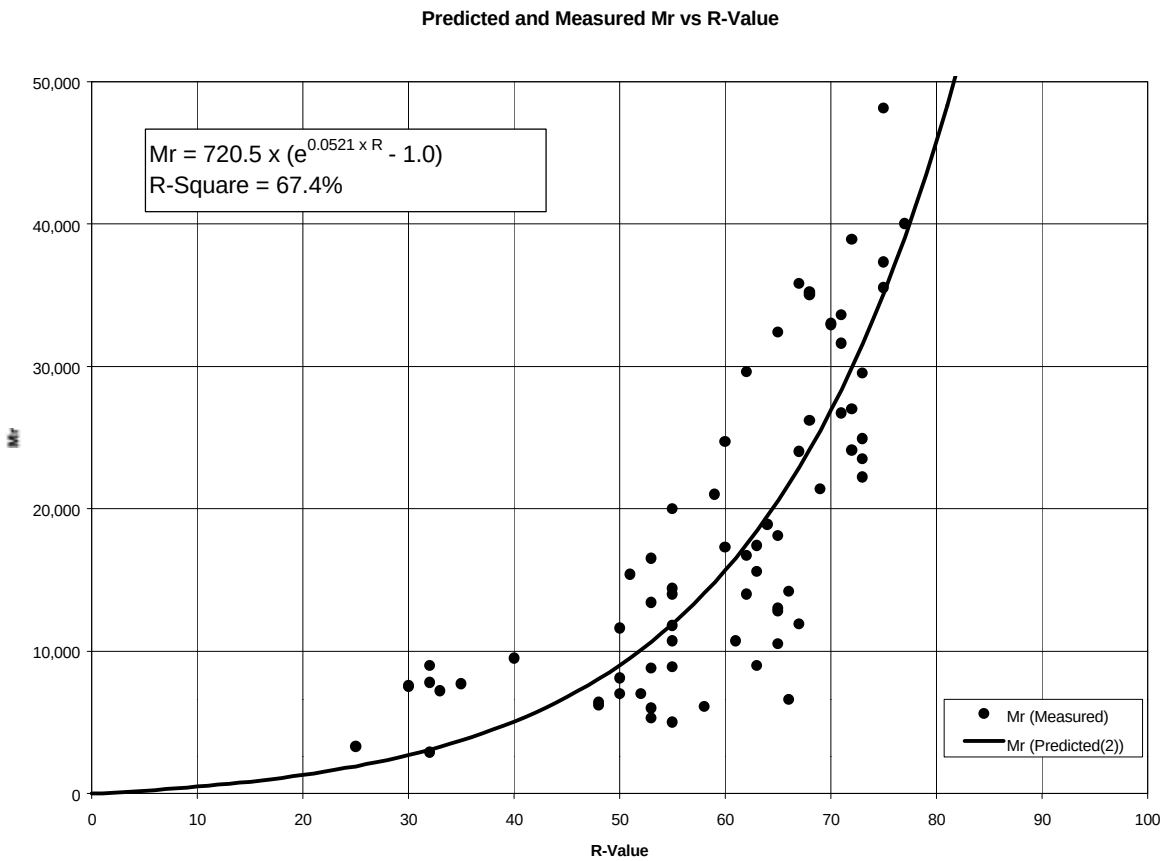


Figure 2.21b WSDOT Developed Correlation Between R-Value and Resilient Modulus

2.3.2.5 Portland Cement Concrete

ACI 318-89 (Building Code Requirements for Reinforced Concrete) provides correlations which can be used to estimate the elastic modulus of PCC. For PCC with unit weights between 1440 to 2480 kg/m^3 (90 to 155 lb/ft^3) and compressive strengths up to 41.4 MPa (6,000 psi:)

$$E_c = (w_c)^{1.5}\,(33)(f'_c)^{0.5}$$

where E_c = static elastic modulus, psi

w_c = unit weight of PCC, lb/ft^3, and

f'_c = specified compressive strength for 6" x 12" cylinders (normally 28-day cure)

For example, if $w_c = 150 \text{ lb/ft}^3$

$f'_c = 4{,}000 \text{ psi}$

then $E_c = (150)^{1.5}\,(33)(4000)^{0.5} \cong 3{,}800{,}000 \text{ psi}$

ACI 318 also includes another formula to estimate the elastic modulus of normal weight PCC:

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

Again, if we use $f'_c = 4{,}000$ psi, then

$$E_c = 57{,}000\,(4{,}000)^{0.5} \simeq 3{,}600{,}000 \text{ psi}$$

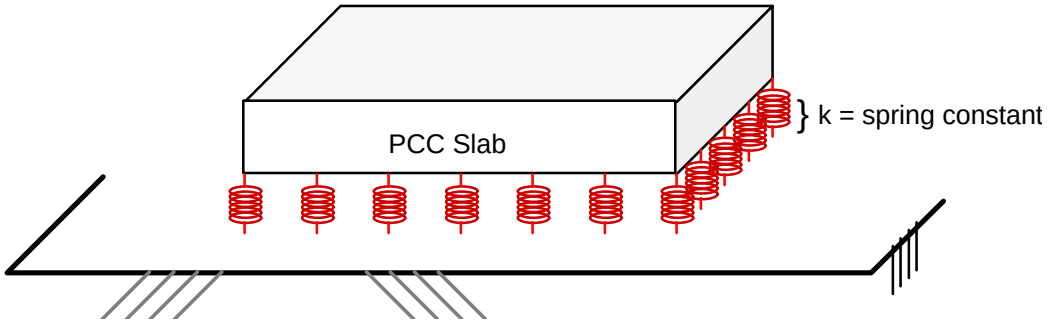
In the laboratory, test methods such as ASTM C469 (Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression) can be used. The test is conducted on standard 152 by 305 mm (6 in. by 12 in.) cylinders. These moduli values are usually less than those derived from dynamic tests.

A dynamic modulus of elasticity can be obtained by the method described in ASTM C215 (Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens). As for ASTM C649, standard PCC cylinders can be used as test specimens.

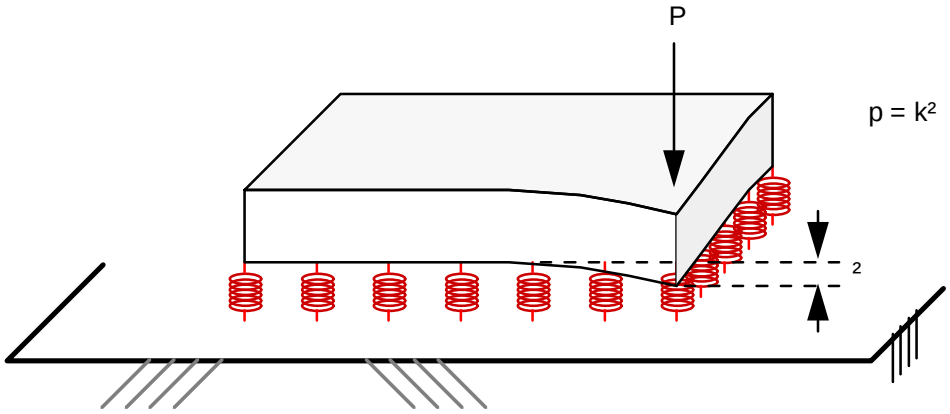
2.4 MODULUS OF SUBGRADE REACTION

The modulus of subgrade reaction (k) is used to estimate the support of the layers below the PCC slab (the subbase and subgrade) and is used as a primary input for rigid pavement design. The k-value can be determined by field tests (such as AASHO T222: Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements) or by correlation with other tests. There is no direct laboratory procedure for determining k-value.

The work done by Westergaard during the 1920s made use of the k-value to model the reaction of the layers beneath a PCC slab to slab deflection (refer to SECTION 4.0). The k-value was developed basically as a spring constant (as shown below) to describe the support beneath the slab.



The deflection of the springs is proportional to p, i.e., the reactive pressure to resist a load is proportional to k and the slab deflection, as shown below.

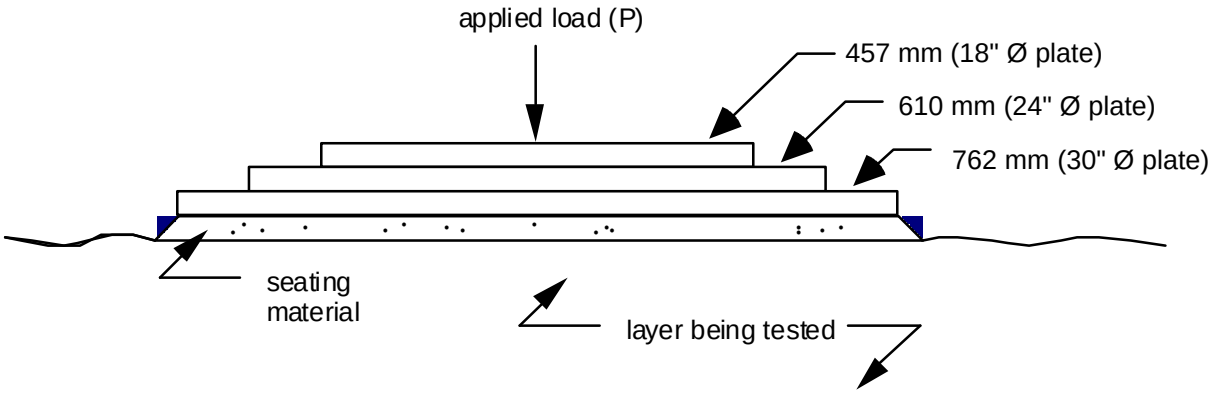


where p = reactive pressure (psi) to support deflected slab,

k = spring constant = modulus of subgrade reaction, and

Δ = slab deflection (in.).

The plate load test (AASHTO T222) is a field test which has been used to determine in situ k values, and is conducted with three steel plates as shown below.



and $k = \frac{p}{\Delta}$

where p = applied pressure = P/Area of 762 mm (30") diameter plate
Δ = measured deflection of the 762 mm (30") diameter plate

Naturally, the test procedure has very specific protocols for the test setup and conduct.

WSDOT has only rarely conducted the plate load test in recent years. The k-value is more likely to be estimated from resilient moduli in accordance with the procedure in the *AASHTO Guide* (see SECTION 6.0 of this Volume).

2.5 STRENGTH TESTS — ASPHALT CONCRETE SURFACE COURSE

2.5.1 STRENGTH TESTS

Strength tests can include such tests as:

(a) Marshall stability

(i) ASTM D1559: Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus

(ii) AASHTO T245: Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus

(b) Hveem stabilometer

(i) ASTM D1560: Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus

(ii) AASHTO T246: Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus

(c) Unconfined compression

(i) ASTM D1074: Compressive Strength of Bituminous Mixtures

(ii) AASHTO T167: Compressive Strength of Bituminous Mixtures

2.6 FATIGUE CHARACTERIZATION OF STABILIZED LAYERS

2.6.1 ASPHALT CONCRETE

(a) No standardized laboratory fatigue test currently exists; however, both rectangular beams (laboratory compacted or field sawed) and 4 inch diameter samples (laboratory compacted or field core) have been used to conduct fatigue testing. The results of such testing generally has to be "shifted" to approximate field fatigue conditions

(b) ASTM D3202: Preparation of Bituminous Mixture Beam Specimens by Means of the California Kneading Compactor

This compaction method has been commonly used to prepare laboratory compacted beams for fatigue testing.

(c) The results of fatigue testing are generally plotted as a log-log relationship as initial tensile strain (ε_t) versus load to failure (N_f). This general relationship is

$$N_f = k_1\left(\frac{1}{\varepsilon_t}\right)^{k_2}$$

where N_f = loads to failure,

ε_t = initial tensile (bending) strain, and

k_1, k_2 = constants obtained from test results

Thus, the loads to failure are inversely proportional to tensile strain

2.6.2 CEMENT TREATED BASE

As for asphalt treated base, no standardized laboratory test currently exists; however, when such tests are conducted the following general relationship is used

$$N_f = c\left(\frac{1}{\sigma}\right)^{n}$$

where N_f = loads to failure,
σ = initial tensile stress, and
c, n = constants obtained from test results.

Thus, the loads to failure are inversely proportional to tensile stress.

2.7 MINERAL AGGREGATES

2.7.1 INTRODUCTION

(a) Generally account for 30 percent of the cost of a pavement structure

(b) By weight: 92 to 96 percent of asphalt concrete to 65 to 85 percent of portland cement concrete

(c) Aggregate (definition) [after Ref. 2.8]: Combination of sand, gravel, crushed stone or other material of mineral composition with a binding medium (water, bituminous, portland cement, lime, etc.) to form materials such as asphalt concrete, portland cement concrete, etc. Aggregates may be natural or manufactured.

2.7.2 PARTICLE SHAPES

(a) Gravel: naturally rounded particles - smooth texture
(b) Crushed stone: artificial crushing of rock - generally with rough texture

2.7.3 AASHTO DEFINITIONS

2.7.3.1 AASHTO M146

The following definitions are from AASTHO M146 (Terms Relating to Subgrade, Soil-Aggregate, and Fill Materials):

(a) Soil-Aggregate (Dense-Graded Aggregate): Natural or prepared mixtures consisting predominately of stone, gravel or sand and containing silt-clay (minus 75 µm) material.

(b) Binder (Soil Binder): Portion of soil passing a 425 µm sieve.

(c) Stone: Crushed or naturally angular particles of rock which will pass a 75 mm sieve and be retained on a 2.00 mm sieve.

(d) Gravel: Rounded particles of rock which will pass a 75 mm sieve and be retained on a 2.00 mm sieve.

(e) Sand: Granular material resulting from the disintegration, grinding, or crushing of rock and which will pass the 2.00 mm sieve and be retained on the 75 µm sieve.

(f) Silt-Clay: Fine soil particles which will pass the 75 µm sieve.

(g) Silt Fraction: Material passing the 75 µm and larger than 0.002 mm.

(h) Clay Fraction: Material smaller than 0.002 mm.

2.7.3.2 AASHTO M147

Additional definitions from AASHTO M147 (Materials for Aggregate and Soil-Aggregate Subbase, Base and Surface Courses):

(a) Coarse Aggregate: Aggregate retained on the 2.00 mm sieve and consisting of hard, durable particles or fragments of stone, gravel or slag. A wear requirement (AASHTO T96) is normally required.

(b) Fine Aggregate: Aggregate passing the 2.00 mm sieve and consisting of natural or crushed sand, and fine material particles passing the 75 µm. The fraction passing the 75 µm sieve shall not be greater than two-thirds of the fraction passing the 425 µm sieve. The portion passing the 425 µm sieve shall have a LL ≤ 25 and a PI ≤ 6. Fine aggregate shall be free from vegetable matter and lumps or balls of clay.

2.7.4 GRADATION

2.7.4.1 Terminology

(a) Dense or well-graded (same terms)

(b) Gap, uniform and open-graded (in general, same terms)

2.7.4.2 Fuller's maximum density curve

(a) Equation

$$P = 100\left(\frac{d}{D}\right)^n$$

where P = % finer than the sieve,

d = aggregate size being considered,

D = maximum aggregate size to be used,

n = parameter which adjusts curve for fineness or coarseness (for maximum particle density n ≈ 0.45 to 0.5)

(b) Illustration

To further illustrate Fuller's maximum density curve refer to Figure 2.22. This figure includes a maximum density curve (n = 0.45) for a maximum aggregate size of 19.0 mm (100% passing). The associated calculations for this curve follow:

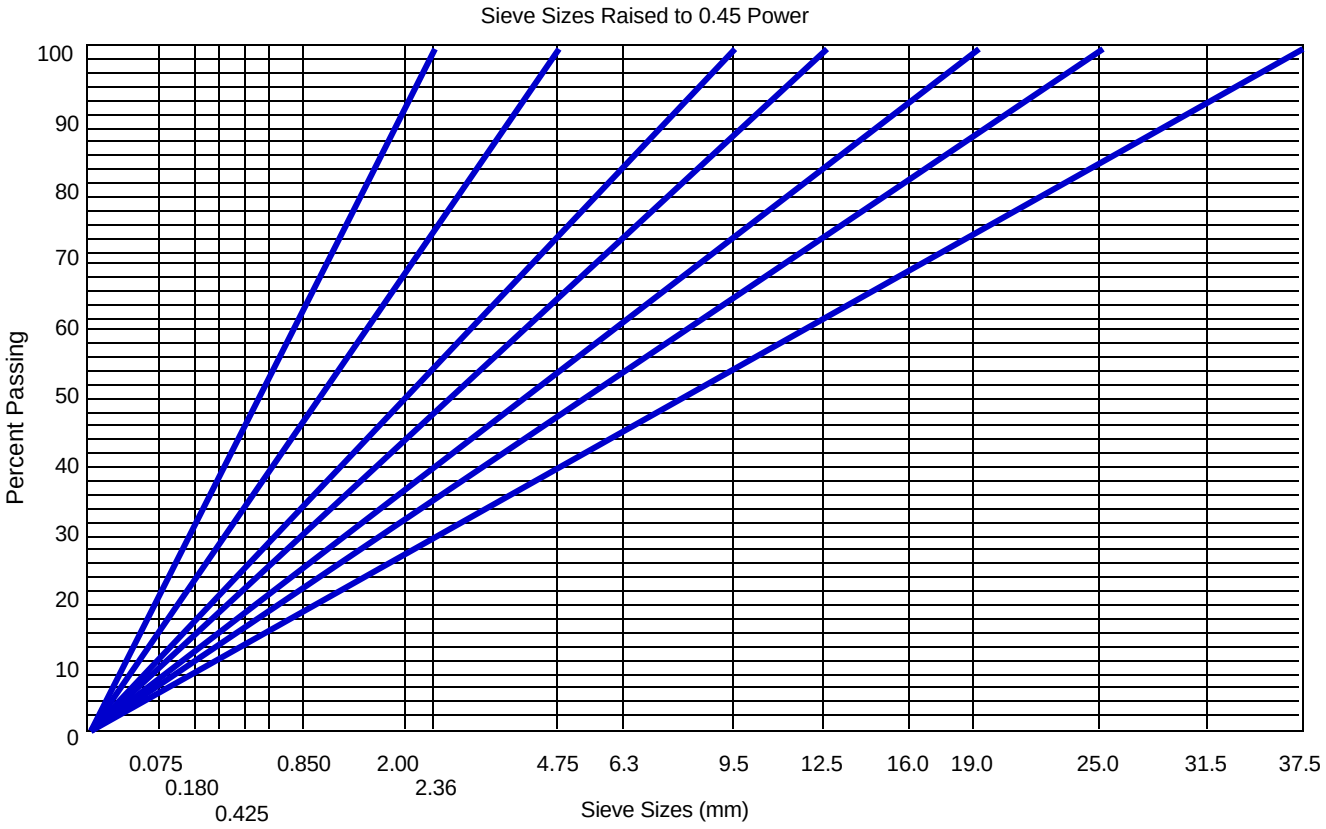


Figure 2.22. Maximum Density Curves for 0.45 Power Gradation Chart [2.11]

Particle Size (mm)	% Passing
19.0	$P = \left(\frac{19.0}{19.0}\right)^{0.45} = 1.000$ (100%)
12.5	$P = \left(\frac{12.5}{19.0}\right)^{0.45} = 0.833$ (83.3%)
9.5	$P = \left(\frac{9.5}{19.0}\right)^{0.45} = 0.732$ (73.2%)
2.00	$P = \left(\frac{2.00}{19.0}\right)^{0.45} = 0.363$ (36.3%)
0.300	$P = \left(\frac{0.300}{19.0}\right)^{0.45} = 0.154$ (15.4%)
0.075	$P = \left(\frac{0.075}{19.0}\right)^{0.45} = 0.082$ (8.2%)

Typically specified WSDOT aggregates are shown in Table 2.5. Are all three of these "dense gradation"?

Typical aggregate gradations are shown in Figure 2.23a (that is well-graded, open-graded, etc.). The associated permeabilities imply that even small amounts passing the 75 µm result in very low permeability. Thus, are crushed surfacing top course and base high permeability materials? What about Shoulder Ballast? To help answer these questions, inspect the gradation bands for CSTC, CSBC, and Shoulder Ballast shown in Figure 2.23b. (Also refer to SECTION 9.0, Paragraph 6, for more information on permeability of WSDOT unstabilized materials.) A few typical gradations for CSTC, CSBC, and Shoulder Ballast are shown in Table 2.6. The gradation bands for the various WSDOT paving mixes are shown in Appendix 2.2.

2.7.5 BASE COURSE AGGREGATE

2.7.5.1 The gradation of the aggregate will affect structural capacity, drainage and frost susceptibility. Thus, gradation control is important.

Table 2.5. WSDOT Gradations for Shoulder Ballast, Crushed Surfacings (Top Course and Base) from WSDOT 1994 Standard Specifications—Section 9-0.3.9

Size (mm)	Percent Passing		
	Shoulder Ballast	Crushed Surfacing	
		Top Course	Base Course
63	100	—	—
31.5	—	—	100
19.0	40 - 80	—	—
16.0	—	100	50 - 80
6.3	5 max	55 - 75	30 - 50
0.425	—	8 - 24	3 - 18
0.150	0 - 2	—	—
0.075	—	10 max	7.5 max
% Fracture	75% min.	75% min.	75% min.

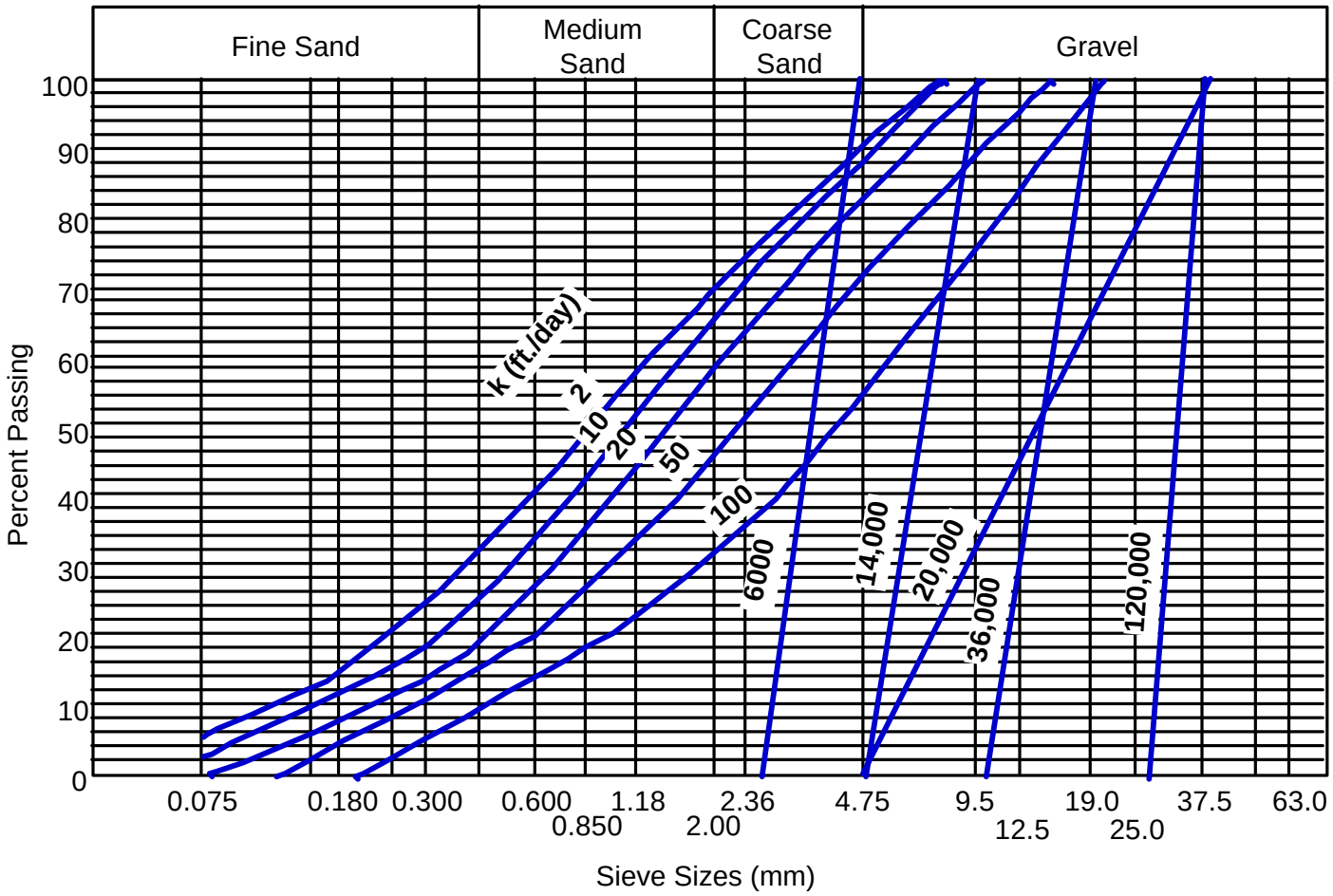


Figure 2.23a. Sketch of Typical Aggregate Gradations and Permeabilities [after 2.12]

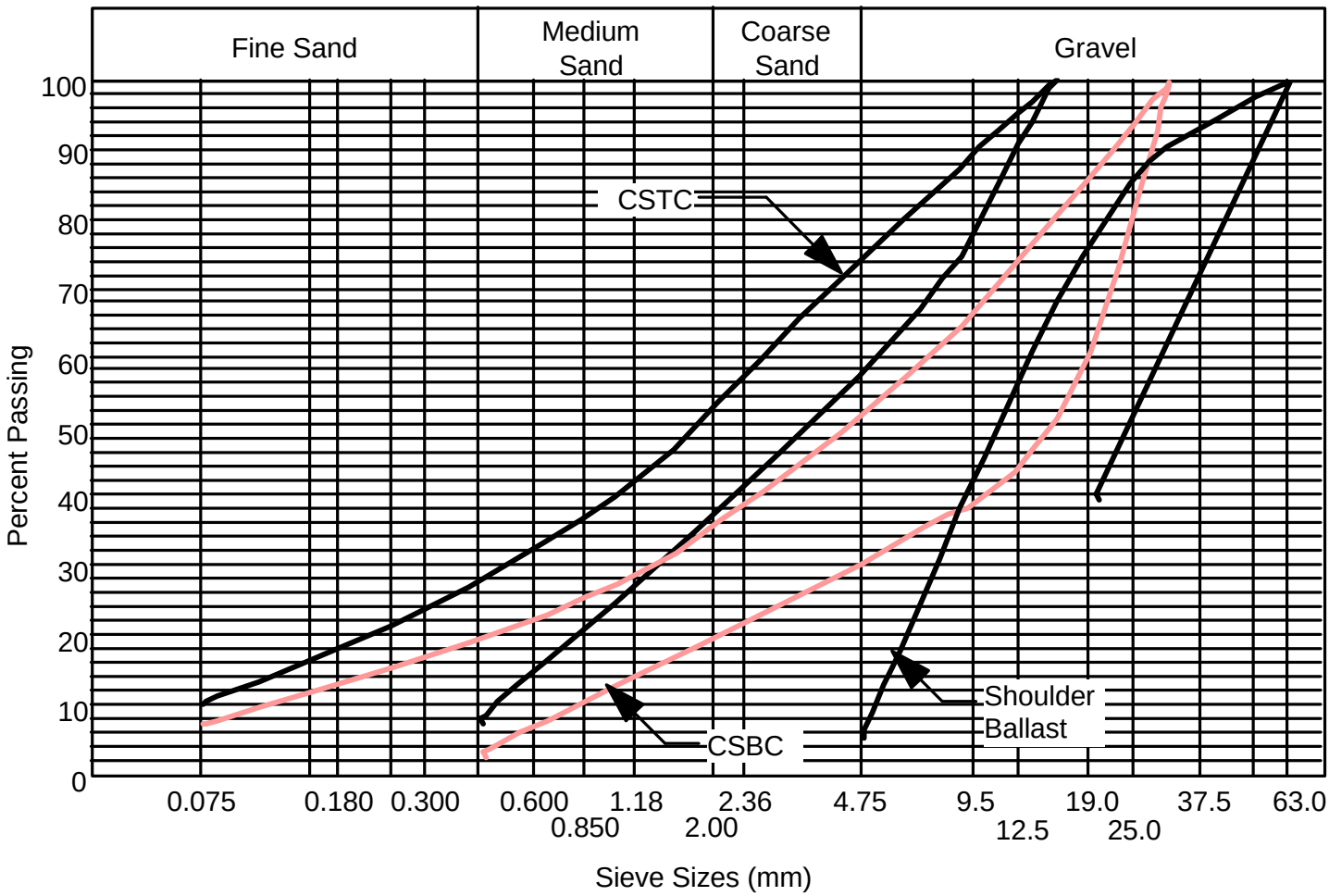


Figure 2.23b. Sketch of Gradation Bands for Crushed Surfacing Top and Base Courses and Shoulder Ballast

Table 2.6. Typical WSDOT Gradations for Crushed Surfacings (Top Course and Base) for Miscellaneous Projects

Size (mm)	Percent Passing							
	Crushed Surfacing							
	Top Course				Base Course			
	1	2	3	4	5	6	7	8
63	—	—	—	—	—	—	—	—
31.5	—	—	—	—	100	100	100	100
19.0	—	—	—	—	—	—	—	—
16.0	100	100	99	100	74	69	61	74
6.3	63	53	49	71	34	29	41	43
0.425	14	15	12	12	8	5	15	16
0.150	—	—	—	—	—	—	—	
0.075	8.1	5.7	5.6	6.0	4.2	3.3	4.7	4.7

Notes

1. Sampled February 25, 1994, SR 395, Spokane County
2. Sampled May 6, 1993, SR 12, Lewis County
3. Sampled August 9, 1993, SR 410, Pierce County
4. Sampled May 17, 1994, SR 90, Spokane County
5. Sampled January 12, 1994, SR 395, Adams County
6. Sampled January 20, 1994, SR 504, Cowlitz County
7. Sampled December 14, 1993, SR 12, Lewis County
8. Sampled June 13, 1994, SR 97, Yakima County

2.7.5.2 Typical Material Specifications

(a) ASTM D1241: Standard Specification for Soil-Aggregate Subbase, Base and Surface Courses.

(b) AASHTO M147: Standard Specification for Materials for Aggregate and Soil-Aggregate Subbase, Base and Surface Courses.

Note: These two standard specifications are slightly different.

2.7.6 COMPACTION REQUIREMENTS

2.7.6.1 Standard compaction tests

(a) AASHTO T99 "Standard AASHTO" (also ASTM D698): Moisture - Density Relations of Soils Using a 5.5 pound Rammer and a 12 inch Drop.

(b) AASHTO T180 "Modified AASHTO" (also ASTM D1557): Moisture - Density Relations of Soils Using a 10 pound Rammer and an 18 inch Drop.

(c) Percent Compaction: $\frac{\text{In-situ dry density}}{\text{Max. dry density}}$ (100) (from compaction curve)

(d) Typical compaction requirements

(i) AASHTO M57: Standard Specification for Materials for Embankments and Subgrades.

2.8 SUBGRADES

2.8.1 AASHTO AND ASTM TERMINOLOGY

A variety of terms can and are used to describe subgrade soils. Such terms at least indirectly relate to strength, stiffness, and other design related parameters. The Unified Soil Classification System and the AASHTO systems will be briefly summarized.

2.8.1.1 AASHTO

The basic reference for the AASHTO classification system is AASHTO M145-87. Particle size terms include:

- Boulders: Material retained on a 75 mm sieve.
- Gravel: Material passing a 75 mm sieve and retained on a 2.00 mm sieve.
- Coarse Sand: Material passing a 2.00 mm sieve and retained on a 425 µm.
- Fine Sand: Material passing a 425 µm and retained on a 75 µm.
- Silt Clay: Material passing a 75 µm sieve.
- "Silty": Fine material (passing 425 µm sieve) with a PI ≤ 10.
- "Clayey": Fine material (passing 425 µm sieve) with a PI ≥ 11.

The AASHTO soil classification system is broadly:

(a) Granular materials: Contain ≤ 35 percent passing a 75 µm sieve. In the AASHTO classification system various groups and subgroups (refer to AASHTO M145 for specific definitions) are used:

- Group A-1 (Subgroups A-1-a, A-1-b)
- Group A-3
- Group A-2 (Subgroups A-2-4, A-2-5, A-2-6, A-2-7)

(b) Silt-Clay materials: Contain > 35 percent passing a 75 µm sieve (again, refer to AASHTO M145 for specific definitions):

- Group A-4
- Group A-5
- Group A-6
- Group A-7 (Subgroups A-7-5, A-7-6)

2.8.1.2 ASTM

The basic reference for the Unified Soil Classification System is ASTM D2487-90. Particle size terms include

(a) Coarse-Grained Soils: More than 50 percent retained on a 75 µm sieve

- Gravel: Material passing a 75 mm sieve and retained on a 4.75 mm sieve.
  - Coarse Gravel: Less than 75 mm sieve and retained on 19.0 mm sieve.
  - Fine Gravel: Less than 19.0 mm sieve and retained on 4.75 mm sieve.

- Sand: Material passing a 4.75 mm sieve and retained on a 75 µm sieve.

  - Coarse Sand: Less than 4.75 mm sieve and retained on a 2.00 mm.
  - Medium Sand: Less than 2.00 mm sieve and retained on a 425 µm.
  - Fine Sand: Less than 425 µm sieve and retained on a 75 µm.

(b) Fine-Grained Soils: 50 percent or more passes a 75 µm sieve

- Clay: Material passing a 75 µm sieve which exhibits plasticity, and strength when dry. Has a PI ≥ 4.

- Silt: Material passing a 75 µm sieve which is nonplastic, and little strength when dry. Has PI < 4.

- Peat: Soil of vegetable matter.

The classification system is (again, details provided in ASTM D2487):

•	GW:	Well-graded gravel	•	CL:	Lean clay
•	GP:	Poorly graded gravel	•	ML:	Silt
•	GM:	Silty gravel	•	OL:	Organic silt or organic clay
•	GC:	Clayey gravel	•	CH:	Fat clay
•	SW:	Well-graded sand	•	MH:	Elastic silt
•	SP:	Poorly graded sand	•	OH:	Organic clay or organic silt
•	SM:	Silty sand	•	PT:	Peat
•	SC:	Clayey sand			

2.8.1.3 Summary

A review of the above definitions reveals modest differences between AASHTO and ASTM with regard to particle size and classification designations.

2.8.2 GEOLOGIC SOIL TERMS

The document entitled *Engineering Geology in Washington* [2.31] contains commonly used geological terms which relate to subgrade soils. The following definitions are largely from Ref. 2.31 modified with a bit of additional background information:

- Alluvial: Sediment deposited by flowing water such as streams or rivers.

- Colluvial: Heterogeneous soil aggregates transported and deposited by processes such as landslides, rockfalls, and avalanches. One dictionary defines colluvial as composed of "filth; excrement; impure matter" — possibly a bit more graphic than the term "heterogeneous."

- Eolian: Wind deposited sediment (Note: Eolian or Aeolian pertains to Aeolus, the mythological Greek god of the winds).
    - Dune Sand: Sand-size sediment
    - Loess: Fine-grained sediment, generally fine sand and silt.
- Glacial: Glacial deposited materials.
    - Till: Heterogeneous materials deposited by glacial ice, typically unstratified.
    - Outwash: Sediment deposited by glacial meltwaters.
- Lacustrine: Lake deposited sediment. The general definition of "lacustrine" is "of or pertaining to a lake" or "found on or in a lake."
- Marine: Marine deposited sediment.
- Residual: Soil created in-place by weathering or chemical processes on the parent material.
- Volcanic: Volcanic derived deposits.

2.8.3 BACKGROUND INFORMATION AND WASHINGTON SOILS

2.8.3.1 Introduction

Background information contained in *Engineering Geology in Washington* [2.32, 2.33] is used to provide a bit more insight into soil characteristics encountered on WSDOT projects.

2.8.3.2 General

After Laprade and Robinson [2.32]:

(a) Western Washington cities such as Seattle, Tacoma, Bellevue, Everett, and Olympia are sited on a variety of glacially derived soil deposits.

(b) Cities such as Bellingham, Bremerton, and Spokane are partially sited on rock outcrops.

(c) Cities such as Vancouver, Yakima, Wenatchee, Mt. Vernon, Kennewick, Richland, and Walla Walla are sited largely on alluvial deposits (however, various sources ranging from rivers to glaciers).

2.8.3.3 Glacial Soils

After Laprade and Robinson [2.32]:

(a) General

- Such soils can have large differences in grain-sizes and compositions.
- Soil characteristics can change within very short distances.
- Whether a glacial soil has been <u>overriden</u> by glaciers (over–consolidated) or <u>not overriden</u> (normally consolidated) significantly affects properties such as stiffness or strength.

(b) Glacially Overriden — Till

- Puget Sound basin thicknesses about 6 to 9 m thick.
- Sometimes referred to as "hardpan."
- Generically granular.
- Can be difficult to excavate.
- Relatively impervious (1×10^{-5} cm/sec or less) and can act as a "perching layer" for water.

(c) Glacially Overriden — Outwash

- Often found in some areas as till.
- In the Puget Sound basin, two notable formations include the Esperance Sand and the Colvos Sand.
    - In general the deposits are
        - Dense to very dense
        - Clean to silty
        - Fine to coarse sand and commonly gravelly
        - Sometimes interbedded with thin silt layers
- Permeabilities range from about 1×10^{-4} to 1×10^{-1} cm/sec. Deposits can be water-bearing.

(d) Glacially Overriden — Glaciolacustrine Deposits

- Generally, these deposits are clayey silts or silty clay — very stiff to hard — mostly of low plasticity.
- Permeabilities are low (1×10^{-6} to 1×10^{-8} cm/sec) except water transmitted through joints and fractures.
- The soil through which the I-90 Mt. Baker Ridge tunnel was placed is glaciolacustrine silt and clay.

(e) Normally Consolidated — Glaciomarine Drift

- Heterogeneous mixture ranging from clay to boulders (mostly clay).
- Deposition by
  - Ice rafting,
  - Subaqueous flows, or
  - Meltwater — fan building.
- Located throughout the northern portion of Puget Sound (Whidbey Island, Bellingham, San Juan Islands, Anacortes).
- Upper 4.5 to 6 m of such material is stiffer due to desiccation and is less stiff with increasing depth — the reverse of what one normally expects.
- Permeabilities are low (1×10^{-5} to 1×10^{-8} cm/sec); however, granular deposits may contain water.

(f) Normally Consolidated — Recessional Outwash

- Outwash material deposited as glaciers receded — generally silty sand and sandy silt.
- Generally loose to medium dense in layers 0.6 to 3.0 m thick.

(g) Normally Consolidated — Depression Fillings [after Refs. 2.32 and 2.33]

- These are generally peaty deposits formed over the last 10,000 - 13,000 years in water-filled basins.
- Settlements of 30 to 50 percent of the thickness of the peat layer can occur. About one-half of the settlement generally will occur within 6 months to 2 years — the remainder over the next 20 years. Settlement will not be uniform.

- Some low volume roads have been built by using geotextiles to separate the road fill material from the soft soil.
- Peat has been located in 28 of Washington's 39 counties.
- Deposits can range in size from 0.4 to 1400 ha (1 to 3,500 acres).
- Moisture contents range from 100 to 500 percent.
- Dry densities range from 160 to 640 kg/m^3 (10 to 40 pcf).

2.8.3.4 Nonglacial Soils

After Laprade and Robinson [2.32]:

(a) Residual Soils

- General soil types are silts and clays.
- Thicknesses are about 1.5 to 3.0 m on Columbia River basalts (southwestern Washington) to 5 to 15 m thick on sedimentary rocks.
- Engineering properties increase with depth.
- Examples of rock formations which have developed significant residual soil deposits include:
    - Skookumchuck Formation near Centralia,
    - Montesano Formation in Grays Harbor area,
    - Ohanapecosh Formation in south-central Washington,
    - Troutdale Formation in southwest Washington, and
    - Columbia River basalts.

(b) Alluvial Soils

- Formed by river deposition. Found in all Washington river valleys.
- Generally sand and/or silt.
- Deposits often loose and soft to significant depths.
- Due to location in river valleys, expect shallow groundwater tables.
- Generally expect settlement in such soils.

(c) Eolian Soils [after Refs. 2.32 and 2.33]

- Loess soils of high interest (wind-blown silt) due to large portions of eastern Washington state covered by same.
- Common loess soil is "Palouse soil" or "Palouse silt."

- Vertical porosity is 100 to 1,000 times greater than horizontal porosity in undisturbed state (due to root holes). Thus, vertical drainage can be good.

- Dry unit weights about 1440 to 1520 kg/m^3 (90 to 95 pcf).

- Material is primarily rock flour generated by Pleistocene glaciers which covered the northern parts of the state.

- Typically silt with some clay and fine sand.

- Typical engineering properties

  - SPT N = 20 to 30 blows/ft
  - Moisture contents: 15 to 30 percent
  - LL = 35 to 45
  - PI = 10 to 20

(d) Colluvial Soils

- Loose to medium-dense soils at the sides and toes of slopes.

- Grain-sizes extremely variable.

- Generally more pervious than underlying soil — results in frequent perched groundwater at the interface.

2.8.4 SOIL PROPERTY RANGES

2.8.4.1 Introduction

A few soil properties are included in this subsection which may be of occasional assistance.

2.8.4.2 Coefficients of Permeability

From Terzaghi and Peck [2.34] the permeability ranges can help in interpreting earlier subparagraphs:

Degree of Permeability	k (cm/sec)
High	1×10^{-1}
Medium	1×10^{-1} to 1×10^{-3}
Low	1×10^{-3} to 1×10^{-5}
Very Low	1×10^{-5} to 1×10^{-7}
Practically Impermeable	less than 1×10^{-7}

2.8.4.3 **Standard Penetration Test** (in accordance with AASHTO T206-87: Penetration Test and Split-Barrel Sampling of Soils)

(a) Clay Consistency (from Terzaghi and Peck [2.34])

Consistency	N (Blows/ft)
Very Soft	< 2
Soft	2-4
Medium	4-8
Stiff	8-15
Very Stiff	15-30
Hard	> 30

(b) Sand Strata Relative Density (from Terzaghi and Peck [2.34])

Relative Density	N (Blows/ft)
Very Loose	0-4
Loose	4-10
Medium	10-30
Dense	30-50
Very Dense	>50

2.8.4.4 **Relationships Between Geologic and the Unified Soil Classifications**

The approximate relationships between the geologic and USCS systems along with dry densities is summarized below. This information was obtained from Koloski, et al. [2.31]:

Classification		Classification	
Geologic	USCS	Geologic	USCS
• Alluvial		• Lacustrine	
• High Energy	GW, GP, GM	• Inorganic	ML, SM, MH
• Low Energy	ML, SM, SP, SW	• Organic	OL, PT
• Colluvial	Variable	• Residual	Variable
• Eolian		• Volcanic	
• Dune Sand	SP	• Tephra	ML, SM
• Loess	ML, SM	• Lahar	SM, SW, GM
• Glacial			
• Till	SM, ML		
• Outwash	GW, GP, SW, SP, SM		
• Glaciolacustrine	ML, SM, SP		

2.8.4.5 Resilient Moduli

There has been little work attempting to tie an important pavement material property, resilient modulus, to other soil properties and classifications. As such information is gained, this subparagraph will be updated.

Triaxial results on Palouse silt have indicated moduli in the range of 55 to 76 MPa (dry densities of about 1890 kg/m^3 (118 pcf) and a moisture content of 15 percent).

3. ENVIRONMENT

The environment has a significant impact on pavement performance. It is often difficult to assess damage caused by climate variables. Variables to consider:

(a) Frost (refer to Paragraph 3.1)
(b) Rainfall and seasonal distribution
(c) Temperature (daily and seasonal)
(d) Location of groundwater table

Table 2.7 is used to present selected climate information for various cities worldwide. Included in this table are mean temperatures for January, July, and yearly (and standard deviation of the 12 monthly temperatures). Such measures illustrate the temperature extremes (January and July) and overall average (yearly). Also included are annual precipitation and the number of days per year with measurable precipitation (≥ 0.25 mm/day in U.S. and Canada; ≥ 1 mm/day elsewhere). (Note that the more northern cities have the most days with measurable precipitation—in general.) Figure 2.24 contains several plots of mean monthly temperatures for selected cities. Based on such temperature data alone, Seattle is similar to London; Spokane is similar to Berlin; and Phoenix, Houston, and Riyadh all share common trends.

A classification of climates by Griffiths [2.29] is of assistance:

Classification	Condition
Hot	Mean temperature of all months ≥18 °C (64.4 °F)
Warm	Mean temperature of all months ≥ 6 °C (42.8 °F)
Short Winter	Mean temperature of 7 to 11 months ≥ 6 °C (42.8 °F)
Long Winter	Mean temperature of 3 to 6 months ≥ 6 °C (42.8 °F)
Cold	Mean temperature of 0 to 2 months ≥ 6 °C (42.8 °F)

Table 2.7 Climate Statistics—Climate and Temperature Summaries—Various World Cities

Location	Temperatures								Precipitation		Number of Days with Measurable Precipitation
	January Mean		July Mean		Yearly Mean		Yearly Std. Dev.				
	°C	(°F)	°C	(°F)	°C	(°F)	°C	(°F)	mm	(in.)	
United States											
• Seattle, WA	4.4	(40)	18	(64)	11	(52)	-13	(8)	838-991	(33-39)	150
• Spokane, WA	-2.2	(28)	21	(70)	9.4	(49)	-9.4	(15)	381	(15)	113
• Portland, OR	-3.9	(39)	19	(67)	12	(53)	12	(10)	1 067	(42)	153
• Boise, ID	-1.1	(30)	23	(74)	11	(51)	-9.4	(15)	330	(13)	78
• Salt Lake City, UT	-3.3	(26)	24	(76)	11	(51)	-8.3	(17)	406	(16)	90
• Las Vegas, NV	6.7	(44)	30	(86)	18	(64)	-9.4	(15)	102	(4)	18
• Phoenix, AZ	11	(52)	33	(91)	21	(70)	-10	(14)	203	(8)	39
• San Diego, CA	13	(55)	20	(68)	17	(62)	15	(5)	254	(10)	44
• San Francisco, CA	10	(50)	15	(59)	14	(57)	-16	(4)	559	(22)	67
• Anchorage, AK	-11	(12)	14	(57)	2.2	(36)	-7.2	(19)	381	(15)	97
• Fairbanks, AK	-24	(-11)	16	(60)	-3.3	(26)	-3.3	(26)	305	(12)	111
• Boston, MA	-2.2	(28)	22	(72)	10	(50)	-8.9	(16)	1 041	(41)	125
• Washington, D.C.	1.1	(34)	26	(78)	13	(56)	-8.9	(16)	1 067	(42)	124
• Atlanta, GA	6.1	(43)	26	(78)	16	(61)	-10	(14)	1 245	(49)	124
• Minneapolis, MN	-10	(14)	23	(73)	7.2	(45)	-2.2	(28)	737	(29)	108
• Chicago, IL	-1.1	(30)	24	(75)	10	(50)	-7.8	(18)	914	(36)	139
• Dallas, TX	7.8	(46)	29	(84)	19	(66)	10	(14)	940	(37)	85
• Houston, TX	12	(53)	28	(83)	21	(69)	-12	(11)	1 168	(46)	98
• Miami, FL	20	(68)	28	(82)	24	(75)	-14	(6)	1 499	(59)	135
Canada											
• Toronto, ON	-5.0	(23)	21	(69)	7.2	(45)	-8	(17)	813	(32)	145
• Edmonton, AL	-14	(6)	17	(62)	2.8	(37)	-7	(20)	432	(17)	133
• Vancouver, BC	2.2	(36)	18	(64)	10	(50)	-12	(10)	1 448	(57)	172
Europe											
• Brussels, Belgium	1.7	(35)	18	(64)	10	(50)	-12	(11)	864	(34)	208
• Prague, Czech Republic	-2.8	(27)	18	(64)	7.8	(46)	-10	(14)	406	(16)	143
• Copenhagen, Denmark	0.0	(32)	18	(64)	8.9	(48)	-11	(12)	610	(24)	171
• Helsinki, Finland	-5.6	(22)	17	(63)	5.0	(41)	-10	(15)	686	(27)	191
• Paris, France	3.3	(38)	19	(67)	12	(53)	-12	(11)	610	(24)	162
• Berlin, Germany	-0.6	(31)	19	(66)	8.9	(48)	-11	(13)	610	(24)	167
• Frankfurt, Germany	1.1	(34)	20	(68)	11	(51)	-11	(13)	686	(27)	173
• London, GB	4.4	(40)	18	(64)	11	(51)	-13	(9)	584	(23)	153
• Reykjavik, Iceland	0.0	(32)	11	(52)	5.0	(41)	-13	(8)	787	(31)	213

Table 2.7 Climate Statistics—Climate and Temperature Summaries—Various World Cities (Continued)

Location	Temperatures								Precipitation		Number of Days with Measurable Precipitation
	January Mean		July Mean		Yearly Mean		Yearly Std. Dev.				
	°C	(°F)	°C	(°F)	°C	(°F)	°C	(°F)	mm	(in.)	
Europe											
• Dublin, Ireland	4.4	(40)	16	(60)	9.4	(49)	-14	(7)	762	(30)	139
• Rome, Italy	7.8	(46)	25	(77)	16	(61)	-12	(11)	762	(30)	77
• Oslo, Norway	-4.4	(24)	18	(64)	6.1	(43)	-9.4	(15)	711	(28)	160
• Madrid, Spain	5.0	(41)	24	(75)	14	(57)	-11	(12)	457	(18)	87
• Stockholm, Sweden	-3.3	(26)	18	(64)	6.7	(44)	-10	(14)	559	(22)	164
• Zürich, Switzerland	-0.6	(31)	19	(66)	9.4	(49)	-11	(13)	1 092	(43)	170
Russia											
• St. Petersburg	-10	(14)	17	(62)	4.4	(40)	-7.8	(18)	610	(24)	191
• Moscow	-13	(9)	18	(64)	4.4	(40)	-6.7	(20)	635	(25)	181
South Africa											
• Durban	24	(75)	17	(62)	21	(69)	-15	(5)	1 016	(40)	84
• Johannesburg	20	(68)	11	(51)	16	(61)	-14	(7)	711	(28)	70
• Cape Town	21	(69)	12	(54)	17	(62)	-14	(6)	508	(20)	69
People's Republic of China											
• Beijing	-4.4	(24)	26	(79)	12	(53)	-6.1	(21)	610	(24)	66
Hong Kong	16	(60)	28	(82)	22	(72)	-13	(9)	2 159	(85)	110
Japan											
• Tokyo	3.3	(38)	24	(76)	14	(58)	-9.4	(15)	1 575	(62)	107
Middle East											
• Kuwait City	13	(55)	34	(94)	25	(77)	-9.4	(15)	127	(5)	11
• Beirut	13	(56)	27	(80)	21	(69)	-13	(9)	889	(35)	68
• Jidda	24	(75)	32	(89)	28	(82)	-15	(5)	51	(2)	5
• Riyadh	14	(58)	33	(92)	25	(77)	-11	(13)	76	(3)	10
Australia											
• Canberra	20	(68)	5.6	(42)	13	(56)	-12	(10)	584	(23)	101
• Melbourne	20	(68)	9.4	(49)	14	(58)	-14	(7)	660	(26)	156

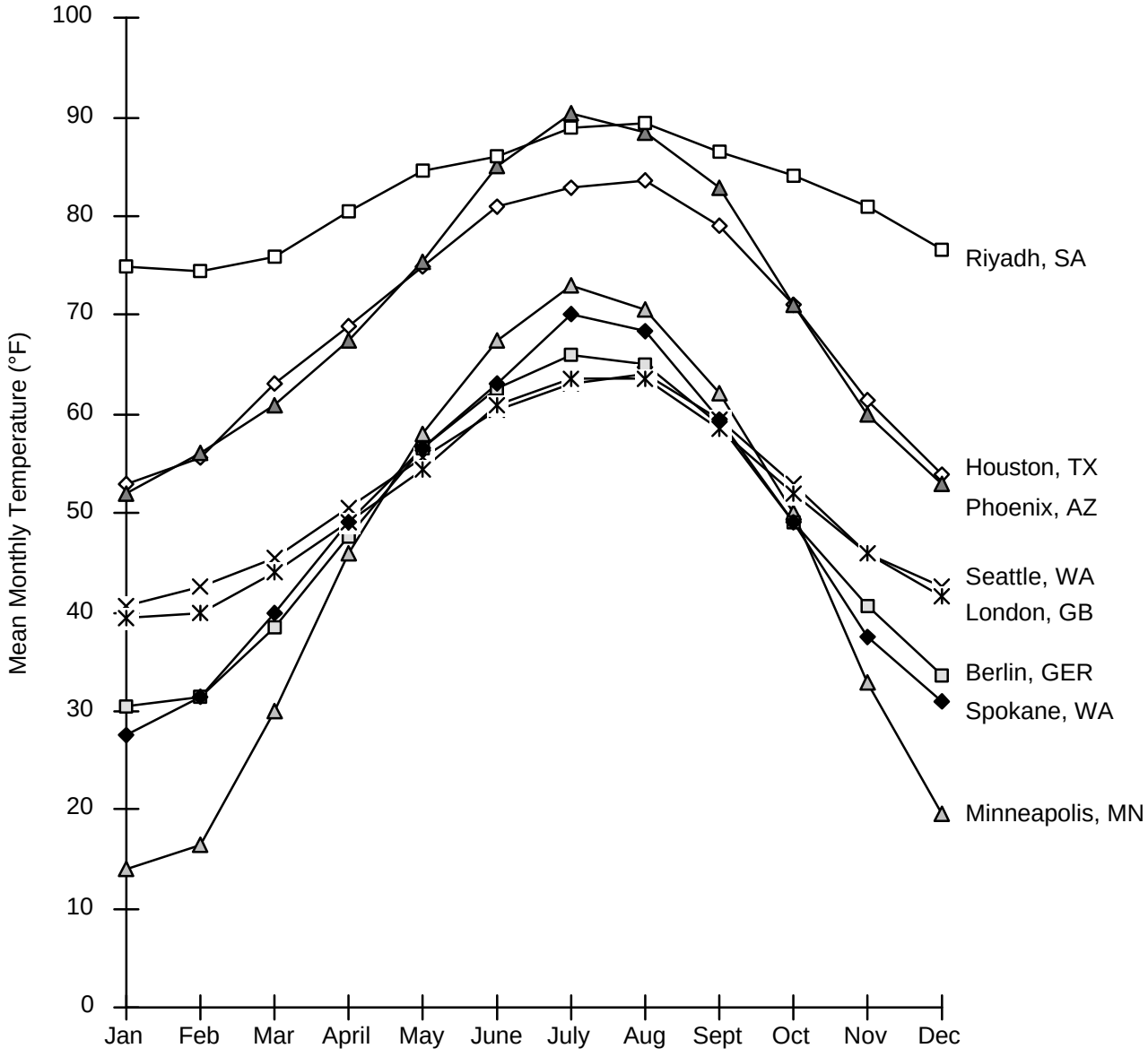


Figure 2.24. Seasonal Temperatures for Various Cities

Using the above classification scheme, the following cities are:

City	Climate
Seattle	Short Winter
Spokane	Short Winter
Minneapolis	Short Winter
Berlin	Short Winter
London	Short Winter
Phoenix	Warm
Houston	Warm
Riyadh	Warm

What the above information hopefully provides are a few basic points such as:

(a) The range of climates in the U.S. encompass much of the climate ranges seen throughout the world. For example, if a pavement material or system can "work well" in Phoenix, Seattle, or Minneapolis, it can work well most anywhere else.

(b) The climate of western Washington (as characterized by Seattle) shares similarities with Great Britain and much of central Europe.

(c) The climate of eastern Washington (as characterized by Spokane) shares similarities with parts of Europe and Scandinavia.

(d) The temperature extremes for the states (Table 2.8) are generally within the same range (both highs and lows). The largest temperature range (difference between extreme high and low) is Montana 104 °C (187 °F), and the smallest is Florida 62 °C (111 °F). Washington has a range of 92 °C (166 °F).

Most of what follows in Paragraph 3 will concentrate on frost action in pavements — which is of major interest in Washington state. However, the preceding hopefully suggests that the broader climate characterization (temperature, precipitation) of Washington state is not particularly unique and we do share common characteristics with much of the world. Thus, a pavement advance made in Paris, London, or Stockholm may work equally well in Washington (at least based on climate).

Table 2.8 Climate Statistics—Temperature Extremes for Continental U.S. States

State	Highest Recorded Temperatures °C	(°F)	Lowest Recorded Temperatures °C	(°F)
Alabama	44	(112)	-33	(-27)
Arizona	53	(127)	-40	(-40)
Arkansas	49	(120)	-34	(-29)
California	57	(134)	-43	(-45)
Colorado	48	(118)	-52	(-61)
Connecticut	41	(105)	-36	(-32)
Delaware	43	(110)	-27	(-17)
Florida	43	(109)	-19	(-2)
Georgia	44	(112)	-27	(-17)
Idaho	48	(118)	-51	(-60)
Illinois	47	(117)	-37	(-35)
Indiana	47	(116)	-37	(-35)
Iowa	48	(118)	-44	(-47)
Kansas	49	(121)	-40	(-40)
Kentucky	46	(114)	-37	(-34)
Louisiana	46	(114)	-27	(-16)
Maine	41	(105)	-44	(-48)
Maryland	43	(109)	-40	(-40)
Massachusetts	42	(107)	-37	(-35)
Michigan	44	(112)	-46	(-51)
Minnesota	46	(114)	-51	(-59)
Mississippi	46	(115)	-28	(-19)
Missouri	48	(118)	-40	(-40)
Montana	47	(117)	-57	(-70)

State	Highest Recorded Temperatures °C	(°F)	Lowest Recorded Temperatures °C	(°F)
Nebraska	50	(118)	-44	(-47)
Nevada	41	(122)	-46	(-50)
New Hampshire	43	(106)	-43	(-46)
New Jersey	47	(110)	-37	(-34)
New Mexico	42	(116)	-46	(-50)
New York	42	(108)	-47	(-52)
North Carolina	43	(110)	-37	(-34)
North Dakota	49	(121)	-51	(-60)
Ohio	45	(113)	-39	(-39)
Oklahoma	49	(120)	-33	(-27)
Oregon	48	(119)	-48	(-54)
Pennsylvania	44	(111)	-41	(-42)
Rhode Island	40	(104)	-31	(-23)
South Carolina	44	(111)	-28	(-19)
South Dakota	49	(120)	-50	(-58)
Tennessee	45	(113)	-36	(-32)
Texas	49	(120)	-31	(-23)
Utah	47	(117)	-56	(-69)
Vermont	41	(105)	-46	(-50)
Virginia	43	(110)	-34	(-30)
Washington	48	(118)	-44	(-48)
West Virginia	44	(112)	-38	(-37)
Wisconsin	46	(114)	-48	(-54)
Wyoming	46	(114)	-53	(-63)

3.1 FROST ACTION IN PAVEMENTS

3.1.1 INTRODUCTION

Frost action refers to two separate but related processes:

(a) Frost heaving resulting mainly from accumulation of moisture (ice lenses) in the soil during the freezing period (note: ice lenses form perpendicular to the direction of heat flow), and

(b) Thaw weakening of soil when thawing temperatures occur.

The information which follows in this paragraph (3.1) and Paragraph 3.2 will be of benefit to those who must design and maintain pavements in frost areas of Washington State.

3.1.2 CONDITIONS NECESSARY FOR FROST HEAVE TO OCCUR

(a) subfreezing temperatures
(b) water, and
(c) frost susceptible soil

Remove any of the three conditions above and frost effects will be eliminated or at least minimized. If the three conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur resulting in pavement cracking and roughness. Additional factors which will affect the degree of frost susceptibility (or ability of a soil to heave):

(d) rate of heat removal,
(e) temperature gradient,
(f) mobility of water (e.g. permeability of soil),
(g) depth of water table, and
(h) soil type and condition (e.g. density, texture, structure, etc.)

3.1.3 DETAILS ABOUT FACTORS FUNDAMENTAL TO FROST ACTION

3.1.3.1 Heaving

Frost heaving of soil is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins, or other ice masses. An ice lens grows in thickness in the direction of heat transfer until the water supply is depleted or until freezing conditions at the freezing interface no longer support further crystallization. Ice segregation occurs primarily in soils containing fine particles (i.e., frost susceptible). Clean sands and gravels are non-frost susceptible (NFS). The amount of frost susceptibility is mainly a function of the percentage of fine particles

(more on this later). Figure 2.25 illustrates the formation of ice lenses in a frost susceptible soil.

(a) Recall, the conditions necessary for ice lenses (or segregation) are:

(i) Soil (must be frost susceptible)

(ii) Temperature (freezing temperatures must penetrate the soil and, in general, the thickness of an ice lens will be thicker with slower rates of freezing), and

(iii) Water (must be available from the groundwater table, infiltration, an aquifer, or held within the voids of fine-grained soil).

(b) Uniform heaving: Generally not feasible to preclude heave, thus the usual objective is to reduce its magnitude and make it more uniform.

(c) Differential heave: This is more likely to occur at locations such as:

(i) Where subgrades change from clean NFS sands to silty frost susceptible materials.

(ii) Abrupt transitions from cut to fill with groundwater close to the surface.

(iii) Where excavation exposes water-bearing strata.

(iv) Drains, culverts, etc., frequently result in abrupt differential heaving due to different backfill material or compaction and the fact that open buried pipes change the thermal conditions (i.e., remove heat resulting in more frozen soil).

Tabor, in 1930 [2.41], recognized that frost heaving required substantially more water than was naturally available in the soil pores (characterized as "moisture content"). He noted:

> "The average soil seldom contains as much as 50 percent water, but if all the water in such a soil were to freeze in situ, the change in volume could cause an uplift of less than 5 percent of the depth of freezing. The depth of freezing in the colder parts of the United States seldom exceeds 2 or 3 feet; yet a surface heaving of 6 inches is not uncommon and an uplift of a couple of feet has been reported."

Figure 2.25 illustrates the important role capillary water "plays" in frost heaving.

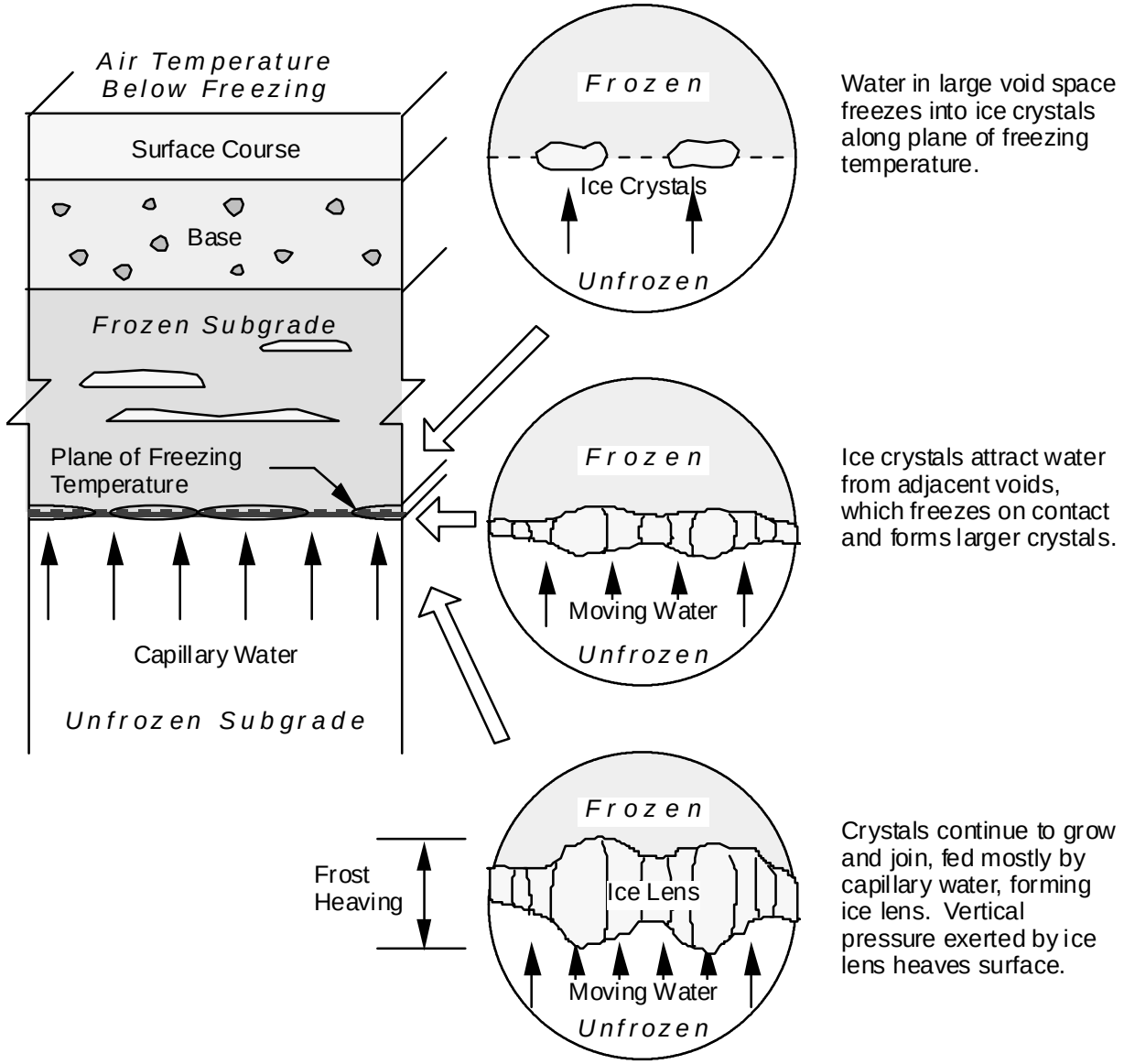


Figure 2.25. Formation of Ice Lenses in a Pavement Structure

The capillary rise of water can be substantial, up to 6 m (20 ft) or more. The potential capillary rise can be estimated by the following:

$$h_C = \frac{C}{(e)\,(D_{10})}$$

where h_c = capillary rise (cm),
e = void ratio
D_{10} = soil particle size, 10 percent finer passing (cm), and
C = constant which can range from 0.1 to 0.5 cm^2

Thus, the smaller the soil grain size, the greater the potential for vertical water movement. Silty soils present the greatest problem, the very type of soil which covers much of central and eastern Washington.

A quote by Lobacz, et al. [2.45] further illustrates the serious nature of capillary rise:

> "A potentially troublesome water supply for ice segregation is present if the highest ground water table at any time of the year is within 5 ft of the proposed subgrade surface or the top of any frost-susceptible base materials used. When the depth to the uppermost water table is in excess of 10 ft throughout the year, ice segregation and frost heave may be expected to be reduced."

Note that Lobacz et al. stated that a water table with a depth greater than 10 ft only reduces the potential for ice lenses.

Use of geotextiles: Geotextiles have frequently been used by WSDOT as separators and occasionally as filters. Work performed by Bell et al. [2.5] at Oregon State University suggests that geotextiles have potential as capillary breaks.

3.1.3.2 Thawing

(a) Temperature distribution during thawing: Thawing can proceed from the top downward, or from the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take.

(b) Loss of bearing (or load) capacity: Loss of bearing capacity during "spring" thaw periods is one of the most serious problems associated with frost action. The usual pattern of seasonal variation in base and subgrade support includes (usually) a significant increase from "normal" summer/fall values during the time the base and subgrade is frozen. Thawing produces a rapid decrease to levels below the

summer/fall values, followed by a gradual recovery over a period of weeks (or months). Figure 2.26 illustrates this process.

Tabor [2.41] also noted an added effect:

> "The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition."

This is helpful in two ways: (1) the reduced density of base or subgrade materials helps to explain the long recovery period for material stiffness or strength following thawing, and (2) refreezing following an initial thaw can create the potential for greater weakening when the "final" thaw does occur (usually late February or early March for much of central and eastern Washington).

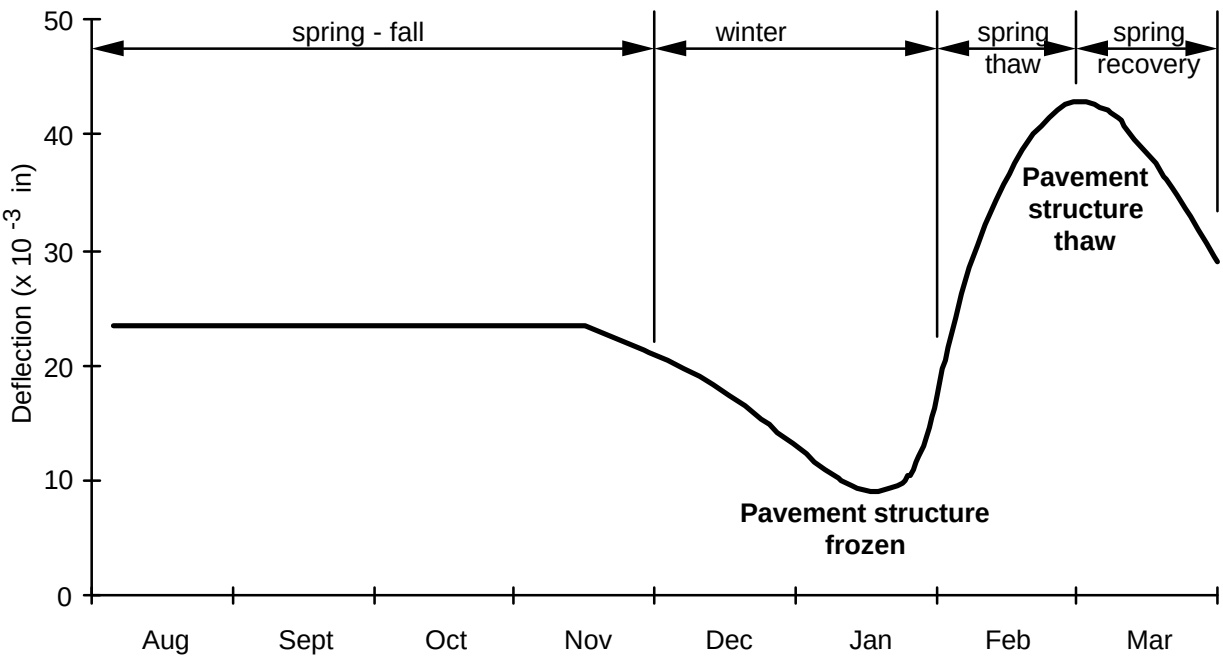


Figure 2.26. Typical Pavement Deflections Illustrating Seasonal Pavement Strength Changes (SR 172 - WSDOT Region 2)

(c) Differential thaw: If thawing does not proceed at the same rate over all parts of a paved area, nonuniform subsidence of a previously heaved surface results. Differential thaw can be due to any of the following

(i) differing thermal properties of adjacent sections of pavement (e.g. non-uniform soil strata),

(ii) nonuniform exposure to sun's rays (e.g. shaded portions of pavement due to trees, deep cuts, buildings, overpasses, etc.),

(iii) differing surface and subsurface drainage conditions, and

(iv) differing color of pavement.

(d) Migration of coarse particles: As a result of freeze-thaw cycles over a period of years, large stones present within the frost zone in a subgrade or base course that is frost susceptible will eventually be heaved upward to the surface (results in pavement roughness and structural damage).

3.1.3.3 Criteria for Identifying Frost Susceptible Soil

(a) Introduction: Most studies have shown that a soil is susceptible to frost action only if it contains fine particles. Early investigations found that soils free of fines, comprising only particles retained on the 0.075 mm mesh sieve, did not develop significant ice lenses. It has been observed that other soil properties — such as overall grain size distribution (texture), grain shape, mineral composition, and plasticity characteristics — contribute in varying amounts.

(b) Casagrande criterion: This individual in 1932 proposed the following widely known rule-of-thumb criterion for identifying potentially frost susceptible soils:

> "Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

Application of the Casagrande criterion requires a hydrometer test of a soil suspension (in water) to determine the distribution of particles passing the 0.075 mm sieve and to compute the percentage of particles finer than 0.02 mm.

(c) U.S. Army Corps of Engineers frost design classification: The Corps of Engineers frost design classification system was developed in the late 1940s to make use of the Casagrande criterion regarding frost susceptibility and to account for the reduced stability of the various types of frost susceptible soils during the thaw-weakened period.

In this system frost susceptible soils (with 3 percent or more, by weight, finer than 0.02 mm) are classified into one of four groups, F1, F2, F3, or F4, for frost design purposes. Soil types are listed in Table 2.9 in approximate order of increasing susceptibility to frost heaving and/or weakening as a result of frost melting. The basis for distinction between the F1 and F2 groups is the F1 material may be expected to show higher bearing capacity than F2 material during thaw, even though both may have experienced equal ice segregation. The F3 and F4 soils, grouped together for reduced strength design, show the greatest weakening during thaw.

Table 2.9. Corps of Engineers Frost Design Soil Classification and USCS Equivalent Grouping (after NCHRP Synthesis 26, 1974)

Frost Group	Soil Type	Percentage finer than 0.02 mm by weight	Typical soil types under Unified Soil Classification System
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	(a) Gravelly soils	>20	GM, GC
	(b) Sands, except very fine silty sands	>15	SM, SC
	(c) Clays, Pl > 12	–	CL, CH
F4	(a) All silts	–	ML, MH
	(b) Very fine silty sands	>15	SM
	(c) Clays, Pl < 12	–	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	–	CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

In the early 1950s a laboratory freezing test was developed by the Corps of Engineers, principally to determine the suitability of silty granular materials whose gradation, compared with the existing Casagrande criterion, caused uncertainty regarding their suitability for use as a base and subbase in airfield pavements. The freezing test was also intended to serve as a basis for identification of frost-susceptible soils and for classification according to their relative degree of frost susceptibility. The test consists of unidirectional laboratory freezing from the top downward, at a relatively slow rate, on cylindrical soil specimens, 6 inches high and 6 inches in diameter, in an open-system test where free water is available at the base of the specimen. Specimen containers are insulated on the sides, slightly tapered inside (wider at the top), and lubricated to reduce wall friction. The heave rate is measured under a freezing penetration rate of one quarter to one half inch per day. A minimum surcharge pressure of 3.4 kPa psi is applied to each specimen to stimulate the overburden pressure of minimum thickness of 150 mm of pavement and base.

The freezing test thus subjects the soil to a severe combination of conditions conductive to frost heaving. The unlimited source of water corresponds to an extremely pervious aquifer only 6 inches or less below the freezing plane and it results in virtually the maximum possible rate of ice segregation and heave that the soil could exhibit under natural field conditions. The results do not quantitatively represent the actual magnitude of heave to be anticipated in the field. They provide, however, a satisfactory relative measure of potential frost behavior. The test has been used by the Corps of Engineers for more than 20 years for design and construction of military airfield pavements and roads. The following scale of frost susceptibility based on the average measured rate of heave in laboratory tests has been adopted:

Average Rate of Heave (mm/day)	Frost Susceptibility Classification
0.0 - 0.5	Negligible
0.5 - 1.0	Very Low
1.0 - 2.0	Low
2.0 - 4.0	Medium
4.0 - 8.0	High
>8.0	Very High

Corps of Engineers data reveal that there is no sharp dividing line between frost susceptible and non-frost susceptible soils, not a unique, well-defined relationship with respect to percentage of particles finer than 0.02 mm. The data confirm indications of earlier investigators that factors other than grain size influence frost behavior.

There has been debate as to whether the COE frost classification system is an adequate scheme for identifying soils that have significant potential for thaw weakening. It is, in fact, more oriented toward identifying soils that have frost

heaving potential. Not all soils share the same characteristics (i.e., frost heaving and thaw weakening) as was noted in a discussion at the Symposium on Frost Action in Roads held in Oslo, Norway, in October 1973 (Summary of Discussions, Session III, Freezing Mechanism and Criteria). An example noted at the Symposium was that numerous plastic clays are highly susceptible to strength loss but experienced little volume change.

WSDOT's continued emphasis on grain size measurements and resilient moduli represents a logical approach; however, more frequent use of hydrometer analysis to determine grain size distributions for minus 0.075 mm material is appropriate. As shown in Table 2.9, percent finer than 0.02 mm is of special interest.

3.1.3.4 Sources of Water

Sources of water contributing to frost problems in pavement can be separated into two broad categories: surface and subsurface water. Refer to Figure 2.27.

(a) Surface water: Enters the pavement primarily by infiltration through surface cracks and joints, and through adjacent unpaved surfaces, during periods of rain and melting snow and ice. Many crack-free pavements are not entirely impermeable to moisture.

(b) Subsurface water: Three primary sources

(i) Groundwater table (or perched watertable),

(ii) moisture held in soil voids or drawn upward from a watertable by capillary forces, and

(iii) moisture that moves laterally beneath a pavement from an external source (i.e., pervious water bearing strata, etc.).

3.1.4 FROST ACTION

3.1.4.1 Introduction

A 1993 summary on state pavement design practices relative to "frost heave" was prepared by Forsyth [2.43]. The question posed by Forsyth's survey was: "Does your state's flexible [or rigid] pavement design procedure include consideration of serviceability loss due to frost heave?"

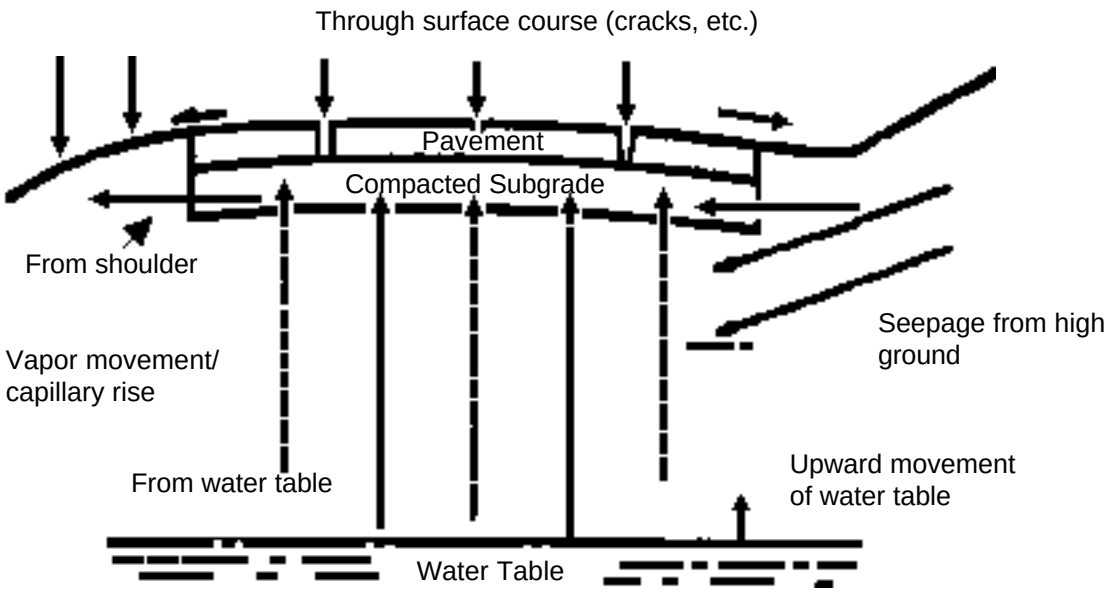


Figure 2.27. Sources of Water Contributing to Frost Problems in Pavements (after NCHRP Synthesis 26, 1974)

The following selection of responses were noted.

State	Response
Alaska	Control minus 0.075 mm sieve size material to a depth of 1.1 m (42 in.)
Arizona	Judgment
Georgia	AASHTO (72) Regional Factor
Illinois	Top foot of subgrade chemically modified or replaced
Maine	Minimum of 0.9 m (36 in.) of pavement and gravel based on degree days
Massachusetts	Increase Structural Number
Michigan	Replace to a depth of 1.5 m (5 ft) from pavement surface
Minnesota	Blend frost susceptible soils to frost depth 1.2 to 1.8 m (4 to 6 ft)
Nebraska	AASHTO (86)
New Mexico	AASHTO (72) Regional Factor
Ohio	Frost susceptible material removed to a depth of 0.9 m (3 ft) from the pavement surface
Utah	Remove and replace frost susceptible material or increase pavement thickness
West Virginia	AASHTO 86
Ontario	Increase base and subbase thickness based on frost susceptibility of subgrade soils

Thus, the various states responding to Forsyth's question have a variety of approaches. Clearly, a commonly used technique is to remove or modify frost susceptible materials to some preset depth from the pavement surface. Many of those depths reported appear to approach the expected depth of freeze, but certainly not all. Another way to view this is that the pavement structure is increased to ensure that frost susceptible materials are at some acceptable depth as measured from the pavement surface.

Several states use the process described in the 1993 AASHTO Guide [2.36], or one of its earlier versions (the AASHTO Regional Factor (pre-1986 Guide) is more fully described in SECTION 5.0, Paragraph 1.8.5).

In subsequent paragraphs, the following will be briefly overviewed with respect to pavement frost design considerations:

- Practices of Washington cities and counties
- AASHTO Guide (1993)
- Corps of Engineers
- Granular or Rock Caps
- Survey of Practice — 1963
- Other Design Considerations

3.1.4.2 Washington State Cities and Counties

A series of questions were posed to Washington State, City, County, and WSDOT Regions (formerly Districts) concerning pavement frost design issues. These results were compiled by the Washington State Policy Plan Subcommittee on Weight Restrictions and Road Closures [2.44]. One of the questions asked was: "What do you feel are the most important factors in causing road deterioration in your jurisdiction?"

Of a wide range of possible responses, three of the top-ranked factors (frost heaving, road use during freeze/thaw, excess subgrade moisture (which may not be frost related)) constituted the following percentages of all responses:

- Counties: 46%
- Cities: 48%
- State: 30%

Thus, frost-related factors are considered to be very important in contributing to road deterioration in Washington State. Of those agencies which noted spring thaw conditions, typically 30 percent of their route system experiences seasonal structural weakening.

Another frost-related question relates to how frost effects are considered in the design of new pavements. Of the six counties which responded (Chelan, Columbia, Lincoln, Skamania, Walla Walla, and Whatcom), all stated that extra base course thickness was used. Of the cities which state that frost design is a consideration, they generally use an extra thickness of base course as well.

3.1.4.3 AASHTO Guide (1993)

The AASHTO Guide (1993) contains a treatment dealing with frost heave in pavement design. The goal is to estimate the differential effects on the road profile and ultimately to estimate the heaves residual effects on the Present Serviceability Index (PSI). Thus,

the decrease in PSI with time due to frost effects is "overlayed" onto the loss of PSI due to ESALs.

To date, WSDOT has not used this procedure in its design process. In general, the AASHTO treatment of frost heaving appears to have limited value for application in Washington State.

3.1.4.4 Corps of Engineers

Since World War II, the Corps of Engineers has developed pavement design procedures (street and airfield) which can be used to develop structural design requirements. The available design procedures for pavements subject to freezing and thawing in the underlying soils are based on two basic concepts (Lobacz et al. [2.45]):

- Control of surface deformation resulting from frost action.
- Provision for adequate bearing capacity during the most critical climate period.

Based on the above concepts, three separate design approaches can be used:

- Complete Protection Method

  Sufficient thickness of pavement and non-frost susceptible base course is provided to prevent frost penetration into the subgrade.

- Limited Subgrade Frost Penetration Method

  Sufficient thickness of pavement and non-frost susceptible base course is provided to limit subgrade frost penetration to amounts which restrict surface deformation to within acceptable, small limits.

- Reduced Subgrade Strength Method

  The amount of frost heave is neglected and the design is based primarily on the anticipated reduced subgrade strength during the thaw period.

Yoder and Witczak [2.46] noted in reference to the three design approaches mentioned above:

> "…design of highway pavements should be based generally on the <u>reduced subgrade strength design method</u>, with additional thickness (based on local field data and experience) used where necessary to keep pavement heave and cracking within tolerable amounts."

The design period traffic is developed in terms of 80 kN (18,000 lb) single-axle loads. By use of design charts in Lobacz et al. [2.45] and traffic information in Yoder and

Witczak [2.46] and the National Stone Association [2.51], the following thicknesses were developed from the COE design charts:

Flexible Pavement—Combined Thickness of Surface Course
and Non-Frost Susceptible Base, mm (in.)

Traffic [1, 2]		COE Subgrade Frost Group[3]		
COE Design Index	Upper Limit of ESAL Range	F1	F2	F3 and F4
DI-1	1,825/yr	225 (9)	250 (10)	400 (16)
DI-2	7,300/yr	250 (10)	300 (12)	475 (19)
DI-3	27,375/yr	300 (12)	350 (14)	550 (22)
DI-4	91,250/yr	325 (13)	400 (16)	625 (25)
DI-5	328,500/yr	350 (14)	450 (18)	700 (28)
[4] DI-6	1,095,000/yr	400 (16)	475 (19)	750 (30)

Notes:
1. Assumes a life expectancy of 20 years.
2. ESAL's can be estimated using AASHTO LEFs.
3. Frost Groups described in Paragraph 3.1.3.3, this SECTION.
4. Higher Design Indices are available (up to DI-10), with a maximum combined thickness of 1.1 m (42 in.) for F3 and F4 Frost Groups.

There are additional requirements on the base course to meet all design requirements (drainage, etc.).

For additional, related information, refer to Paragraph 3.2.4 (this SECTION) and Paragraph 1.2.2.2 in SECTION 5.0.

3.1.4.5 Granular or Rock Cap

One fundamental way to reduce frost action in a pavement is to stop (or reduce) the available water from forming ice lenses or otherwise saturating the upper layers of the pavement structure. Tabor commented on this in 1930 [2.41]:

> "The troubles resulting from the formation of segregated ice under pavements can be entirely prevented if, in addition to the usual methods of draining, a thick layer of coarse material is introduced under the pavement extending down to the extreme depth of frost penetration."

This concept has been applied by the Idaho DOT as reported by Mathis [2.42]. He noted a number of features used in northern Idaho to reduce frost action. The primary element is to use a "rock cap" layer immediately on top of the prepared subgrade to

intercept the flow of water. A primary concern was to intercept water entering the pavement section through the surface course(s) as well as lower water sources. Mathis noted:

- The rock cap material is open-graded, with typically 100 percent passing the 75 mm sieve and 0 to 5 percent passing the 19.0 mm sieve. When placed on fine-grained subgrade soils, a geotextile is used as a separator (a filter layer could be used in lieu of a geotextile).

- Apparently, two separate design concepts have been used. One uses a thick rock cap layer with asphalt concrete layers applied directly to the rock cap. The other approach places a dense aggregate base on top of the rock cap. Asphalt concrete is then placed on the dense base to complete the pavement section. (Caution is warranted when placing a dense graded base material over an open graded material such as the rock cap. A downward movement of fines from the dense graded material can occur if not properly filtered.) Rock cap thicknesses (as reported by Mathis) have ranged from 814 mm (2.67 ft) (I-90 project) to 305 mm (1.0 ft).

- Material properties: Backcalculated material properties (layer moduli) for the rock cap layer range from 172 to 414 MPa (25,000 to 60,000 psi). Significantly, the Idaho DOT has <u>not</u> observed significant seasonal change in these moduli. Further, for structural design purposes, the rock cap material is substituted on a 1:1 basis for untreated base.

- Construction: The rock cap material should be 100 percent crushed material for constructibility purposes. Thick layers of such material are inherently unstable, requiring special construction techniques.

- The cost of the rock cap material was reported by Mathis as being about three times less expensive than aggregate base ($2.50/ton vs. $9.00/ton as reported in 1991). Performance data is limited since the earliest rock cap project was built a bit over 10 years ago (1991).

In 1973, Johnson [2.48] reported on a survey of North American DOT practices with respect to roadway design in seasonal frost areas. He noted that the State of Maryland used a 305 mm (12 in.) "granular cap" over frost susceptible subgrade soils. A CBR of 7 was assigned to such layers. Johnson also noted that Maryland has the option of stabilizing frost susceptible subgrades with cement. The states of Maine and Nebraska were reported as undercutting frost susceptible subgrades with the undercut material being replaced by granular fill.

3.1.4.6 Survey of Practice—1963

Several papers were presented at the 42nd Annual Meeting of the Highway Research Board (January 1963) which summarized a substantial amount of information on

pavement design for frost conditions. Practices of SHAs as well as Canadian provinces, Sweden, Switzerland, Germany, and Finland are contained in HRB Record No. 33 (specific papers which are cited are shown as References 2.52 through 2.60). Some of the information contained in these papers will be summarized by use of the following categories:

- Classification of Frost Susceptible Soils
- Thickness Design
- Depth to Water Table—Depth of Ditches
- Capillary Breaks
- Rate of Freeze
- General Comments

Thirty years later, the practices reported may still be valid or changed.

(a) Classification of Frost Susceptible Soils

Many SHAs classified materials as being frost susceptible if 10 percent or more passed a 0.075 mm sieve or 3 percent or more passed a 0.02 mm sieve (such as Vermont, Maryland, West Virginia, North Carolina, Oregon, and Washington). This is identical to the Casagrande criterion noted in Paragraph 3.1.3.3(b). A few states used higher percentages to designate frost susceptibility such as

SHA	Minimum Passing 0.075 mm Sieve
Delaware	35%
Utah	25%
Massachusetts	15%
Colorado	All soils susceptible

The Swiss Road Engineers used soil classification to address susceptibility.

Frost Classification	USCS	AASHTO[1]
A (No frost sensitivity)	GW, GP, SW, SP	A-1, A-2, A-3
B (Little frost sensitivity)	GM, GC	A-1b
C (Medium frost sensitivity)	SM, SC, CH, OH	A-2, A-4, A-6, A-7
D (Heavy reaction to frost)	ML, MH, CL, OL	A-4, A-5, A-7-5

[1]Formerly known as the Bureau of Public Roads classification in 1963.

In Canada, Alberta and Saskatchewan used the U.S. Corps of Engineers system (previously described in Paragraph 3.1.3.3). Newfoundland used the following percentages passing a 0.075 mm sieve:

Frost Susceptibility Classification	Percent Passing 0.075 mm Sieve
• Non-susceptible	0-6
• Moderately susceptible	6-12
• Susceptible	12-24

Québec used a variation of the above as follows:

Frost Susceptibility Classification	Percent Passing 0.075 mm Sieve	Percent Silt and Fine Sand[1]
• Non-susceptible	0-10	0-20
• Susceptible	10 -30	20-40
• Very Susceptible	> 30	> 40

[1]Presumably grain size limits for silt are 0.005 to 0.05 mm and fine sand 0.05 mm to 0.1 mm.

In general, it can be concluded from these state, provincial, and national practices that materials were considered potentially frost susceptible if more than 10 percent passed a 0.075 mm sieve.

(b) Thickness Design

The practices of a few agencies will be summarized:

Agency	Comments
New Hampshire	For primary and interstate pavements, the total depth of the pavement structure was based on the complete removal of frost susceptible soils within the zone of freezing. For lower road classifications, lesser requirements were used.
Delaware	The pavement structure was designed by a traditional method. If the total structural thickness was less than the frost penetration, the subbase thickness was increased to equal the maximum depth of frost penetration.
Colorado	The total pavement structural thickness was partially determined by the expected depth of frost penetration.

Agency	Comments
Oregon	For frost areas, the total depth of the pavement structure was equal to one-half the expected depth of frost.
Nevada	The total base and subbase thickness was increased 100 mm in the northern one-half of the state and frost areas.
Washington	The total depth of the pavement structure must equal one-half the expected depth of frost.
National Swedish Road Research Institute	The then proposed design table developed by the Institute contained minimum total pavement thicknesses (surfacing, base, subbase) as a function of subgrade frost susceptibility and traffic. For the least frost susceptible subgrade soils (mostly gravels), these thicknesses ranged from 150 mm (lightest traffic: < 50 heavy vehicles/day and < 500 ADT) to 350 mm (heaviest traffic: > 3000 heavy vehicles/day and > 30,000 ADT). For the most frost susceptible soils (silts, etc.), these thicknesses ranged from 500 mm for the "lightest" traffic category to 1000 mm for the heaviest truck category.
Canadian Federal Department of Public Works	A minimum depth of pavement structure (surface, base, subbase) must equal one-half the average depth of frost penetration (determined from a 10-year average Freezing Index (FI)). Typical thicknesses as a function of FI are shown below.

Average Freezing Index (°F-days)	Minimum Total Thickness of Flexible and Rigid Pavements (in.)
150	15
200	17
300	20
400	22
500	23
600	24
700	25
800	26
900	27
1000	27
1500	30
2000	32

Ontario	The total pavement structural thicknesses were generally about 50 to 75 percent of the depth of frost penetration.

In general, a number of highway agencies increased the total depth of the pavement structure to meet some percentage of the anticipated depth of freeze ranging from 50 to 100 percent.

(c) Depth of Water Table—Depth of Ditches

A number of highway agencies used techniques to enhance winter/spring drainage of water away from the pavement structure. Some of the responses provided in the HRB Record No. 33 follows:

Agency	Comments
New Hampshire	Bottom of ditches were at least 1.2 m (4 ft) below edge of shoulder.
New York	Attempts were made to keep the water table below the zone of freezing beneath the pavement.
New Jersey/ Vermont	When needed, subdrains were used to lower the water table.
Michigan	Grade was set at a minimum of 1.5 m (5 ft) above the water table.
Idaho	The ditch bottom was maintained at least 150 mm (0.5 ft) below the base course or any select granular material.
Washington	If the water table is within 1.5 m (5 ft) of the top of the subgrade and the soil is frost susceptible, then frost design was used.
Canada	A quote from the paper by Armstrong and Csathy [2.58]: "In several cases, pronounced heaving has been reported in pavements where the water table was more than 3 m (10 ft) below the surface." (The potential for capillary rise was previously discussed in Paragraph 3.1.3.1.) As noted by Armstrong and Csathy, all Canadian provinces (except two) required the subgrade surface to be at a specified minimum height above the water table. Often, side ditches are used to "control" the water table height. In some cases, a granular or clay capillary cutoff was used as an alternative to the minimum height requirement. Grades were generally set as a minimum about 1.2 m (4 ft) above the water table.
Québec	The bottom of the side ditches were set 0.5 m (18 in.) below the top of the subgrade.

Agency	Comments
Switzerland	Ground water conditions were considered "good" if the ground water depth was more than three times the maximum expected frost penetration.
Finland	Ditch depths were set at about 1.0 m to "dry" the base and subbase and reduce frost effects.

Though practices varied from agency to agency, in general, the top of the subgrade was maintained about 1.2 to 1.5 m (4 to 5 ft) above the water table. Ditch bottoms were set to keep water out of any granular layers which were part of the pavement structure. Further, the subgrade surface was often sloped to aid drainage of the base and subbase layers.

(d) Capillary Breaks

A few agencies reported in 1963 that they used capillary breaks to reduce the upward movement of water and/or enhance subsurface drainage. Such techniques are variations of those described in Paragraph 3.1.4.5. The then general practices of two counties and two Canadian provinces will be described below:

Agency	Comments
Sweden	It was reported that the use of sand layers to cut off capillary flow had been used on a "wide scale." One example shown by Rengmark [2.56] contained a sand layer 200 mm thick on top of a silt subgrade. It was noted that the sand layer should be placed <u>above</u> the bottoms of the adjacent ditches.
Finland	Use was made of sand (or gravel) layers to stop the capillary flow of water and were placed on the top of the subgrade. Such layers were generally 150 to 200 mm (6 to 8 in.) thick but could be up to 400 mm (16 in.) thick.
British Columbia	A 300 mm (12 in.) thick layer of clean sand was placed on subgrades as part of corrective measures to address frost heaving problems. It was unclear as to whether the sand layer was to enhance subsurface drainage, in general, and/or as a capillary break.
Québec	It was noted that a "cutoff blanket" of sand or gravel was used in areas with high water tables.

(e) Rate of Freeze

It was noted by Küber (2.60) that the rate of ground freezing had a strong influence on the eventual pavement distress which occurred on German roads during the spring thaw. Küber observed that when a pavement freezed rapidly, the movement of water, formation of ice lenses, etc. was diminished. To address this, he developed a criterion based on "temperature sum curves" to assess rate of freezing influences. In essence, Küber proposed the following:

Potential Frost Damage	Rate of Daily Sum of Below Freezing Temperatures
Severe (slow freeze)	≤ 3.5 °C/day
Moderate	≥ 3.5 °C/day ≤ 11.1 °C/day
Slight (rapid freeze)	≥ 11.1 °C/day

Whether the above criterion is broadly applicable is unclear but, nevertheless, is interesting. The implication is that a slow rate of winter freezing is potentially much more damaging than a rapid rate of freeze. (The freezing data shown for Mansfield, Washington in Appendix 2.3 reveals daily average below freezing temperatures of 8.8 °C for December 1985, 4.1 °C for January 1986, and 5.1 °C for February 1986. This meets Küber's "moderate" category for potential frost damage.)

(f) General Comments

First, a comment by Armstrong and Csathy [2.58] about conditions which contribute to severe spring thaw conditions. They noted:

> "The general experience is that the greatest losses are encountered when a wet fall is followed by a winter with many cycles of freezing and thawing without much snow, and then by a rapid thaw during the spring. If the thaw takes place before the ditches are clear of snow and ice, then conditions become even worse because dispersal of surface and melt waters is very slow. … In Saskatchewan it has been found that the reduction in load-carrying capacity during the spring period is dependent on the amount of rain during the two preceding years."

This suggests that the dedensification which can occur in unstabilized layers due to the volume increase which occurs when water freezes plays a role in the severity of the thaw period on the pavement structure.

LeClerc in his discussion of Erickson's paper [2.55] reinforced the view that moist conditions along with freeze/thaw cycles are of special concern. He noted:

> "… it is believed that frost contributes to roadway deterioration in another manner which is less obvious and which does not require the classic conditions …The sequence of events occurs in this matter:

"The rains in the fall season are somewhat continuous and contribute to an increase in the water content of the underlying base, subbase, and possibly subgrade. A prolonged cold spell moves in with freezing temperatures prevailing for approximately a week or ten days. During this period, particularly if there is snowfall, the thawing which might occur during the day does not penetrate to a depth sufficient to permit vertical drainage, and lateral drainage is hardly ever present, even in unfrozen shoulders. The alternate freezing and thawing tends to accumulate water under the roadway, and promotes an abnormally high water content in the surfacing [base and subbase] courses. Also, the freezing contributes to some expansion and consequent loss of density in these materials. The freezing temperatures need not be exceedingly severe, just 15 or so degrees [°F] below freezing, to bring about this condition.

"With the warming trend that follows, the accumulation of water plus the loss in density, however small, is manifested in increased amplitude of roadway surface deflection under load. Before the water can be dissipated and the density of the affected base and subbase courses regained, the fatigue life of the pavement surface has been seriously shortened or surpassed.

"The sequence of events has been noted often in Washington and no doubt occurs in other states."

LeClerc in his discussion described Washington's approach toward frost design. He noted:

"It [Washington approach] falls into the group which calls for a depth of frost-free cover equal to one-half of the maximum depth of frost penetration in the location concerned. This is used only where the three elements of frost potential are evident …

"The depths of maximum frost penetration were obtained by field measurements during an exceptionally cold winter of 1949-50. The measured depths and their location were spotted on a state map [shown as Figure 2.36 in this SECTION] and rough contour lines of equal frost penetration drawn through these points. The maximum frost penetration depth used in the design is taken from this map."

LeClerc also noted:

"Frost susceptibility is judged by the amount passing the No. 200 [0.075 mm] sieve—any soil having more than 10 percent passing this size sieve is considered frost susceptible."

3.1.4.7 Other Design Considerations

The information contained in the following Paragraph 3.2 (ESTIMATION OF FREEZING OR THAWING DEPTHS IN PAVEMENTS) is, in part, directed toward how to estimate the depth of freeze. Such estimates have been used as a fundamental consideration in designing the pavement section to resist frost action as were described above. Such a criterion is helpful in designing for frost conditions, but oversimplifies the complex conditions which can accompany various pavement materials, depths of freeze, and water sources.

Pavement designs for frost areas of Washington State should consider past pavement performance in the vicinity of the project in developing the pavement section. Further, the designer should consider items such as:

- The need for a capillary break such as the rock cap layer used by the Idaho DOT.
- The gradation of all materials used in the pavement section which relates to frost susceptibility (recall "high" percent fines passing the 0.075 mm sieve can make a material (even crushed stone) frost susceptible).
- The anticipated seasonal changes in unstabilized materials (stiffness and/or strength).
- The need for positive subsurface drainage. Recall that the majority of WSDOT specified materials are not free-draining (as discussed earlier in this SECTION).
- The depth to saturated layers or the water table must be considered.
- As stated previously, the anticipated depth of freeze must be considered (refer to Paragraph 3.2.4 in this SECTION and Paragraph 1.2.2.2 in SECTION 5.0 for more information).
- Removal of highly frost-susceptible materials down to the expected depth of frost.
- Modify high frost-susceptible materials by adding granular material.
- Various combinations of treatments can be considered.

3.2 ESTIMATION OF FREEZING OR THAWING DEPTHS IN PAVEMENTS

This paragraph will provide sufficient information to estimate the depth of freeze beneath pavements. First, needed terminology will be presented. All units will be in U.S. customary due to the source material.

3.2.1 TERMINOLOGY

3.2.1.1 Basic Characterization of Soil

(a) Density (γ_d or γ_{dry})

Typical densities for commonly encountered dry soils. Ice rich soils can have substantially smaller dry densities.

Soil	γ_d (lb/ft^3)
Gravel and sand	120-140
Silts and clays	90-100
Peat	≅ 20

Normally, dry densities are used in most calculations (γ_d); however, the total density of a soil (including moisture) can be calculated as follows:

$$\gamma_{total} = \gamma_d \left(1 + \frac{w}{100}\right)$$

where w = moisture content of soil expressed as a percentage

if a gravel has γ_d = 130 lb/ft^3 and w = 5%,

then γ_t = 130(1 + 5/100)

= 136.5 lb/ft^3

(b) Moisture Content (w)

Soil moisture content can be calculated as follows:

$$w = \frac{\text{wt. of water in soil}}{\text{wt. of soil solids}} (100)$$

$$= \frac{\text{(wt. of original sample) - (wt. dried sample)}}{\text{(wt. of dried sample)}} (100)$$

Typical Moisture Contents of Non-Ice Rich Soils

Material	w(%)
Gravel	2-10
Sand	5-15
Silts	5-40
Clays	10-50 (or more)
Organic (peat)	> 50

3.2.1.2 Thermal Conductivity (k)

Rate of heat flow through a unit area under a thermal gradient (recall that 1 BTU is the energy (heat) required to raise the temperature of 1 lb of water 1 °F):

Units: BTU/hr • ft^2 • °F/ft or BTU/hr • ft • °F

Further, $k_{snow\ (loose)} = 0.06$, $k_{snow\ (compact)} = 0.20$.

In the range of water contents (5 to 10%) and dry densities (125-135 lb/ft^3) commonly encountered in embankments and pavement base courses, thermal conductivity is very sensitive to moisture content and soil type. Soil thermal conductivities can be obtained from Figures 2.28 through 2.30 [from Ref. 2.17 and 2.18]:

- Figure 2.28a is for <u>silt and clay</u> soils in a <u>frozen</u> condition,
- Figure 2.28b is for <u>silt and clay</u> soils in an <u>unfrozen</u> condition,
- Figure 2.29a is for <u>granular</u> soils in a <u>frozen</u> condition,
- Figure 2.29b is for <u>granular</u> soils in an <u>unfrozen</u> condition,
- Figure 2.30a is for <u>peat</u> in a <u>frozen</u> condition, and
- Figure 2.30b is for <u>peat</u> in an <u>unfrozen</u> condition.

For example, using some of the above figures:

Base Course (granular)

$$\left.\begin{array}{l} \gamma_d = 135\ lb/ft^3 \\ w = 5\% \end{array}\right\} \begin{array}{l} k_{frozen} = 1.8\ BTU/hr \cdot ft^2 \cdot °F \\ k_{unfrozen} = 1.65\ BTU/hr \cdot ft^2 \cdot °F \end{array}$$

Subgrade (silt)

$$\left.\begin{array}{l} \gamma_d = 100\ lb/ft^3 \\ w = 15\% \end{array}\right\} \begin{array}{l} k_{frozen} = 0.8\ BTU/hr \cdot ft^2 \cdot °F \\ k_{unfrozen} = 0.72\ BTU/hr \cdot ft^2 \cdot °F \end{array}$$

NOTE
Thermal conductivity, k, is expressed in BTU per hour per sq. ft.
per unit thermal gradient (°F/ft.)

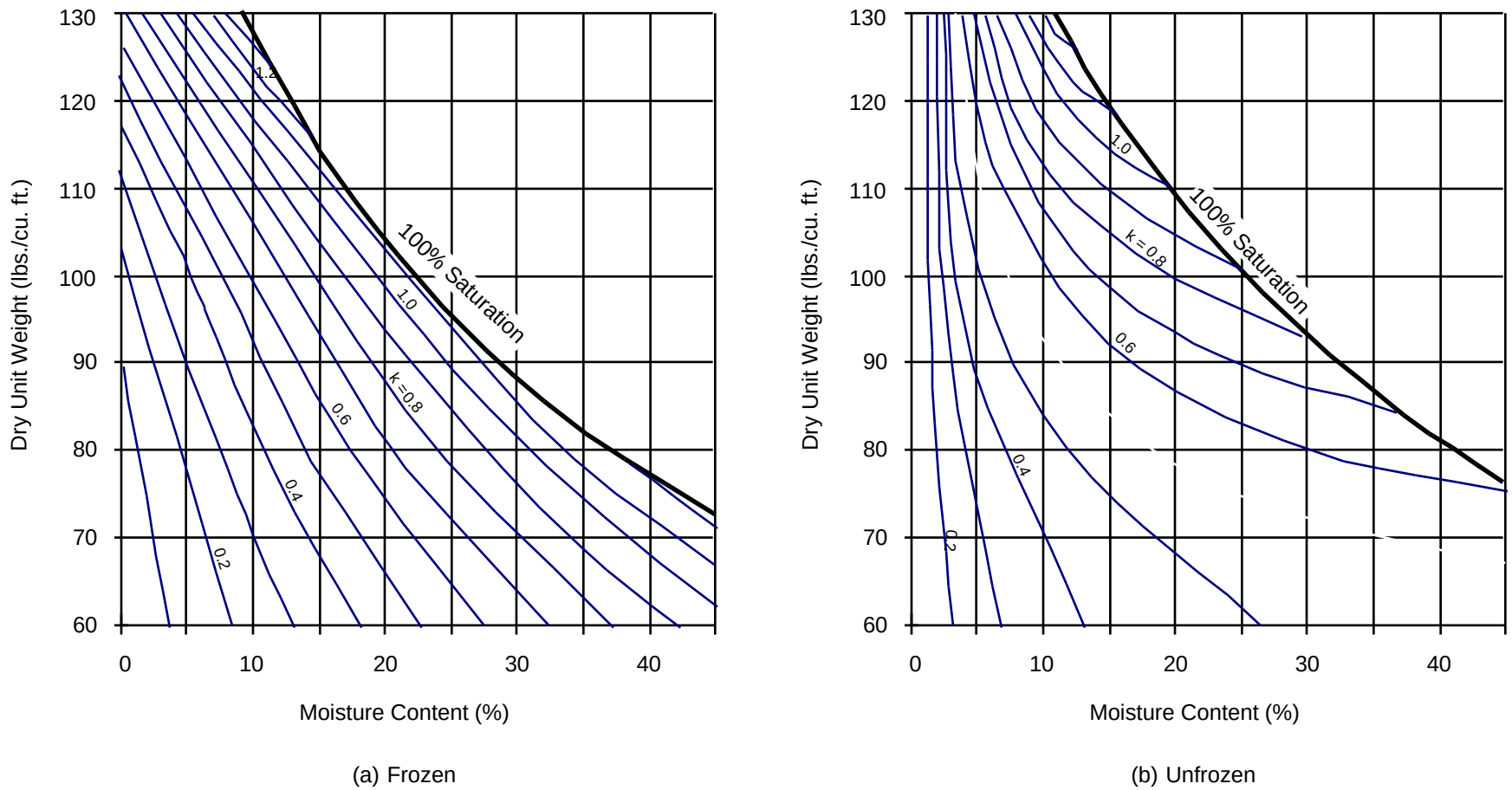
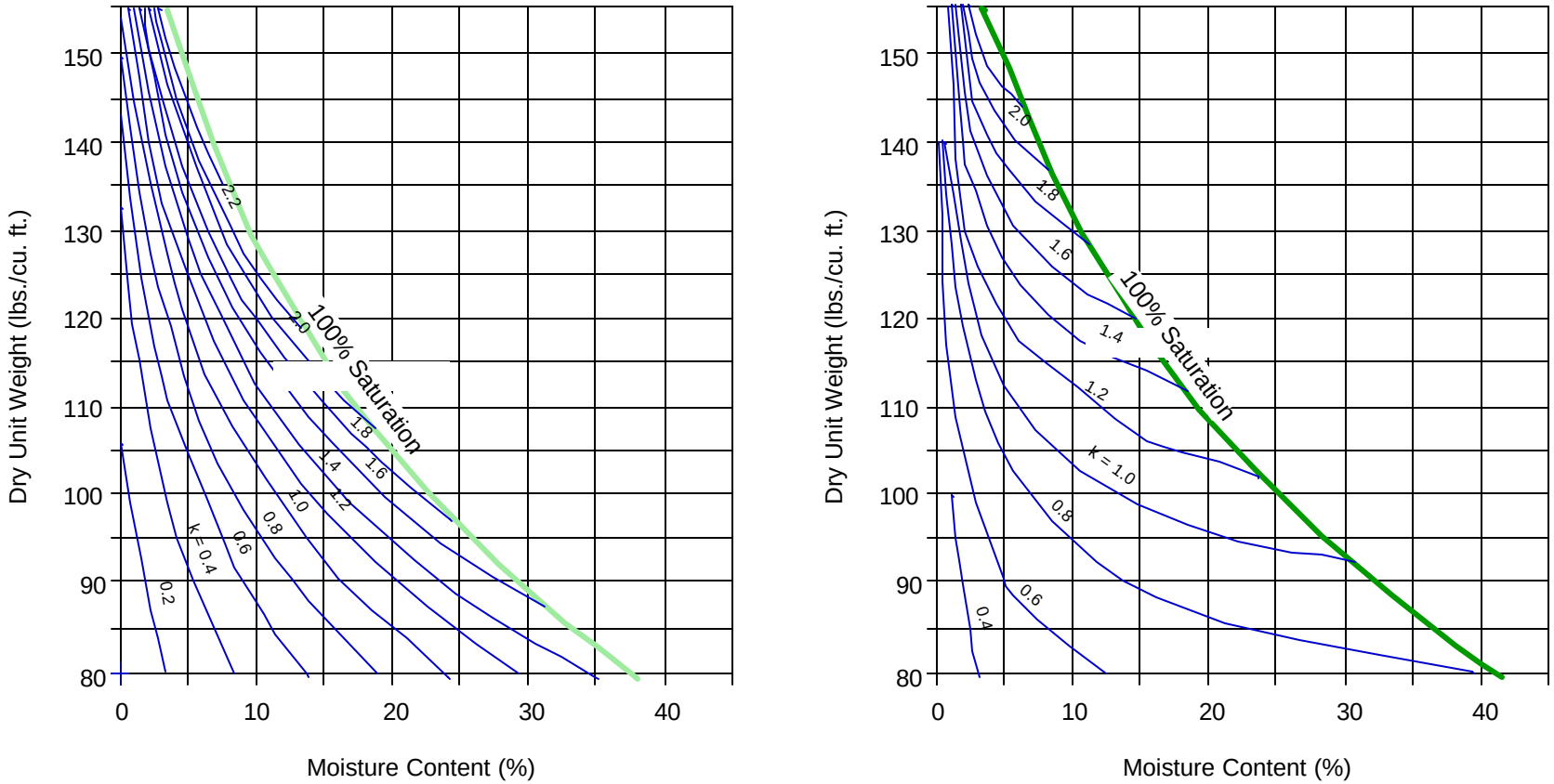


Figure 2.28. Average Thermal Conductivity for Silt and Clay Soils, Frozen and Unfrozen [Redrawn from 2.17, 2.18]

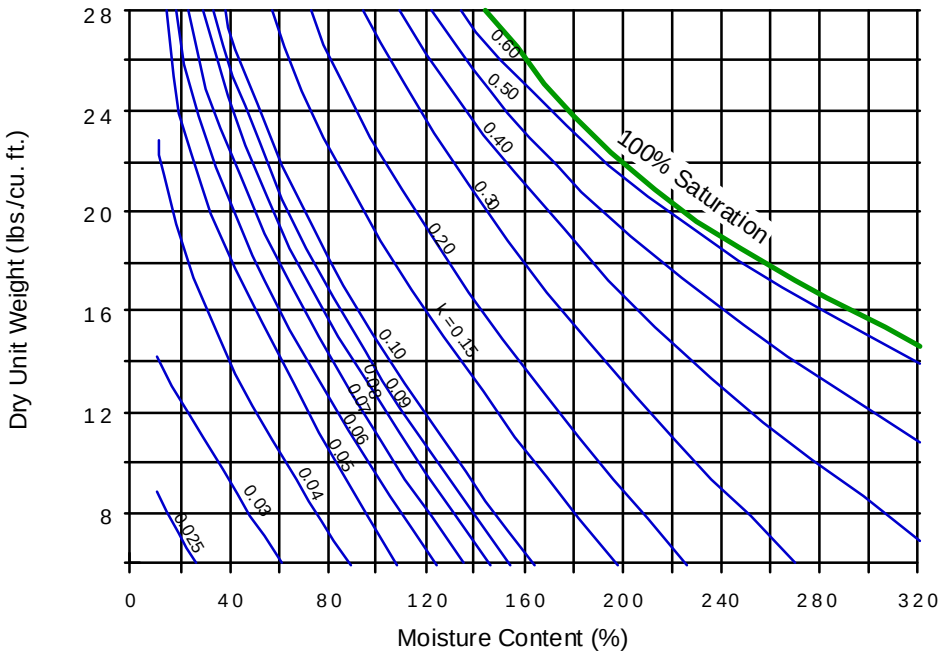
NOTE
Thermal conductivity, k, is expressed in BTU per hour per sq. ft. per unit thermal gradient (°F/ft.)



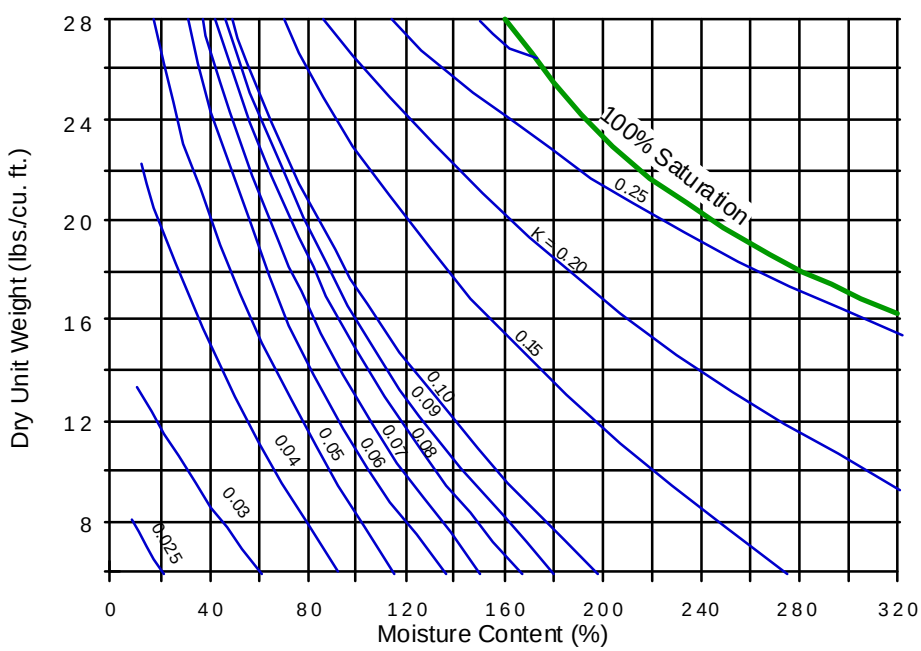
(a) **Frozen** (b) **Unfrozen**

Figure 2.29. Average Thermal Conductivity for Granular Soils, Frozen and Unfrozen [Redrawn from 2.17, 2.18]

NOTE
Thermal conductivity, k, is expressed in BTU per hour per sq. ft. per unit thermal gradient (°F/ft.)



(a) Frozen



(b) Unfrozen

Figure 2.30. Average Thermal Conductivity for Peat, Frozen and Unfrozen [Redrawn from 2.17, 2.18]

The data for figures 2.28 through 2.30 were developed by Kersten [2.18]. The equations used to develop Figures 2.28 and 2.29 follow. There are separate equations for frozen (tests conducted at -4 °C) and unfrozen (tests conducted at +4 °C) conditions.

- Unfrozen silt–clay soils

$$k = [\,(0.9 \log w - 0.2)\,(10)^{0.01\gamma_d}]\;(0.0833)$$

- Frozen silt–clay soils

$$k = (0.0833)\,[\,0.01\,(10)^{0.022\gamma_d} + 0.085\,(10)^{0.008\gamma_d}\,(w)]$$

- Unfrozen granular soils (sands and gravels)

$$k = [\,(0.7 \log w + 0.4)\,(10)^{0.01\gamma_d}]\;(0.0833)$$

- Frozen granular soils (sands and gravels)

$$k = (0.0833)\,[\,0.076\,(10)^{0.013\gamma_d} + 0.032\,(10)^{0.0146\gamma_d}(w)]$$

where k = thermal conductivity (BTU/hr • ft^2 • °F/ft)

w = soil moisture content (%), and

γ_d = soil dry density (lb/ft^3)

The equations for silt–clay were based on five soils and are valid for moisture contents of seven percent or higher. The equations for granular soils were based on four soils (two sands, a sandy loam, and a gravel) and are valid for moisture contents of one percent or higher. It was noted by Farouki [2.47] that Kersten's equations do not apply to dry soils or to crushed rocks. Use of the above equations to estimate k is more accurate than use of Figures 2.28 and 2.29.

3.2.1.3 Volumetric Specific Heat (C)

Expresses the change in thermal energy in a unit volume of soil per unit change in temperature.

Units: BTU/ft^3 • °F

Volumetric heat derived from specific heat. (Recall: specific heat is the change in thermal energy per unit weight per unit change in temperature. If objects of the same weight but of different materials receive the same amount of energy (heat), they will come to equilibrium at different temperatures. Or, stated another way, how much each

object's temperature changes depends on the specific heat of the material, if the mass and energy inputs are identical.)

(a) Typical values of specific heat (c)

c_{water} = 1.0 BTU/lb • °F

c_{ice} = 0.5 BTU/lb • °F

c_{rock} = 0.17 BTU/lb • °F (soil minerals)

(b) Volumetric specific heat relationship (C)

- Unfrozen soil: $C_u = \gamma_d \left(0.17 + \frac{w}{100}\right)$
- Frozen soil: $C_f = \gamma_d \left(0.17 + \frac{0.5w}{100}\right)$

(c) Example

Calculated values for a gravel with γ_d = 130 lb/ft and w = 5%:

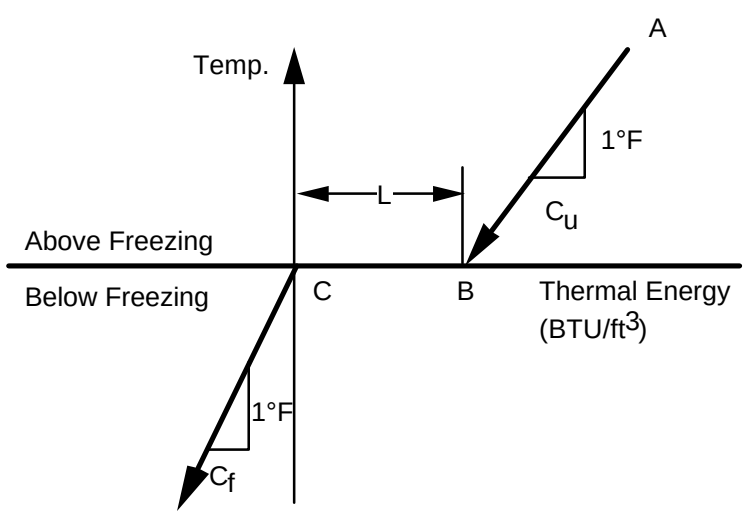
C_u = 130 (0.17 + 5/100) = 28.6 BTU/ft^3 • °F

C_f = 130 (0.17 + (0.5) (5/100)) = 25.4 BTU/ft^3 • °F

C_{avg} = 130 (0.17 + (0.75) (5/100)) = 27.0 BTU/ft^3 • °F

3.2.1.4 Latent Heat (L)

All objects have energy (heat). A portion of this thermal energy (stored heat) is released when the object cools. The sketch below represents a volume of soil with some moisture as it freezes:



As water freezes, thermal energy equal to L is released while the temperature of the soil remains nearly constant. Thus, the latent heat is the energy required to transform 1 lb of

a pure substance from one phase to another at constant temperature. Further, 1 lb of water gives off 144 BTU as it freezes. The latent heat of a soil can be represented by:

$$L = 144(w)\ (\gamma_d)/100$$

An embankment material with w = 5% and γ_d = 140 lb/ft^3,

$$L = \left(144\ \frac{BTU}{lb}\right)(5)\ (140\ lb/ft^3)\ /100 \cong 1000\ BTU/ft^3$$

3.2.1.5 Freezing and Thawing Indexes

Depth of freezing and thawing depends in part on the magnitude and duration of the temperature differential below or above freezing (32 °F) at the ground surface.

Freezing or thawing index is therefore given by the summation of the degree-days for a freezing or thawing season.

(a) Calculations for Freezing Index (FI)

$$\Sigma\ (\bar{T}\ - 32\ °F)$$

where $\bar{T}$ = mean daily temperature

$\bar{T}$ = 1/2 (T_1 + T_2) and T_1 = maximum daily air temperature
T_2 = minimum daily air temperature

Example Calculations

Day	Maximum	Minimum	Average	Degree Days per Day	Cumulative Degree Days
1	29	1	15	-17	-17
2	9	-11	-1	-33	-50
3	10	-8	1	-31	-81
4	15	-1	7	-25	-106
5	30	16	23	-9	-115
6	38	30	34	+2	-113
7	30	18	24	-8	-121

Note: Assume Day 1 start of freezing season. The negative sign in this case indicates freezing degree-days (normally omitted).

(b) Calculation of Thawing Index (TI)

$$\Sigma (\bar{T} - 32\ °F)$$

where $\bar{T}$ = mean daily temperature

Note: For the purpose of assessing spring load restrictions, use 29 °F in lieu of 32 °F. This accounts for the "dark" bituminous surface.

(c) Air and Surface Indexes

Normally, data are only available for air freezing and thawing indexes (≅ 1 meter in air above ground). Need to establish potential heat flow at the air-ground interface (i.e., need temperatures at ground surface).

No simple correlation exists between air and surface indexes. Difference between air and surface temperatures influenced by:

- latitude
- cloud cover
- time of year
- wind speed
- surface characteristics
- subsurface thermal properties
- surface slope and orientation

However, generally use "n-factor" for purposes of correlation.

(i) n-Factor for Freezing Conditions

"n" increases with increases in latitude and wind speed. Snow covered surfaces reflect large portion of incoming solar radiation with resulting larger surface freezing index.

$$n = \frac{\text{surface freezing index}}{\text{air freezing index}}$$

Typical Values

Surface Type	"n"
Snow	1.0
Pavements free of snow and ice	0.9
Sand and gravel	0.9
Turf	0.5

(ii) n-Factor for Thawing Conditions

"n" decreases with increases in latitude and wind speed

$$n = \frac{\text{surface thawing index}}{\text{air thawing index}}$$

"n" = 1.0 for turf
"n" = 2.0 for sand and gravel surfaces

(d) Design Freezing and Thawing Indexes

For design purposes, generally use freezing (or thawing) index based on three coldest winters (or warmest summers) in last 30 years of record. If not available, use air-freezing index for the coldest winter in last 10 years.

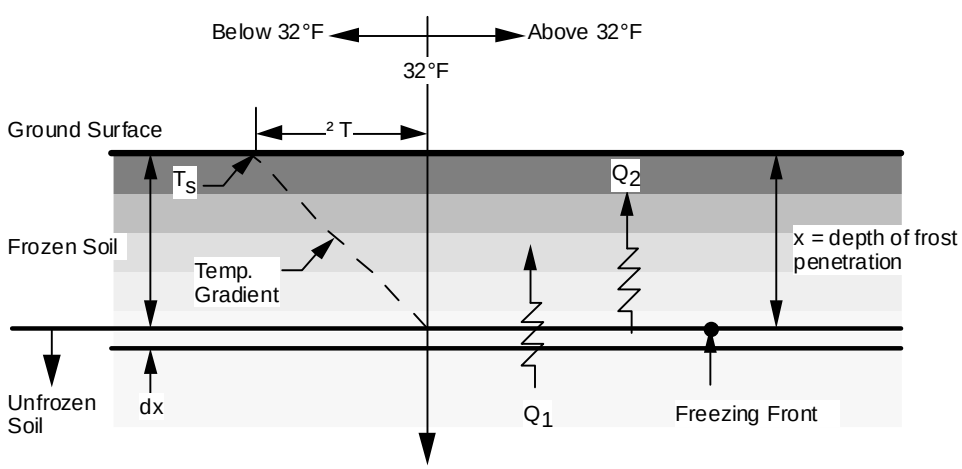
3.2.2 FROST (OR THAW) PENETRATION FORMULA

3.2.2.1 General

(a) Stefan formula
(b) Modified Berggren formula

3.2.2.2 Stefan Formula

In this formula it is assumed that the latent heat of soil moisture is the only heat which must be removed when freezing the soil. Thus, thermal energy stored as volumetric heat and released as soil-temperatures drop to and below freezing not considered. This assumption is equivalent to shifting the sloping lines in the figure on p. 2-69 is to vertical. The latent heat supplied by the soil moisture as it freezes a depth dx in time dt = rate at which heat is conducted to the ground surface. This is illustrated in the sketch below:



Heat removal process can be represented by

$$Q_1 = L\frac{dx}{dt} \rightarrow \text{heat released by freezing a layer of soil dx thick in time dt}$$

$$Q_2 = \frac{\Delta T}{R} = \frac{\Delta T}{(x/k_f)} \rightarrow \text{heat conducted through frozen layer}$$

and $Q_1 = Q_2$ so $L\frac{dx}{dt} = k_f\frac{\Delta T}{x}$

by integrating and solving for x,

$$x = \sqrt{\frac{2k_f}{L}\int \Delta T\, dt} \qquad \text{(Eq. 2.1)}$$

where $\int \Delta T\, dt$ is in units of °F • hr and is called surfacing freezing index.

Freezing index normally expressed as °F • days. Thus, rewrite Equation 2.1 and add "n" factor which results in the Stefan formula:

$$x = \sqrt{\frac{48k_f\, n\, FI}{L}} \qquad \text{(Eq. 2.2)}$$

(a) Formula

Assumes that the soil is a semi-infinite mass with uniform properties and existing initially at a uniform temperature (T_i). Further assumed that surface temperature is suddenly changed from T_i to T_s (below freezing). Modified Berggren formula:

3.2.2.3 Modified Berggren Formula

$$x = \lambda\sqrt{\frac{48k_{avg}\, n\, FI}{L}} \quad \text{or} \quad x = \lambda\sqrt{\frac{48k_{avg}\, n\, TI}{L}}$$

where x = depth of freeze or thaw, (ft)

λ = dimensionless coefficient which takes into consideration the effect of temperature changes in the soil mass (i.e., a fudge factor). Corrects the Stefan formula for the neglected effects of volumetric heats (accounts for "sensible heat" changes).

k = thermal conductivity of soil, average of frozen and unfrozen, (BTU/hr • ft • °F)

n = conversion factor for air freezing (or thawing) index to surface freezing (or thawing) index

FI = air freezing index (°F • days)

TI = air thawing index (°F • days)

(b) Calculation of λ

L = latent heat (BTU/ft^3)

λ = f (FI (or TI), mean annual air or ground temperature, thermal properties of soil)

λ = f(α,μ)

where α = thermal ratio

μ = fusion parameter

To obtain "λ," use charts which follow.

(ii) Calculation of "α"

$$\alpha = \left| \frac{\bar{T} - T_f}{T_f - T_s} \right|$$

where $\bar{T}$ = average annual air or ground temperature (refer to Table 1, SECTION 7.0, Appendix 7.1 for typical average annual air temperatures for various regions of Washington State)

T_f = 32 °F

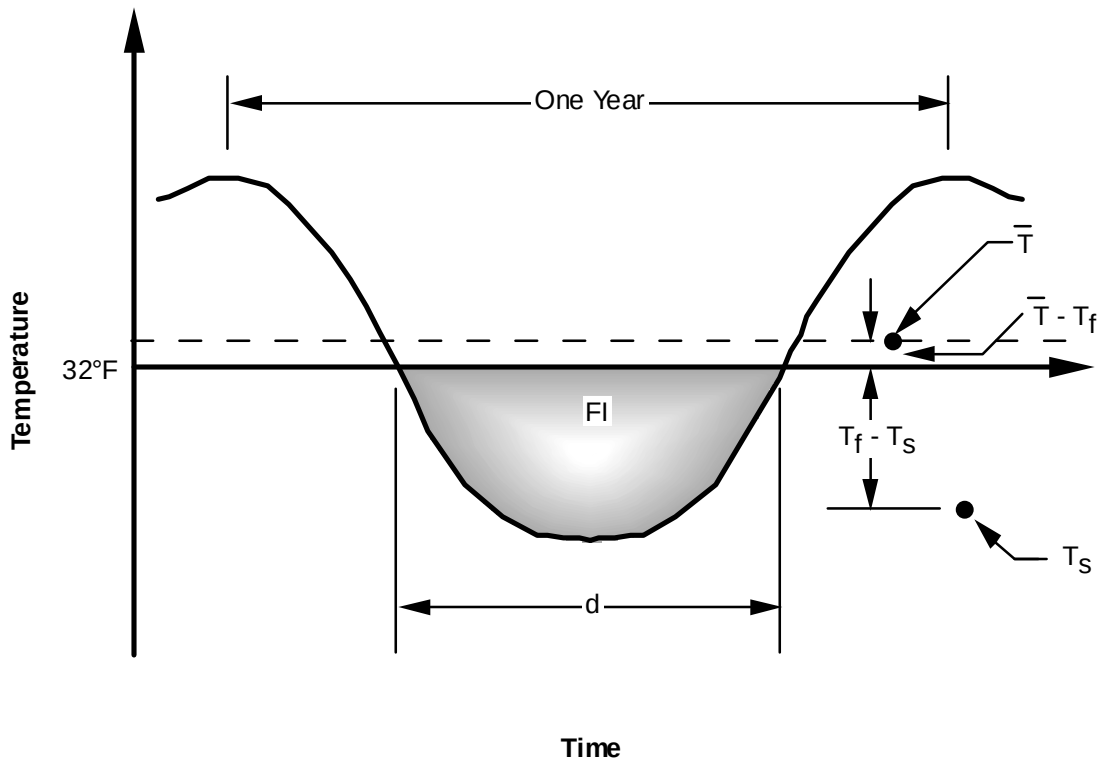
T_s = average surface temperature for the freezing (or thawing) period

and $| T - T_s |$ = surface freezing (or thawing) index, nFI (or nTI) divided by length of freezing (or thawing) season. Represents temperature differential between average surface temperature and 32 °F taken over the entire freeze (or thaw) season.

$$= \frac{n\,FI}{d} \quad \text{or} \quad \frac{n\,TI}{d}$$

and $|\bar{T} - T_f|$ represents the amount that the mean annual temperature exceeds (or is less than) the freezing point of the soil moisture (assumed to be 32 °F).

Example: Freezing Period



recall d = duration of freezing period

(iii) Calculation of "μ"

$$\mu = (\,|T_f - T_s|\,)\left(\frac{C}{L}\right)$$

where $|T_f - T_s| = \frac{n\,FI}{d}$

C = average volumetric heat capacity of a soil (BTU/ft^3 • °F)

L = latent heat (BTU/ft^3)

(iv) Calculation of "d"

Length of freezing or thawing duration. For example, if the winter freezing season is December through February, then the duration of freezing (d) equals about 90 days.

(v) Chart for "λ"

Use Figure 2.31 after "α" and "μ" have been calculated to obtain "λ".

3.2.2.4 Example 1 — Modified Berggren Formula

Determine the depth of frost penetration into a homogeneous sandy silt in the vicinity of Waterville, WA for the following conditions:

(a) Given

(i) Mean annual temperature = $\overline{T}$ = 48 °F (from Table 1, SECTION 7.0, Appendix 7.1)

(ii) Surface freezing index = nFI = 750 °F • days

(iii) Duration of freezing season = d = 100 days

(iv) Soil properties

- Dry density = γ_d = 100 lb/ft^3
- Water content = w = 15%

(b) Solution

(i) Calculate soil thermal properties

- Volumetric latent heat of fusion

  L = (144 BTU/lb) (100 lb/ft^3) (15/100) = 2160 BTU/ft^3

- Average volumetric specific heat

  C_{avg} = 100 (lb/ft^3) [0.17 + (0.75) (0.15)] = 28.2 BTU/ft^3 • °F

- Average thermal conductivity

  $k_f \cong$ 0.80 BTU/hr • ft • °F (refer to Figure 2.29a)
  $k_u \cong$ 0.72 BTU/hr • ft • °F (refer to Figure 2.29b)
  ∴ k_{avg} = 0.76 BTU/hr • ft • °F

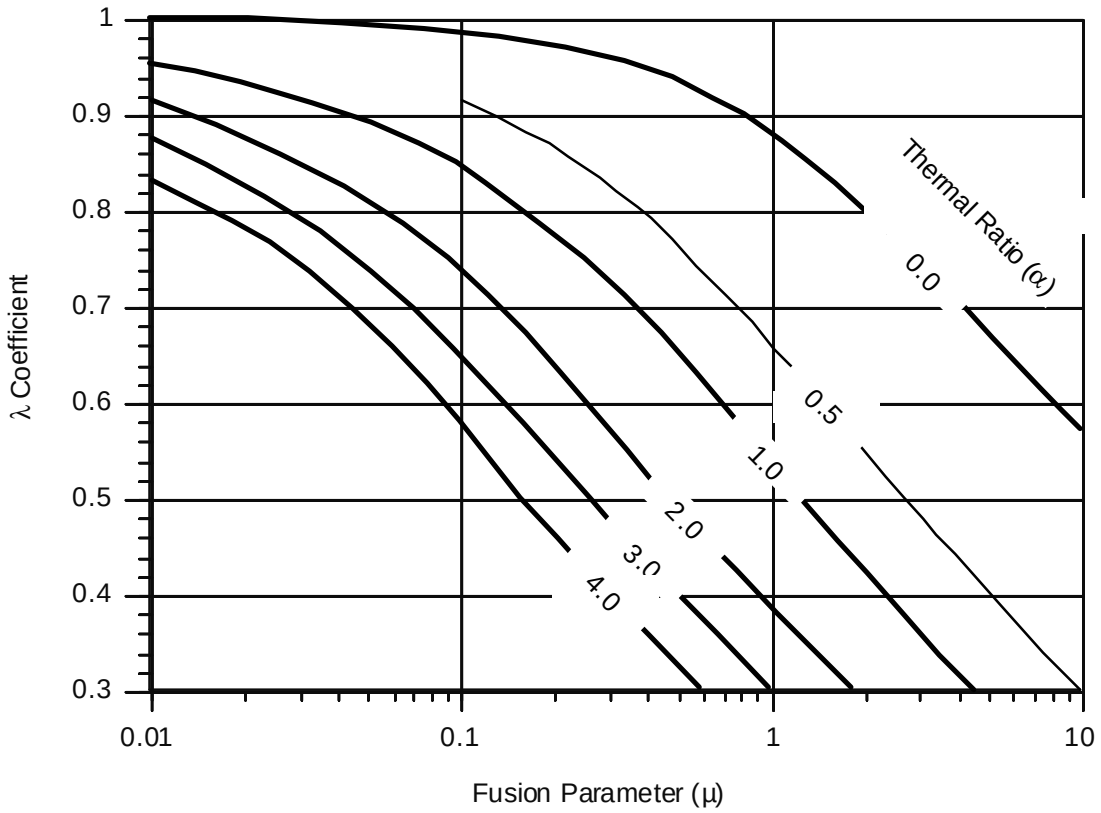


Figure 2.31. λ Coefficient in the Modified Berggren Formula

(ii) Calculate λ: recall $\lambda = f(\alpha, \mu)$

(i) First $\alpha = \dfrac{|\bar{T} - T_f|}{|T_f - T_s|} = \dfrac{|\bar{T} - T_f|\,(d)}{nFI}$

$$= \frac{|48 - 32|\,(100)}{750} = 2.13$$

(ii) Second $\mu = (|T_f - T_s|)\left(\dfrac{C_{avg}}{L}\right) = \dfrac{nFI}{d}\left(\dfrac{C_{avg}}{L}\right)$

$$= \left(\frac{750}{100}\right)\left(\frac{28.2}{2160}\right) = 0.10$$

(iii) Third, determine λ from Figure 2.31

$\lambda = 0.74$

(iii) Calculate depth of freezing

$$x = \lambda\sqrt{\frac{48k_{avg}\ nFI}{L}} = (0.74)\sqrt{\frac{48\,(0.76)\,(750)}{2160}}$$

$$= (0.74)(3.6) = 2.7 \text{ ft (or 32 in.) (or 0.8 m)}$$

3.2.2.5 Multilayered Modified Berggren formula

(a) General

- Developed by Aldrich (Highway Research Bulletin No. 135, 1953 — Reference 2.19)
- Uses empirical approach to model "effective" thermal properties for multilayer system
- Approximate computation technique
- One-dimensional technique. Does not accommodate edge effects
- Accounts for average thermal properties (frozen and unfrozen)
- Required data:
  - Thickness of layers

- Dry density and moisture content of each layer
- Surface freezing or thawing index
- Duration of freezing or thawing period and the mean annual air temperature
- Determine k, C, L for each layer in the depth to be studied. Thermal conductivity (k) and volumetric specific heat (C) is the average for frozen and unfrozen conditions.

(b) Solution

As stated above, determine the thermal properties k, C, L for each layer within the estimated depth of freezing (or thawing).

$$k_{avg} = \frac{k_f + k_u}{2}$$

$$C_{avg} = \frac{C_f + C_u}{2}$$

$$L = 144\ (w)\ (\gamma_d)/100$$

where k_f = thermal conductivity, frozen, (BTU/hr • ft • °F)
k_u = thermal conductivity, unfrozen, (BTU/hr • ft • °F)
C_f = volumetric specific heat, frozen, (BTU/ft^3 • °F)
C_u = volumetric specific heat, unfrozen, (BTU/ft^3 • °F)
L = latent heat, (BTU/ft^3)

- To compute x (depth of freeze or thaw), compute from:

$$x = \lambda \sqrt{\frac{48\ nFI}{(L/k)_{eff}}} \quad \text{or} \quad \lambda \sqrt{\frac{48\ nTI}{(L/k)_{eff}}}$$

- Compute L/k "effective" from the following equation:

$$(L/k)_{eff} = \frac{2}{x^2}\left[\frac{z_1}{k_1}\left(\frac{L_1 z_1}{2} + L_2 z_2 + \ldots + L_i z_i\right)\right.$$

$$+ \frac{z_2}{k_2}\left(\frac{L_2 z_2}{2} + L_3 z_3 + \ldots + L_i z_i\right)$$

$$+ \ldots + \frac{z_i}{k_i}\left(\frac{L_i z_i}{2}\right)\Bigg]$$

where x = estimated depth of freezing (or thawing), (ft)
= $z_1 + z_2 + \ldots + z_i$ (i.e., depths of the layers within x)
z_1 = depth of top layer, (ft)
z_2 = depth of second layer, (ft)
•
•
•
z_i = depth ith layer (ft)

- Compute the weighted values of C and L within the estimated depth of freezing (or thawing):

$$C_{wt} = \frac{C_1 z_1 + C_2 z_2 + \ldots + C_i z_i}{x}$$

$$L_{wt} = \frac{L_1 z_1 + L_2 z_2 + \ldots + L_i z_i}{x}$$

where C_1 = volumetric specific heat for top layer, average of frozen and unfrozen, (BTU/ft^3 • °F)
•
•
•
C_i = volumetric specific heat for lowest layer within the depth = x, average of frozen and unfrozen, (BTU/ft^3 • °F)

L_1 = latent heat for top layer, (BTU/ft^3)
•
•
•
L_i = latent heat for lowest layer within the depth = x, (BTU/ft^3)

z = depth as previously defined

- Compute the "effective" values of α and μ from:

$$\alpha_f = \frac{|\overline{T} - T_f|}{|T_f - T_s|} \quad \text{or} \quad \frac{|\overline{T} - T_f|\, d_f}{nFI} \qquad \text{(freezing case)}$$

$$\alpha_t = \frac{|\bar{T} - T_f|}{|T_f - T_s|} \text{ or } \frac{|\bar{T} - T_f|\, d_t}{nTI} \qquad \text{(thawing case)}$$

$$\mu = (T_f - T_s)\frac{C_{wt}}{L_{wt}} = \frac{nFI}{d_f}\left(\frac{C_{wt}}{L_{wt}}\right) \text{(freezing case)}$$

$$= \frac{nTI}{d_t}\left(\frac{C_{wt}}{L_{wt}}\right) \qquad \text{(thawing case)}$$

where $\bar{T}$ = mean annual air temperature,

T_f = 32 °F,

T_s = average surface temperature during the freezing (or thawing) season (°F),

d_f = duration of <u>freezing</u> season, (days)

d_t = duration of <u>thawing</u> season, (days)

C_{wt} = as defined previously,

L_{wt} = as defined previously,

nFI = <u>surface</u> freezing index, (°F • days)

nTI = <u>surface</u> thawing index, (°F • days)

- Determine λ (the correction coefficient) by using α and μ and Figure 2.31.
- Compute the depth of freezing (or thawing) from:

$$x = \lambda\sqrt{\frac{48\ nFI}{(L/k)_{eff}}} \text{ or } \lambda\sqrt{\frac{48\ nTI}{(L/k)_{eff}}}$$

- If the <u>computed</u> x differs significantly from the <u>assumed</u> x, <u>repeat</u> the computations.

3.2.2.6 Example 2 — Use of Multilayered Modified Berggren Formula

(a) Requirement

Calculate the estimated depth of freezing for SR 172, MP 2.0 (near Waterville, WA) on February 6, 1989. (This date was selected since a <u>measured</u> depth of freeze was available.)

(b) Known Conditions

(i) BST's ≅ 0.75 in. ≈ 0.06 ft
Base Course (CSTC) ≈ 9.6 in. ≈ 0.8 ft
Subgrade is a sandy silt (ML).

(ii) Freezing Index (FI)

The air FI was measured at the "nearby" WSDOT maintenance facility with a high-low recording thermometer. The FI = 717 °F - days on February 6, 1989.

(c) Solution

(i) Basic layer properties

Layer	(5) k_{avg} (BTU/hr • ft • °F)	(4) C_{avg} (BTU/ft^3 • °F)	L (BTU/ft^3)
BST	1.0	23.8	0(1)
Base	1.65	28.0	972(2)
Subgrade	0.75	28.2	2160(3)

Notes:

(1) Assumes moisture content in BST = 0%

(2) Assume base course with γ_d = 135 lb/ft^3, w = 5%
L = (144)(135)(5/100) = 972 BTU/ft^3

(3) Assume subgrade with γ_d = 100 lb/ft^3, w = 15%
L = (144)(100)(15/100) = 2160 BTU/ft^3

(4) $C_{avg} = \gamma_d \left(0.17 + 0.75\left(\frac{w}{100}\right)\right)$

BST: C_{avg} = 140(0.17) = 28.3 BTU/ft^3 • °F
Base: C_{avg} = 135(0.17 + 0.75(5/100)) = 28.0 BTU/ft^3 • °F
Subgrade: C_{avg} = 100(0.17 + 0.75(15/100)) = 28.2 BTU/ft^3 • °F

(5) Thermal Coductivities (k_{avg})

BST: k_{avg} = 1.0 BTU/hr • ft • °F
Base: k_{avg} = 1.65 BTU/hr • ft • °F — Figure 2.30
Subgrade: k_{avg} = 0.75 BTU/hr • ft • °F — Figure 2.29

(ii) Climate information

- Freezing Index (air) = 717 °F - days
- n-factor (no snow) ≈ 0.9
- $\underline{d}$ = 104 days (estimated)
- $\overline{T}$ ≈ 48 °F (Waterville, Washington — NOAA data — SECTION 7.0, Appendix 7.1)

(d) Calculations

(i) Assume x = 30 in. ≈ 2.5 ft (first iteration)

$$(L/k)_{eff} = \frac{2}{x^2}\left[\frac{z_1}{k_1}\left(\frac{L_1 z_1}{2} + L_2 z_2 + L_3 z_3\right) + \frac{z_2}{k_2}\left(\frac{L_2 z_2}{2} + L_3 z_3\right) + \frac{z_3}{k_3}\left(\frac{L_3 z_3}{2}\right)\right]$$

$$= \frac{2}{(2.5)^2}\left[\frac{0.06}{1.0}\left(\frac{(0)(.06')}{2} + (972)(0.8') + (2160)(1.64')\right) + \frac{0.8'}{1.65}\left(\frac{(972)(0.8')}{2} + (2160)(1.64')\right) + \frac{1.64'}{0.75}\left(\frac{(2160)(1.64')}{2}\right)\right]$$

$$= 0.32\ [259 + 1906 + 3873]$$

$$\cong 1932$$

$$C_{wt} = \frac{(23.8)(0.06) + (28.0)(0.8) + (28.2)\ (1.64)}{2.5}$$

$$= 28.0$$

$$L_{wt} = \frac{(0)(0.06) + (972)(0.8) + (2160)(1.64)}{2.5}$$

$$= 1728$$

$$\alpha_f = \frac{|\bar{T} - T_f|\, d_f}{nFI} = \frac{|48 - 32|\,(104)}{(0.9)(717)} = 2.6$$

$$\mu = \frac{nFI}{d_f}\left(\frac{C_{wt}}{L_{wt}}\right) = \frac{(0.9)(717)}{104}\left(\frac{28.0}{1728}\right) = 0.10$$

$$\lambda \cong 0.69 \text{ (Figure 2.31)}$$

$$x_{calculated} = (0.69)\left[\frac{48\,(0.9)(717)}{1932}\right]^{1/2} = 2.8 \text{ ft}$$

$$x_{assumed} = 2.5 \text{ ft}$$

$$x_{calculated} = 2.8 \text{ ft.}$$

(ii) Assume x = 36 in. ≈ 3.0 ft (second iteration)

$(L/k)_{eff}$ = 0.22 [324 + 2430 + 6595] = 2057
C_{wt} ≅ 28.1
L_{wt} = 1800
α_f = 2.1
μ = 0.1
λ = 0.73

$$x_{calculated} = (0.69)\left[\frac{48\,(0.9)(717)}{2057}\right]^{1/2} = 2.7 \text{ ft}$$

$$\left.\begin{aligned} x_{assumed} &= 3.0 \text{ ft} \\ x_{calculated} &\approx 2.7 \text{ ft} \end{aligned}\right\} \text{close enough}$$

Solution ≈ 2.7 ft ≈ 32 in.

Actual depth of freezing = 30 in.
(measured with temperature probe)

3.2.3 WASHINGTON STATE CLIMATE DATA

Figure 2.32 and Table 2.10 provide an overview of Washington State mean FI data (summarized for 1951 to 1980). Figure 2.33 is a contour map of Washington for design FI data. The FI contours for both Figures 2.32 and 2.33 are only approximate. FIs should be obtained at specific sites (projects) if possible.

3.2.4 DEPTH OF FREEZE — IMPLICATIONS FOR PAVEMENT DESIGN

One of the implications of the preceding calculations, FI contour maps, etc., is that the total depth of the pavement structure should be influenced in some way by such results. For example, several SHAs use the rule-of-thumb that the pavement structure should

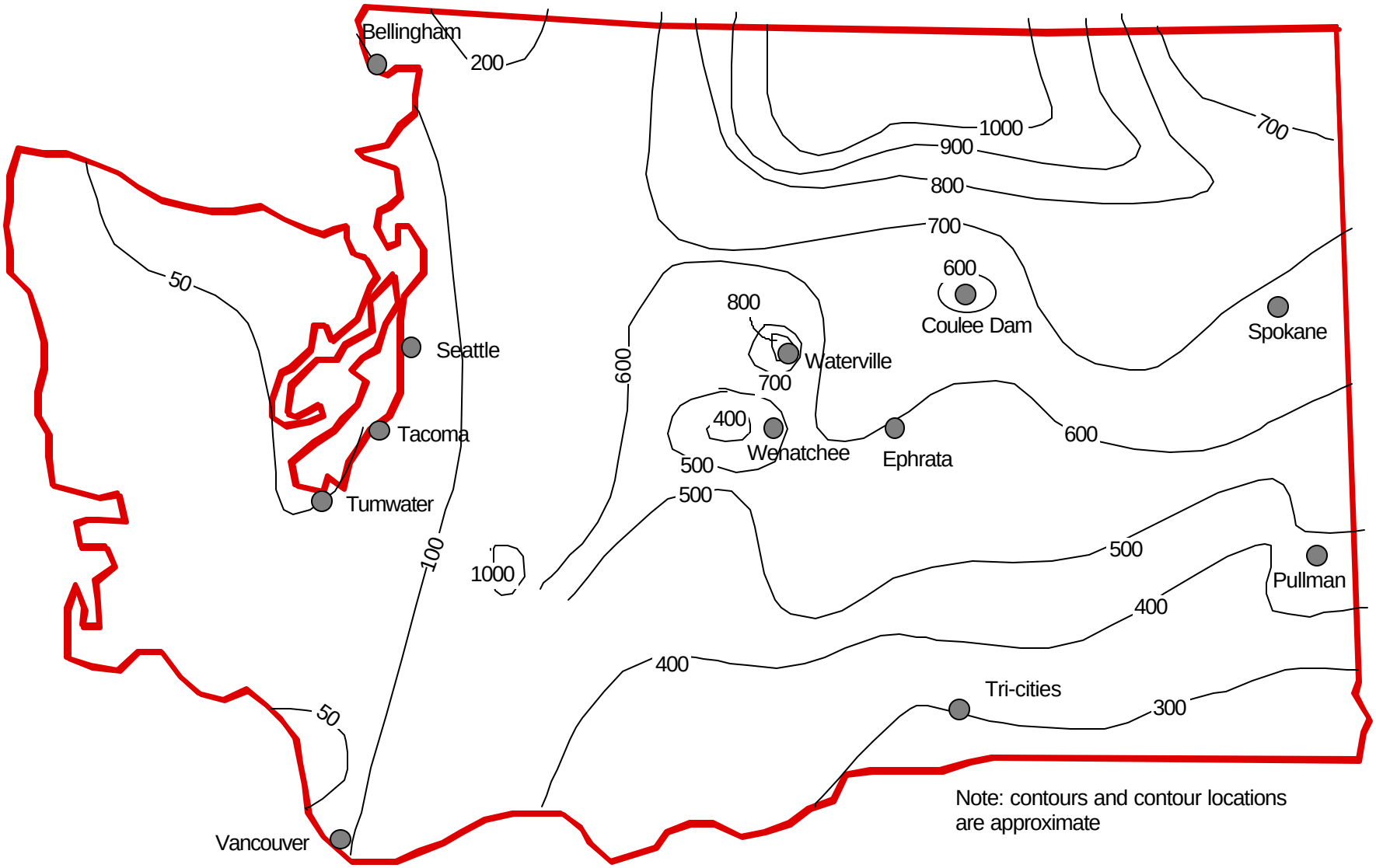


Figure 2.32. Mean Annual Freezing Index Contour Map

Table 2.10. Listing of Mean Freezing Indices for Various State of Washington Locations (based on temperature data from 1951 through 1980)

Station	Monthly Freezing Index (°F-day)												Mean Annual Freezing Index (°F-days)
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Aberdeen	18	0	0	0	0	0	0	0	0	0	0	6	24
Anacortes	30	0	0	0	0	0	0	0	0	0	0	7	37
Battle Ground	45	0	0	0	0	0	0	0	0	0	0	8	53
Bellingham	59	6	0	0	0	0	0	0	0	0	0	18	83
Bellingham Airport	67	7	0	0	0	0	0	0	0	0	0	23	97
Bickleton	231	68	18	7	0	0	0	0	0	0	0	42	120
Blaine	68	7	0	0	0	0	0	0	0	0	0	25	100
Bremerton	20	0	0	0	0	0	0	0	0	0	0	8	28
Buckly	40	8	0	0	0	0	0	0	0	0	0	17	65
Cedar Lake	85	26	10	0	0	0	0	0	0	0	9	140	70
Centralia	29	0	0	0	0	0	0	0	0	0	0	8	37
Chelan	262	110	10	0	0	0	0	0	0	0	29	149	560
Chewelah	339	150	35	0	0	0	0	0	0	0	62	216	802
Chief Joseph Dam	293	136	17	0	0	0	0	0	0	0	34	175	655
Clearbrook	103	13	0	0	0	0	0	0	0	0	6	39	161
Clearwater	22	0	0	0	0	0	0	0	0	0	0	6	28
Cle Elum	268	94	22	0	0	0	0	0	0	0	48	151	583
Colfax	226	45	9	0	0	0	0	0	0	0	29	99	408
Colville	321	119	29	0	0	0	0	0	0	0	75	221	765
Concrete	57	9	0	0	0	0	0	0	0	0	0	18	84
Coulee Dam	284	107	14	0	0	0	0	0	0	0	35	155	595
Coupeville	28	0	0	0	0	0	0	0	0	0	0	11	39
Dallesport Airport	179	14	0	0	0	0	0	0	0	0	8	52	253
Davenport	315	133	25	0	0	0	0	0	0	0	66	198	737
Dayton	206	30	0	0	0	0	0	0	0	0	18	61	315
Diablo Dam	126	23	9	0	0	0	0	0	0	0	10	54	222
Electron Headworks	78	20	8	0	0	0	0	0	0	0	8	40	154
Elma	22	0	0	0	0	0	0	0	0	0	0	7	29
Elwha Rngr Station	47	6	0	0	0	0	0	0	0	0	0	14	67
Ephrata Airport	285	98	5	0	0	0	0	0	0	0	41	167	596
Everett	35	0	0	0	0	0	0	0	0	0	0	11	46
Forks	22	5	0	0	0	0	0	0	0	0	0	10	37
Glenoma	47	9	0	0	0	0	0	0	0	0	0	17	73
Grapeview	17	0	0	0	0	0	0	0	0	0	0	7	24
Hatton	266	54	0	0	0	0	0	0	0	0	43	124	487
Hoquiam	15	0	0	0	0	0	0	0	0	0	0	9	24

Table 2.10. Listing of Mean Freezing Indices for Various State of Washington Locations (based on temperature data from 1951 through 1980) (Continued)

Station	Monthly Freezing Index (°F-day)												Mean Annual Freezing Index (°F-days)
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Kennewick	202	26	0	0	0	0	0	0	0	0	18	54	300
Kent	28	0	0	0	0	0	0	0	0	0	0	8	36
Kid Valley	43	8	0	0	0	0	0	0	0	0	0	16	67
Lacrosse	248	42	0	0	0	0	0	0	0	0	30	103	423
Landsburg	48	8	0	0	0	0	0	0	0	0	0	15	71
Laurier	350	121	35	0	0	0	0	0	0	0	82	238	826
Lind	266	62	0	0	0	0	0	0	0	0	43	126	497
Longview	36	0	0	0	0	0	0	0	0	0	0	6	42
Millin Reservoir	42	6	0	0	0	0	0	0	0	0	0	13	61
Monroe	38	0	0	0	0	0	0	0	0	0	0	14	52
Moses Lake	292	92	0	0	0	0	0	0	0	0	50	153	587
Mt. Adams Rngr Sta.	197	48	22	0	0	0	0	0	0	0	31	90	388
Moxee City	263	63	0	0	0	0	0	0	0	0	31	134	491
Mud Mtn. Dam	56	15	5	0	0	0	0	0	0	0	0	21	97
Newhalem	88	18	6	0	0	0	0	0	0	0	7	39	158
Newport	306	112	49	0	0	0	0	0	0	0	73	199	739
Northport	275	94	19	0	0	0	0	0	0	0	46	174	608
Oakville	35	0	0	0	0	0	0	0	0	0	0	10	45
Odessa	273	82	9	0	0	0	0	0	0	0	41	144	549
Olga	29	0	0	0	0	0	0	0	0	0	0	11	40
Olympia	31	5	0	0	0	0	0	0	0	0	0	15	51
Omak	344	175	28	0	0	0	0	0	0	0	67	234	848
Othello	276	64	0	0	0	0	0	0	0	0	35	125	500
Palmer	58	14	5	0	0	0	0	0	0	0	0	25	102
Pomeroy	201	32	7	0	0	0	0	0	0	0	22	66	328
Port Angeles	14	0	0	0	0	0	0	0	0	0	0	6	20
Prosser	240	46	0	0	0	0	0	0	0	0	22	84	392
Pullman	243	77	0	0	0	0	0	0	0	0	38	118	476
Puyallup	30	0	0	0	0	0	0	0	0	0	0	11	41
Quilcene	39	5	0	0	0	0	0	0	0	0	0	14	58
Quillayute	22	5	0	0	0	0	0	0	0	0	0	9	36
Quincy	303	106	9	0	0	0	0	0	0	0	51	189	658
Paradise	254	161	161	85	27	9	0	0	9	19	103	219	1047
Republic	408	170	73	0	0	0	0	0	0	0	117	304	1072
Richland	199	28	0	0	0	0	0	0	0	0	13	54	294
Ritzville	281	94	10	0	0	0	0	0	0	0	42	143	570

Table 2.10. Listing of Mean Freezing Indices for Various State of Washington Locations (based on temperature data from 1951 through 1980) (Continued)

Station	Monthly Freezing Index (°F-day)												Mean Annual Freezing Index (°F-days)
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Rosalia	269	94	22	0	0	0	0	0	0	0	49	142	576
Seattle	11	0	0	0	0	0	0	0	0	0	0	6	17
Sea-Tac	24	6	0	0	0	0	0	0	0	0	0	9	39
Sea U.W.	14	0	0	0	0	0	0	0	0	0	0	6	20
Sedro Wooley	46	6	0	0	0	0	0	0	0	0	0	15	67
Sequim	20	0	0	0	0	0	0	0	0	0	0	8	28
Shelton	21	0	0	0	0	0	0	0	0	0	0	8	29
Snqlm. Falls	44	10	0	0	0	0	0	0	0	0	0	16	70
Spokane	299	108	24	0	0	0	0	0	0	0	58	178	667
Sprague	287	94	14	0	0	0	0	0	0	0	48	148	591
Stampede Pass	283	150	116	48	13	0	0	0	0	9	105	213	937
Startup	38	7	0	0	0	0	0	0	0	0	0	13	58
Stehekin	195	70	12	0	0	0	0	0	0	0	44	127	448
Sunnyside	216	35	0	0	0	0	0	0	0	0	16	73	340
Tacoma City Hall	17	0	0	0	0	0	0	0	0	0	0	7	24
Vancouver	57	0	0	0	0	0	0	0	0	0	0	8	65
Walla-Walla Airport	192	24	0	0	0	0	0	0	0	0	18	59	293
Walla-Walla	188	20	0	0	0	0	0	0	0	0	14	50	272
Wapato	214	36	0	0	0	0	0	0	0	0	18	80	348
Waterville	349	152	51	0	0	0	0	0	0	0	84	246	882
Wenatchee	233	83	0	0	0	0	0	0	0	0	22	128	466
Wilbur	306	126	20	0	0	0	0	0	0	0	53	189	694
Willapa Harbor	12	0	0	0	0	0	0	0	0	0	0	0	12
Wilson Creek	276	79	6	0	0	0	0	0	0	0	42	163	566
Winthrop	451	206	65	0	0	0	0	0	0	0	108	342	1172
Yakima	258	63	0	0	0	0	0	0	0	0	31	123	475

Source: U.S. Department of Commerce, "Degree Days to Selected Bases," National Climatic Center, Federal Building, Asheville, N.C., December 1982.

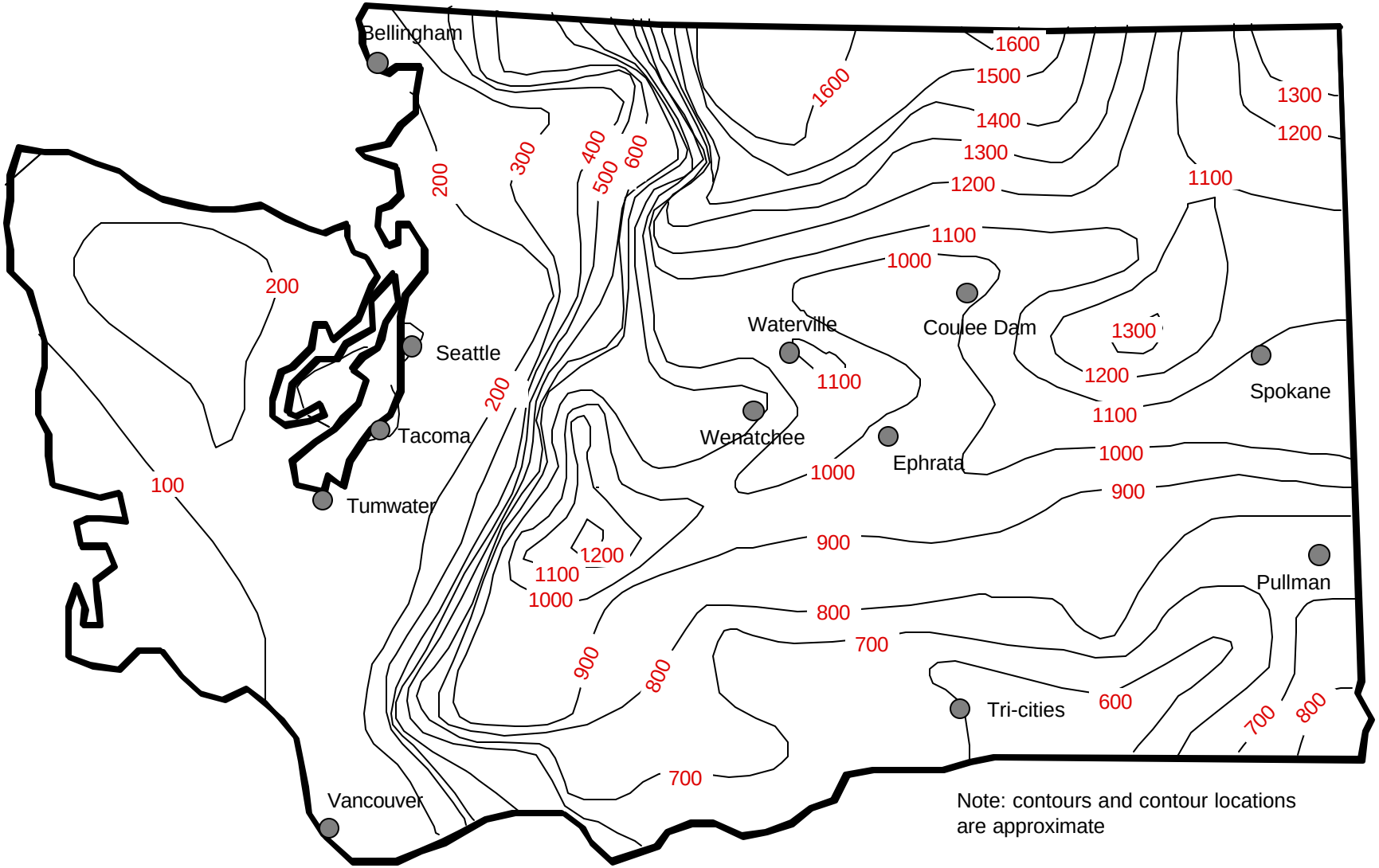


Figure 2.33. Design Annual Freezing Index Contour Map

equal at least one-half of the expected depth of freeze. To this end, Figures 2.34 and 2.35 were prepared. These contour maps show the expected depths of freeze corresponding to the design FI (refer to Figure 2.33) for fine-grain soil (Figure 2.34) and coarse-grain soil (Figure 2.35). The fine-grain soil calculations assumed a γ_d = 100 lb/ft^3 (1600 kg/m^3) and water content = 20 percent. The coarse-grain soil calculations assumed a γ_d = 130 lb/ft^3) (2080 kg/m^3)and water content = 5 percent.

Figure 2.36 shows contours of measured depths of freeze as determined during the extremely cold winters of 1949 and 1950 (letter correspondence from B. Tremper, State Materials and Research Engineer to W.A. Bugge, Director of Highways, dated, October 17, 1951). The freeze depths were measured in dug holes often along the edge of the main lanes. The freeze depths were measured during February 1949 and January and February 1950 (a total of 401 holes). Figure 2.36 is, in general, similar to Figure 2.34 (calculated freeze depths based on Design Freezing Indices and fine-grained soil) with the exception of the Olympic Penninsula which is closer to those results shown in Figure 2.35 (coarse-grained soil). Some observations made by Highway Department personnel during the winters of 1949 and 1950:

- Greatest freeze depths were observed in sandy or gravelly soils
- Snow or ice cover substantially reduced the depth of the freeze
- Frost heaving
    - Most heaving observed in coastal areas (higher availability of water)
    - Heaving somewhat infrequent in Eastern Washington but more severe when it did occur (again, likely related to the availability of water (or lack of))
    - Maximum differential heave of 225 mm (9 in.) noted in District 2
    - Silty sands showed the largest amount of ice lenses
- Specific District Comments
    - District 1 (Seattle): Maximum frost depth was measured between Issaquah and North Bend 0.8 m (30 in.). On Camano Island, a 0.5 m (20 in.) frost depth was measured. Maximum differential heave was 100 mm (4 in.) (several district locations).
    - District 2 (Wenatchee): Maximum depth of freeze was 0.9 m (36 in.) measured in 1949 (Wauconda Summit) and 1.3 m (51 in.) in 1950 (between Brewster and Okanogan).
    - District 3 (Tumwater): Maximum depth of freeze in 1949 was 0.6 m (24 in.) and 0.4 m (17 in.) in 1950.

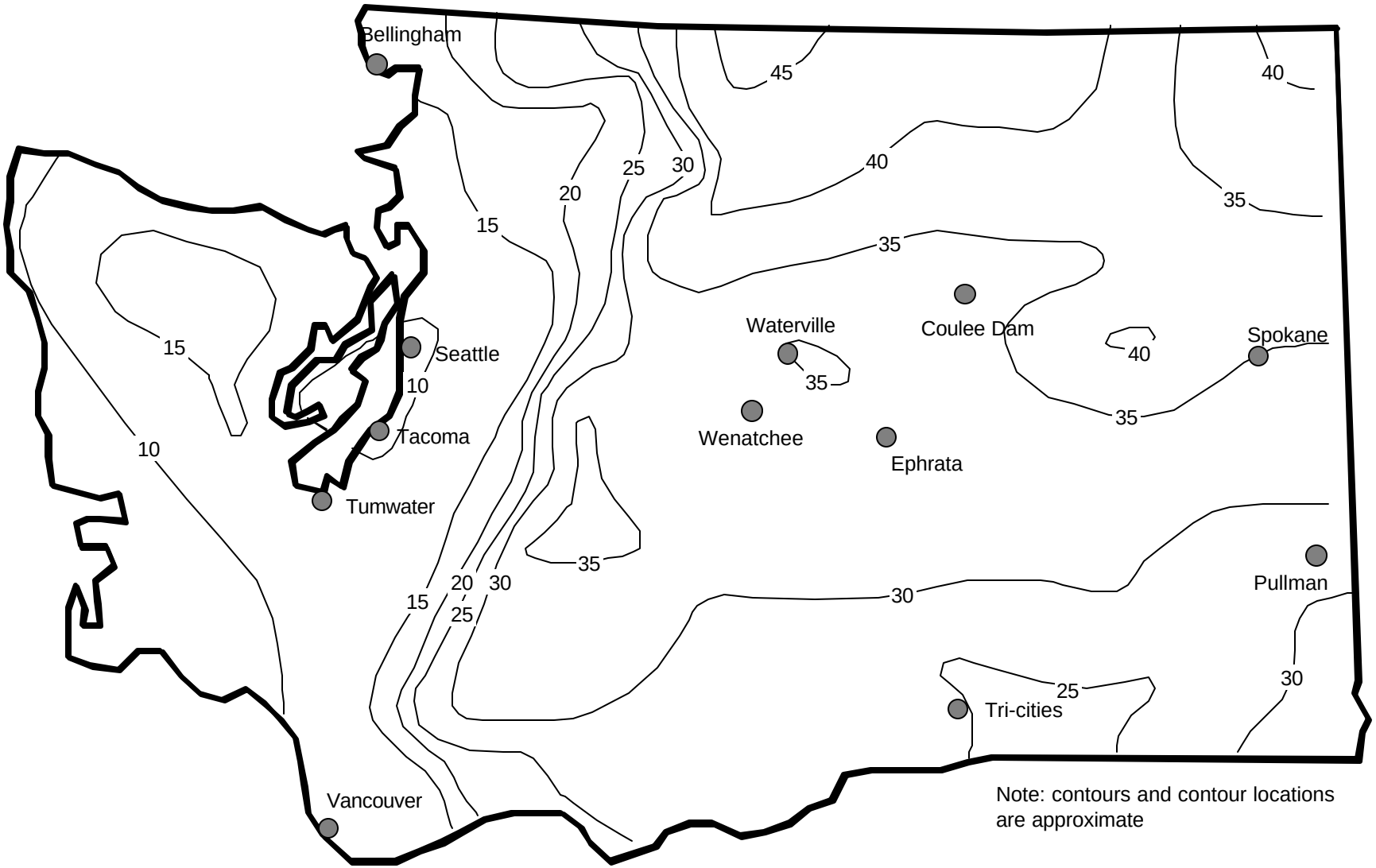


Figure 2.34. Frost Depth Contour Map (inches) for Fine Grained Soil (dry density = 100 pcf, wc = 20%)

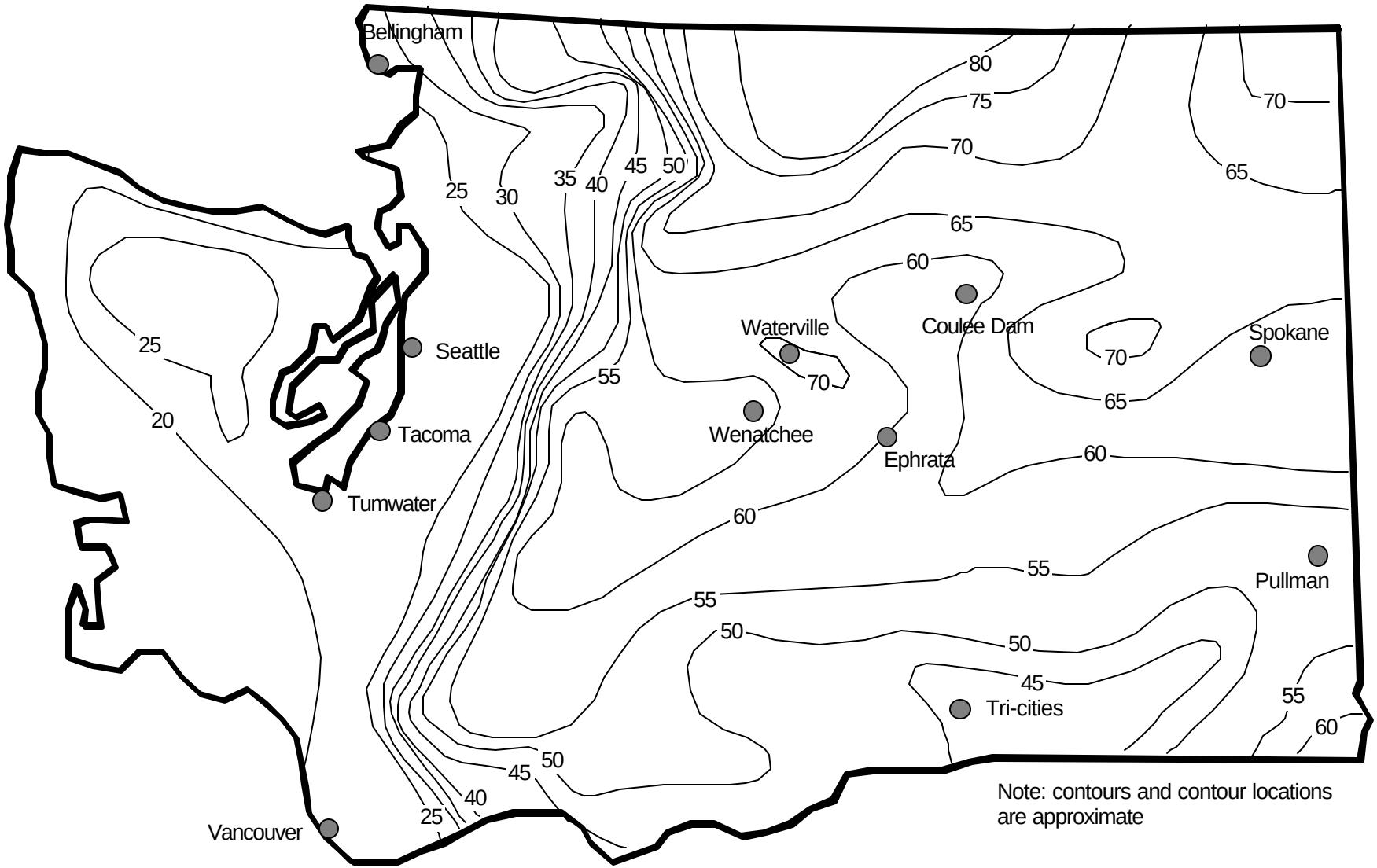


Figure 2.35. Frost Depth Contour Map (inches) for Coarse Grained Soil (dry density = 130 pcf, wc = 5%)

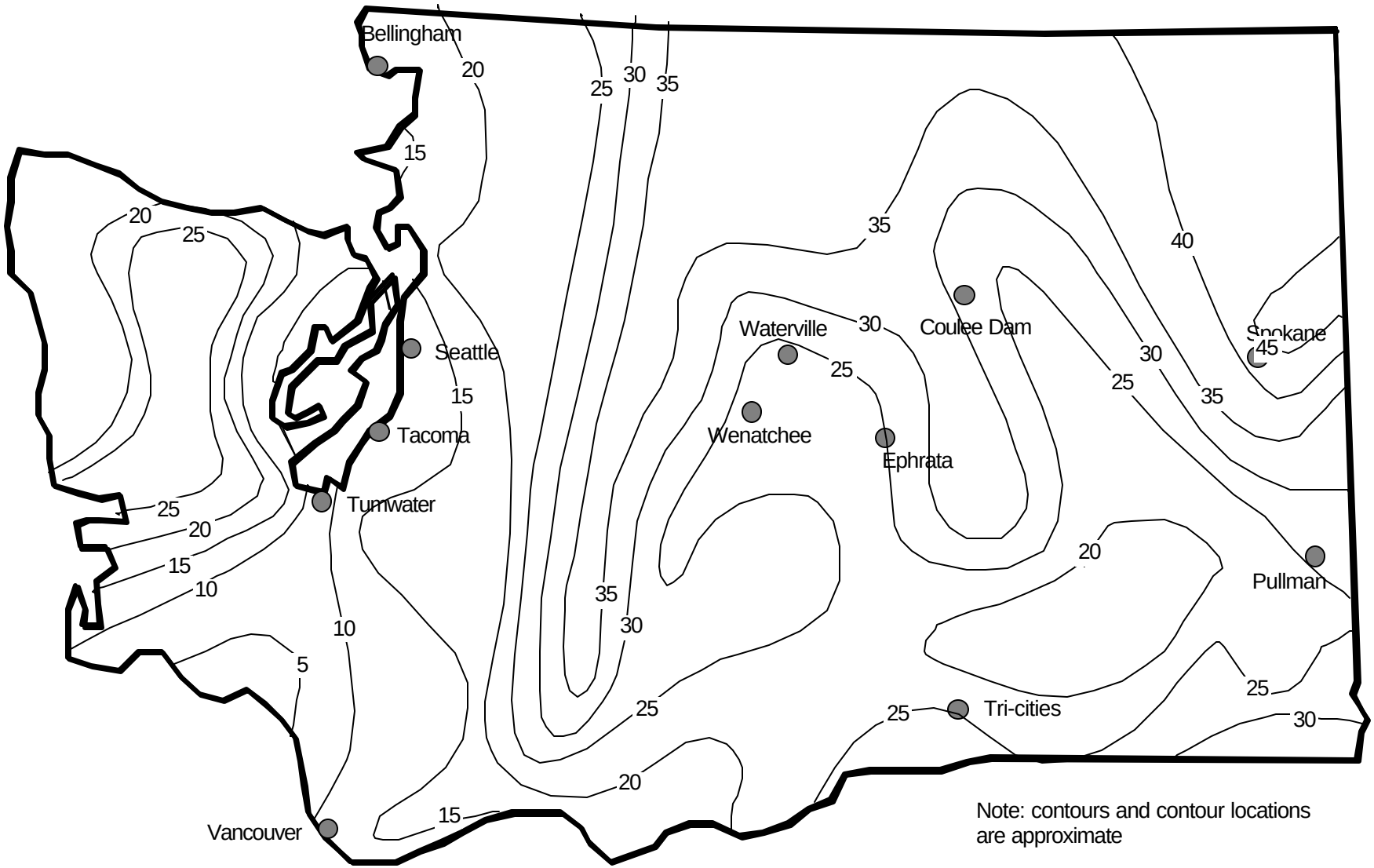


Figure 2.36. Frost Depth Contour Map (inches) Based on Field Measurements – Winters of 1949 and 1950

- District 4 (Vancouver): Maximum frost depth was 0.5 m (20 in.) in 1950.
- District 5 (Yakima): Maximum frost depth was 0.8 m (30 in.) measured in 1950 with a district-wide average of 0.6 m (24 in.) Differential heave of 150 mm (6 in.) was noted.
- District 6 (Spokane): The maximum depth of freeze was 1.1 m (43 in.) with a district average of 0.9 m (35 in.) measured in 1949. In 1950, the maximum was 1.2 m (48 in.) with a district average of 0.7 m (28 in.)

The statement about SHA frost design needs a bit of explanation. A survey conducted during 1985 [2.23] revealed the following from several "northern" states:

Agency	Use of Frost Protection in Thickness Design
• Alaska DOT	• More than 50 percent but not full
• Maine DOT	• More than 50 percent but not full
• Montana DOT	• Frost protection not included in design
• North Dakota DOT	• Frost protection not included in design
• Oregon DOT	• More than 50 percent but not full
• Washington DOT	• Depth > 50 percent of maximum frost depth expected

Thus, SHAs such as Alaska, Maine, Oregon, and Washington use knowledge about expected frost depths in the design process. Presumably, limiting the depth of frost into the subgrade soils limits, adequately, the potential for frost heave and thaw weakening for most projects/locations.

The above percentages (pavement structural section as a percentage of expected frost depth) are further reinforced by Japanese practice. Kono et al. [2.49] reported in 1973 that on the island of Hokkaido the pavement structure is set at 70 percent of the expected frost penetration (the pavement materials are non-frost susceptible).

3.2.5 FREEZING AND THAWING INDICES — IMPLICATIONS FOR MAINTENANCE OPERATIONS

The calculated FI and TI can be used to estimate the depth of freeze at a specific site (FI) and the resulting thaw (TI). The TI can be used to assess the need for seasonal load limits by maintenance personnel. Such measures are commonly used in District 2 during the winter and spring months. For more information on seasonal load restrictions, refer to Rutherford et al. [2.25] or Mahoney et al. [2.26].

Based on the referenced study [2.25, 2.26], the following guidelines relative to spring highway load restrictions were developed and evaluated. These guidelines were verified based on results from District 2 [2.27]. The guidelines can be divided into where, how much, when, and how long to apply load restrictions.

(a) Where to apply load restrictions. If pavement surface deflections are available to an agency, spring thaw deflections greater than 45 to 50 percent of summer deflections suggest a need for load restriction. Further, considerations such as depth of freezing (generally areas with air Freezing Indices of 400 °F-days or more), pavement surface thickness, moisture condition, type of subgrade, and local experience should be considered. Subgrades with Unified Soil Classifications of ML, MH, CL, and CH will result in the largest pavement weakening.

(b) Amount of load reduction. The minimum load reduction level should be 20 percent. Load reductions greater than 60 percent generally are not warranted based on potential pavement damage. A load reduction range of 40 to 50 percent should accommodate a wide range of pavement conditions.

(c) When to apply load restrictions. Load restrictions should be applied after accumulating a Thawing Index (TI) of about 25 °F-days (based on an air temperature datum of 29 °F) and must be applied at a TI of about 50 °F-days (again based on an air temperature datum of 29 °F). Corresponding TI levels are less for thin pavements (e.g., two inches of asphalt concrete and six inches of aggregate base or less) in that the should TI level is 10 °F-days and the must TI level is 40 °F-days.

(d) When to remove load restrictions. Two approaches are recommended, both of which are based on air temperatures. The duration of the load restriction period can be directly estimated by the following relationship which is a function of Freezing Index (FI):

$$\text{Duration (days)} = 25 + 0.01\ (\text{FI})$$

Further, the duration can be estimated by use of TI and the following relationship:

$$\text{TI} \simeq 0.3\ (\text{FI})$$

Appendix 2.3 contains an example set of calculations based on actual data from SR 172 near Mansfield, Washington (winter of 1985-1986).

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SECTION 2.0, APPENDIX 2.1

AASHTO VEHICLE DEFINITIONS
[FROM REF. 2.22]

1.	**Axle**	The common axis of rotation of one or more wheels whether power-driven or freely rotating, and whether in one or more segments, and regardless of the number of wheels carried thereon.
2.	**Axle Group**	An assemblage of two or more consecutive axles considered together in determining their combined load effect on a bridge or pavement structure.
3.	**Automobile Transporter**	Any vehicles or combination designed and used exclusively for the transport of assembled highway vehicles.
4.	**Bus**	A motor vehicle designed primarily for the transportation of persons rather than property and having a passenger-carrying capacity of 10 or more persons, other than a taxicab constructed and designed for transporting persons for commercial purposes.
5.	**Cargo**	The items or freight to be moved; including items placed on or in a vehicle, towed by a vehicle, or a vehicle itself.
6.	**Connecting Mechanism**	An arrangement of parts interconnecting two or more consecutive axles to the frame of a vehicle in such a manner as to equalize the load between axles.
7.	**Dromedary Unit**	A load carrying compartment on a truck-tractor located between the cab and the fifth wheel.
8.	**Gross Weight**	The weight of a vehicle and/or combination of vehicles plus the weight of any load thereon.
9.	**Height**	The total vertical dimension of a vehicle above the ground surface including any load and load-holding device thereon.
10.	**Length**	The total longitudinal dimension of a single vehicle, a trailer, or a semitrailer. Length of a trailer or semitrailer is measured from the front of the cargo-carrying unit to its rear, exclusive of all overhang, safety or energy efficiency devices, including air conditioning units, air compressors, flexible fender extensions, splash and spray suppressant devices, bolsters, mechanical fastening devices, and hydraulic lift gates.

11.	**Load**	A weight or quantity of anything resting upon something else regarded as its support.
12.	**Motor Vehicle**	A vehicle which is self-propelled or propelled by electric power obtained from overhead trolley wires, but not operating upon rails.
13.	**National Truck Network**	Those interstate and other federal-aid primary highways on which commercial vehicles of the dimensions authorized by the STAA of 1982 are allowed to operate.
14.	**Operator**	Every person who drives or is in actual physical control of a motor vehicle upon a highway or who is exercising control over or steering a vehicle being towed by a motor vehicle.
15.	**Owner**	A person, other than a lien-holder, having the property in or title to a vehicle, including a person entitled to the use and possession of a vehicle subject to a security interest in another person, but excluding a lessee under a lease not intended as security.
16.	**Pavement Structure**	The combination of subbase, base course, and surface course placed on an earth subgrade to support the traffic load and distribute it to the roadbed.
17.	**Quadrum Axle**	Any four consecutive axles whose extreme centers are not more than 192 inches (16 ft or 4.9 m) apart and are individually attached to or articulated from, or both, a common attachment to the vehicle including a connecting mechanism designed to equalize the load between the axles.
18.	**Regular Operation**	The movement over highways of vehicles, vehicle combinations, and loads thereon, subject to the recommended limitations contained in this guide governing maximum weights and dimensions for motor vehicles and loads thereon.
19.	**Scale Tolerance**	An allowable variation in the static weight of an axle load in accordance with, but not exceeding the precision of the scale involved.
20.	**Semitrailer**	Every single vehicle without motive power designed for carrying property and so designed in conjunction and used with a motor vehicle that some part of its own weight and that of its load rests or is carried by another vehicle and having one or more load-carrying axles.

21.	**Single Axle**	An assembly of two or more wheels whose centers are in one transverse vertical plane or may be included between two parallel transverse planes 40 inches (3.3 ft or 1.0 m) apart extending across the full width of the vehicle.
22.	**Special Permit**	A written authorization to move or operate on a highway a vehicle or vehicles with or without a load of size and/or weight exceeding the limits prescribed for vehicles in regular operation.
23.	**Special Permit Applicant**	An individual, firm, partnership, corporation, or association making application for a special permit to transport a vehicle, vehicles, and/or load which is oversize or overweight and under whose authority and responsibility such vehicle or load is transported.
24.	**Steering Axle**	The axle or axles of a motor vehicle or combination of vehicles by which the same is guided or steered.
25.	**Stinger-Steered Automobile Transporter**	A truck-tractor semitrailer combination where the fifth wheel is located on a drop frame behind and below the drive axle of the power unit. In this configuration, vehicles are carried behind or both behind and above the cab of the power unit, as well as on the semitrailer.
26.	**Tandem Axle**	Any two axles whose centers are more than 40 inches (3.3 ft or 1.0 m) but not more than 96 inches (8 ft or 2.4 m) apart and are individually attached to or articulated from, or both, a common attachment to the vehicle including a connecting mechanism designed to equalize the load between axles.
27.	**Tire, Pneumatic**	A tire of rubber or other resilient material which depends upon compressed air for support of a load.
28.	**Trailer**	Every single vehicle without motive power designed for carrying property wholly on its own structure, drawn by a motor vehicle which carries no part of the weight and load of the trailer on its own wheels and having two or more load carrying axles.
29.	**Traveled Way**	The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
30.	**Tridum Axle**	Any three consecutive axles whose extreme centers are not more than 144 inches (12 ft or 3.7 m) apart, and are individually attached to or articulated from, or both, a common attachment to the vehicle including a connecting mechanism designed to equalize the load between axles.

31. **Triple Saddle Mount** — A combination of four truck-tractors where the front axle of second truck-tractor is mounted on the fifth wheel of the lead truck-tractor, the front axle of the third truck-tractor is mounted on the fifth wheel of the second truck-tractor, and the front axle of the fourth truck-tractor is mounted on the fifth wheel of the third truck-tractor; and with the rear wheels of the second, third, and fourth truck-tractors trailing on the ground behind the operating motor unit.

32. **Truck** — A single unit motor vehicle used primarily for the transportation of property.

33. **Truck Tractor** — A motor vehicle used primarily for drawing other vehicles and not so constructed as to carry a load other than a part of the weight of the vehicle and load so drawn.

34. **Turning Path** — The path of a designated point on a vehicle making a specified turn.

35. **Turning Track Width** — The radial distance between the turning paths of the outside of the outer front tire and the outside of the rear tire which is nearest the center of the turn.

36. **Variable Load Suspension Axles** — Axles which can be regulated by the driver of the vehicle. These axles are controlled by hydraulic and air suspension systems, mechanically, or by a combination of these methods.

37. **Vehicle** — A device in, upon, or by which any person or property may be transported or drawn upon a highway, except devices moved by human power or used exclusively upon stationary rails or tracks.

38. **Vehicle Combination** — An assembly of two or more vehicles coupled together for travel upon a highway.

39. **Width** — The total outside transverse dimension of a vehicle including any load or load-holding devices thereon, but excluding approved safety devices and tire bulge due to load.

SECTION 2.0, APPENDIX 2.2

GRADATION BANDS FOR WSDOT MIXTURES

Figures 1 through 6 contain plots of the gradation bands and the associated dense grading line (0.45 power) for various WSDOT mixtures. These mixtures are:

- Figure 1: WSDOT ACP Classes A and B (Dense Graded)
- Figure 2: WSDOT ACP Class D (Open Graded)
- Figure 3: WSDOT ACB Class E (Dense Graded)
- Figure 4: WSDOT ACP Class F (Dense Graded)
- Figure 5: WSDOT ACP Class G (Dense Graded)
- Figure 6: WSDOT Open Graded Emulsion Asphalt Pavement

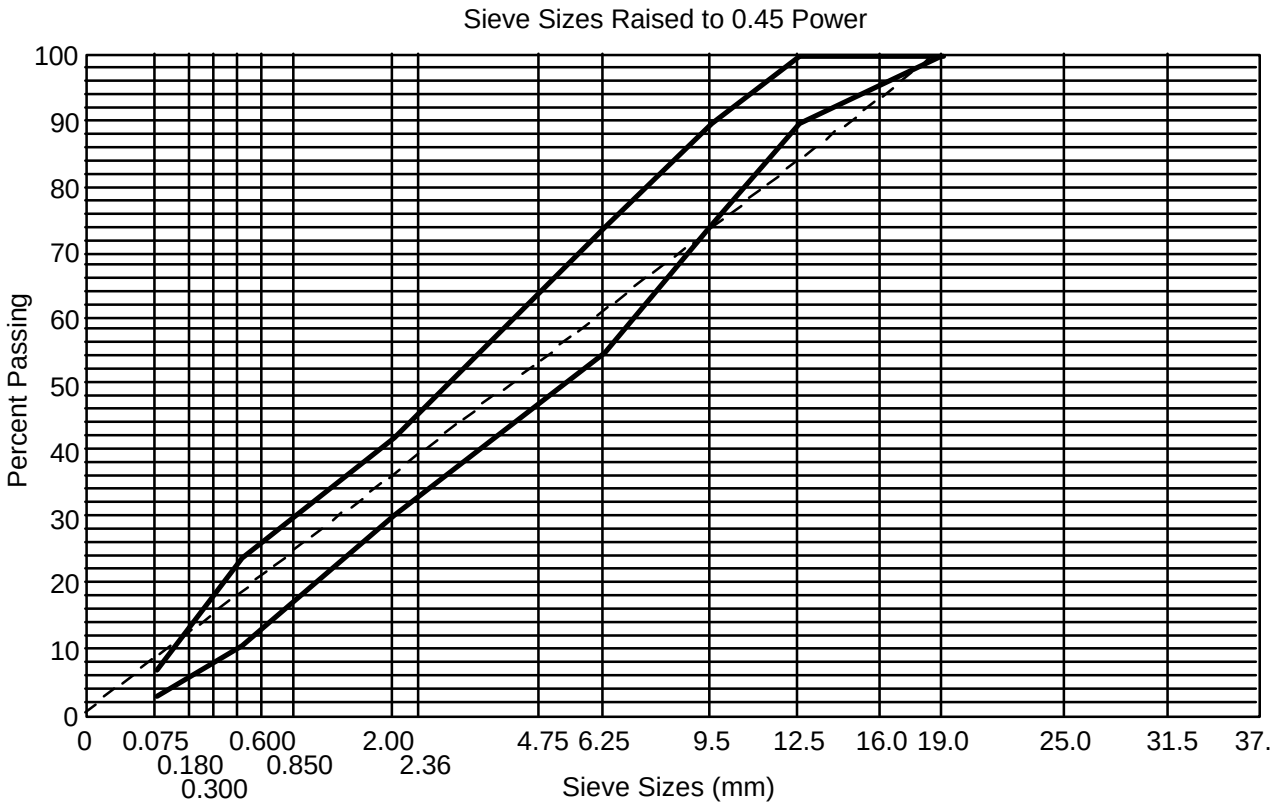


Figure 1. Maximum Density Curve (0.45 Power) for 19.0 mm Maximum Aggregate with the Gradation Band for WSDOT ACP Classes A and B

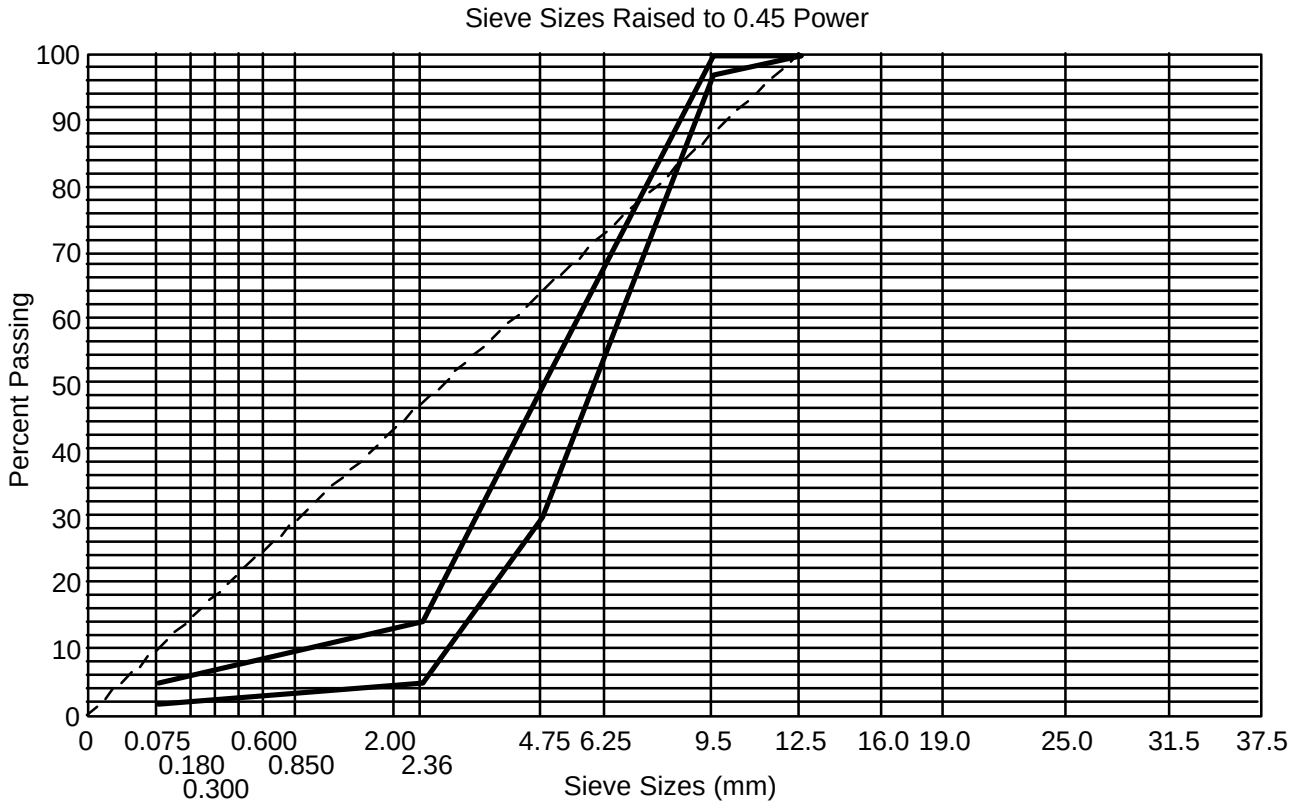


Figure 2. Maximum Density Curve (0.45 Power) for 12.5 mm Maximum Aggregate with the Gradation Band for WSDOT ACP Class D

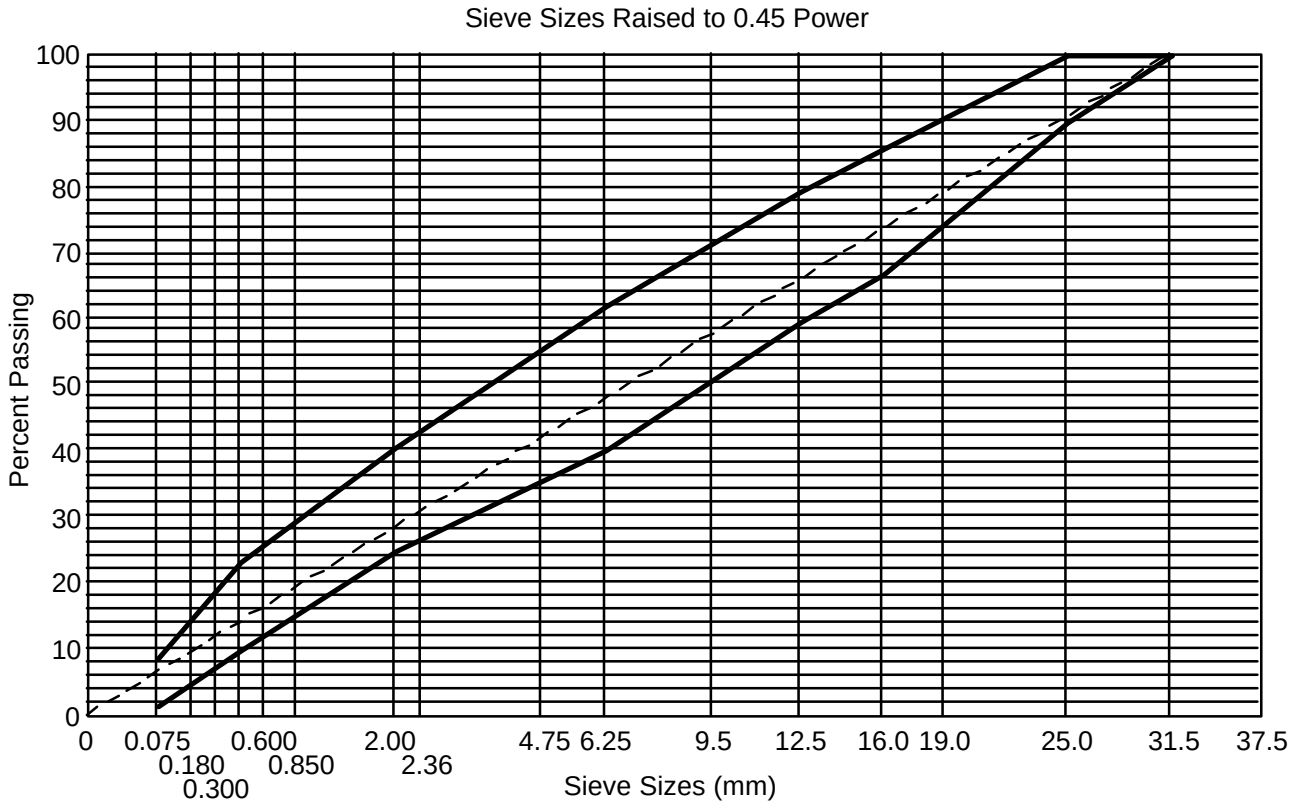


Figure 3. Maximum Density Curve (0.45 Power) for 31.5 mm Maximum Aggregate with the Gradation Band for WSDOT ACB Class E

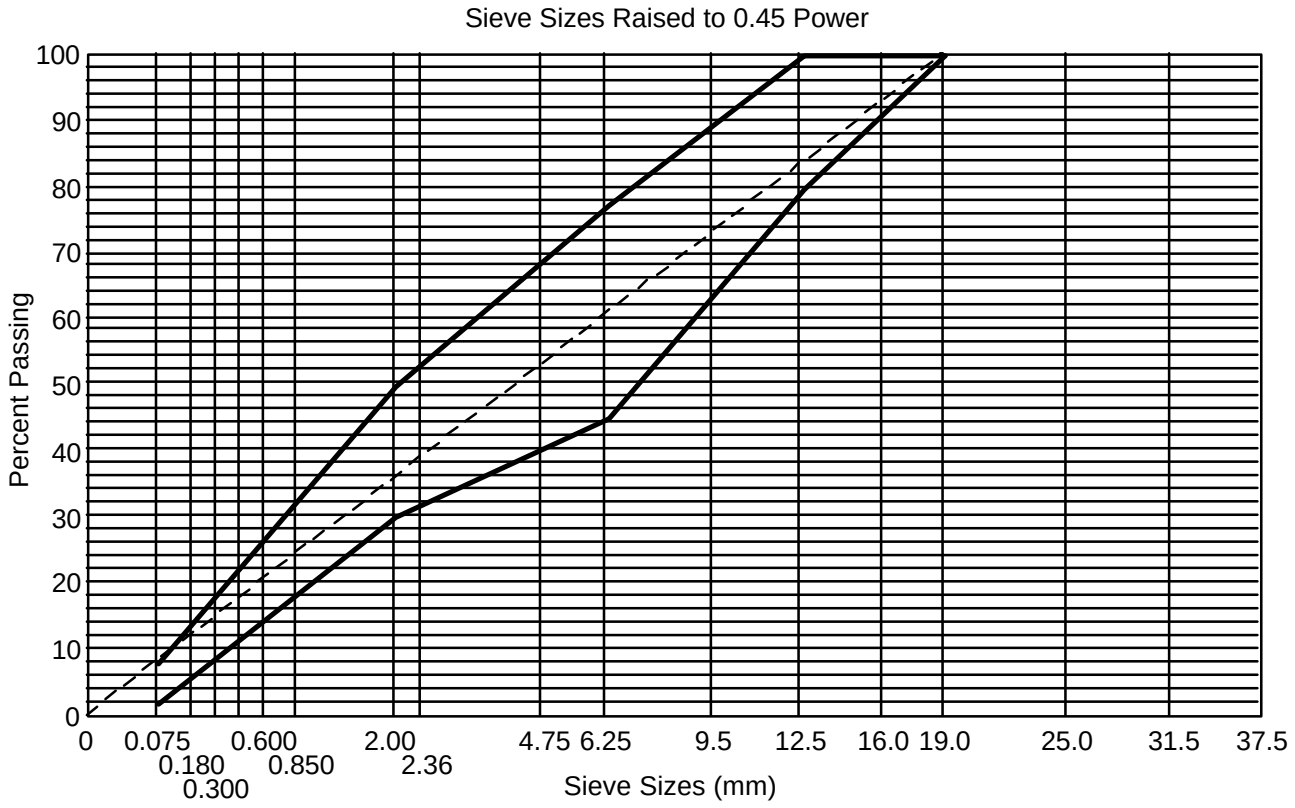


Figure 4. Maximum Density Curve (0.45 Power) for 19.0 mm Maximum Aggregate with the Gradation Band for WSDOT ACP Class F

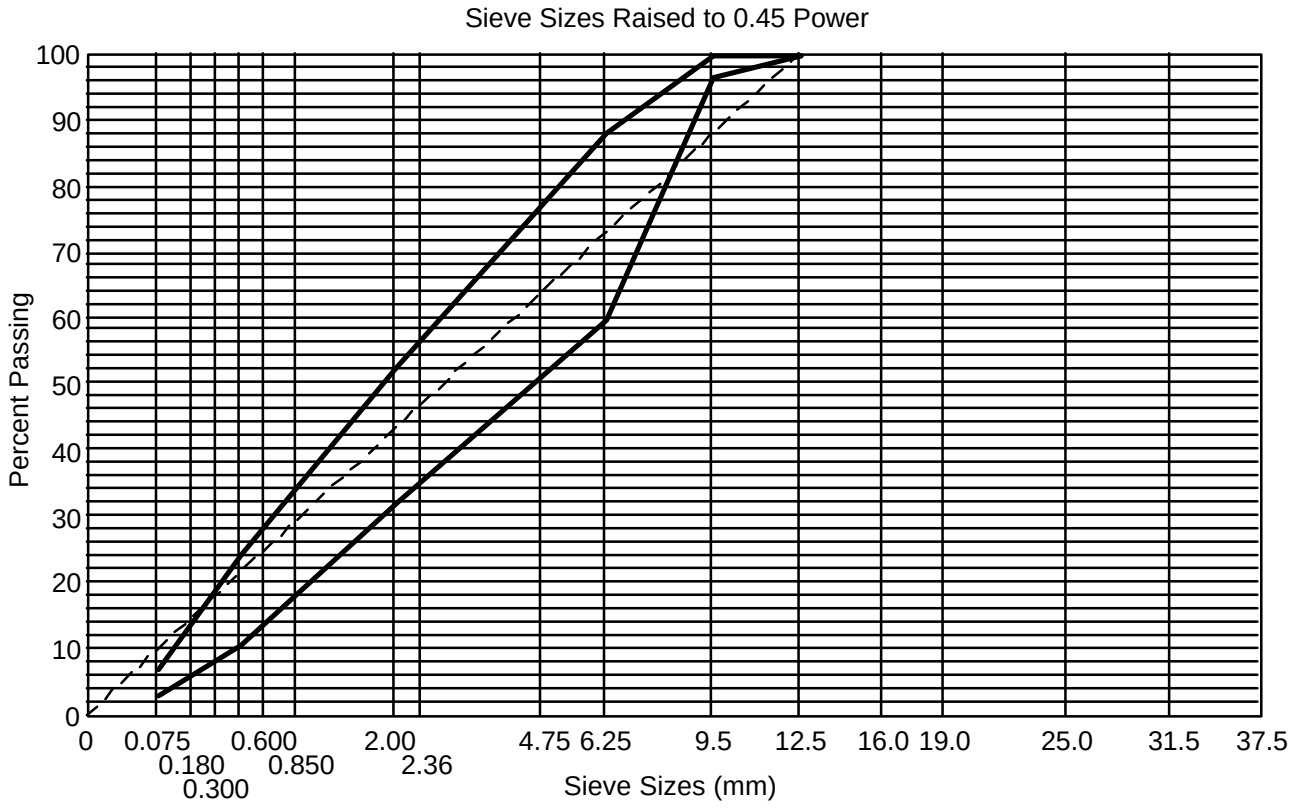


Figure 5. Maximum Density Curve (0.45 Power) for 12.5 mm Maximum Aggregate with the Gradation Band for WSDOT ACP Class G

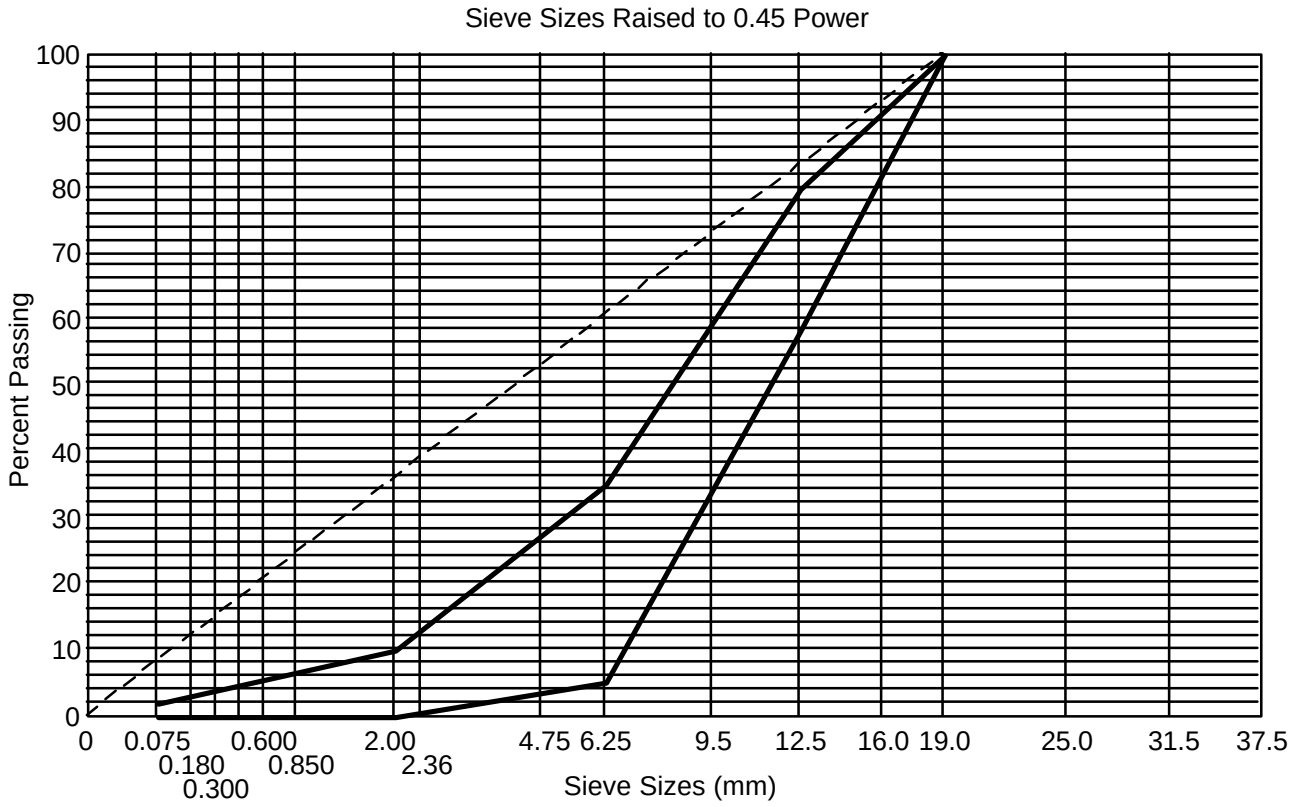


Figure 6. Maximum Density Curve (0.45 Power) for 19.0 mm Maximum Aggregate with the Gradation Band for WSDOT Open Graded Emulsion Asphalt Pavement (OGEAP)

SECTION 2.0, APPENDIX 2.3

EXAMPLE OF DATA COLLECTION AND ESTIMATION OF START AND DURATION FOR IMPOSING LOAD RESTRICTIONS

Location:	Mansfield, Washington (central Washington state) Pavement section typically restricted during spring thawing 2 inches bituminous surfacing 6 inches granular base Silty subgrade

High and low daily temperatures were collected through the freezing and thawing period to calculate Freezing Index, based on 32°, and Thawing Index based on 29 °F (Figure 1).

CALCULATING THE FREEZING INDEX

The Freezing Index is a measure of the magnitude and duration of the temperature differential during the freezing period. The freezing index is calculated using the following equation:

$$FI = \Sigma (32 - \overline{T})$$

where: $\overline{T} = 1/2 (T_H + T_L)$ in °F,

T_H = maximum daily temperature (°F), and

T_L = minimum daily temperature (°F).

The temperature data collected for Mansfield to identify the freezing period and the freezing index are shown in Figure 1.

STEPS:

1. When $\overline{T}$ becomes less than or equal to 32 °F for several days, the freezing season begins. The freezing season for 1985 begins on November 9.

2. The average daily temperature is equal to

$$\overline{T} = 1/2(\text{column 3} + \text{column 4})$$

On November 13, for example:

$$\overline{T} = 1/2(35 + 7) = \underline{21\ °F}$$

3. The freezing degree-days per day (column 6) is equal to

$$\text{Daily FI} = 32 - \overline{T} \text{ (from column 5)}$$

For November 13, for example:

$$\text{Daily FI} = (32 - 21) = \underline{11\ °F\text{-days}}$$

4. The Freezing Index is the accumulation of daily freezing degree days from the start of freezing

$$FI = \Sigma (32 - \overline{T}) \text{ from the start of freezing}$$

For November 13, for example

$$FI = (3 + 7 + 9 + 8 + 11) = 38\ °F\text{-days}$$

5. The end of the freezing season is near for pavements when the average daily air temperatures (column 5) in spring go above 29 °F for several days, causing thawing of the pavement to begin. The thawing season for Mansfield during 1986 begins on February 24 (refer to Figure 1). The Freezing Index for the entire freezing season from November 9 to February 23 is

$$FI = \Sigma (32 - \overline{T})$$

$$FI = (3 + 7 + 9 + 8 + ... + 24 \text{ (February 21)} + 18 \text{ (February 22)} + 10 \text{ (February 23)})$$

$$FI = \underline{1375\ °F\text{-days}}$$

A review of the temperature data in Figure 1 shows that four thawing periods occurred during January and February. Three of these periods were followed by freezing periods, thus canceling any cumulative thawing effects (approximately) and reducing the cumulative freezing effects as well.

ESTIMATING THE TIME TO PLACE LOAD RESTRICTIONS

The pavement consists of 2 inches of asphalt concrete on 6 inches of aggregate base. This is classified as a thin pavement. The "should" level for placing load restrictions for thin pavements is

TI_{29} should restrict = 10 °F-days

The thawing season starts on February 24.

TI_{29} = 9 (February 24) + 14 (February 25)

= 23 °F-days

The load restrictions should be placed by February 26.

TI must restrict = 40 °F-days

The "must" level for restricting a thin pavement is

TI_{29} = 9 (February 24) + 14 (February 25) + 11 (February 26) + 7 (February 27)

= 41 °F-days

The load restrictions must be placed by February 28.

The earlier thaw period (January 31 to February 7) could have been used to start load restrictions. However, this would have been somewhat premature being as this period was followed by more freezing weather. As with any criterion, judgment must be used. For this location (Mansfield, Washington), the normal thaw period starts during the last week of February or the first week of March.

ESTIMATING THE DURATION FOR LOAD RESTRICTIONS

The duration may be estimated in days or in thawing degree-days. It is preferable to estimate the duration of the thawing period using the Thawing Index based on 29 °F.

To estimate the number of thawing degree days required for the restricted period the exact equation is:

$$TI_{29} = 4.154 + 0.259\ (FI)$$

$$TI_{29} = 4.154 + 0.259\ (1375\ °F\text{-days})$$

$$= 360\ °F\text{-days}$$

On March 28, the TI_{29} (column 9) is 347 °F-days

On March 29, the TI_{29} is 368 °F-days

Therefore, the load restrictions should be removed by March 30.

The simpler approximate equation for the thawing degree-days required for the restricted period which may be used in place of the above equation is:

$$TI_{29} = 0.3\ (FI)$$

$$TI_{29} = 0.3\ (1375\ °F\text{-days})$$

$$= 412\ °F\text{-days}$$

On March 31, the TI_{29} is equal to 412 °F-days. Therefore, the load restrictions should be removed by April 1. Alternatively, the duration of the thawing period may be estimated in days.

The exact equation for estimating duration in days is

$$D = 22.62 + 0.011\ (FI)$$

For this freezing season in Mansfield,

$$FI = 1375\ °F\text{-days}$$

$$D = 22.62 + 0.011\ (1375\ °F\text{-days})$$

= <u>38 days</u> from the start of thawing (February 24) = <u>April 2</u>

A simpler approximate equation for estimating duration in days which may be used instead of the preceding equation is

$$D = 25 + 0.01\ (FI)$$

$$D = 25 + 0.01\ (1375\ °F\text{-days})$$

$$= 25 + 14$$

$$= \underline{39\ \text{days}} = \underline{\text{April 3}}$$

Day	Month/ Year	Measured Daily Air Temperature (^oF)		Average Daily Air Temperature, (^oF)	Daily Freezing Index = 32° F - Avg. Daily Temp. (see note) (^oF - days)	Sum of Daily Freezing Index (^oF-days)	Daily Thawing Index = Avg. Daily Temp. - 29° F. (see note) (^oF - days)	Sum of Daily Thawing Index (^{o} F-days)
		High	Low	(High + Low)/2				
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
1	Nov., '85	43	24	34				
2	" "	51	29	40				
3	" "	55	26	40				
4	" "	60	35	48				
5	" "	50	30	40				
6	" "	50	30	40				
7	" "	54	25	40				
8	" "	50	18	34				
9	" "	45	13	29	3	3		
10	" "	40	10	25	7	10		
11	" "	35	11	23	9	19		
12	" "	40	8	24	8	27		
13	" "	35	7	21	11	38		
14	" "	33	4	18	14	52		
15	" "	30	4	17	15	67		
16	" "	32	7	20	12	79		
17	" "	26	5	16	16	95		
18	" "	35	10	22	10	105		
19	" "	20	-6	7	25	130		
20	" "	16	-6	5	27	157		
21	" "	14	-4	5	27	184		
22	" "	14	0	7	25	209		
23	" "	12	-8	2	30	239		
24	" "	11	-8	2	30	269		
25	" "	10	-12	-1	33	302		
26	" "	10	-8	1	31	333		
27	" "	16	-8	4	28	361		
28	" "	13	-8	2	30	391		
29	" "	11	-8	2	30	421		
30	" "	27	-8	10	22	443		

Note: Calculate Daily Freezing Index starting at the beginning of the freezing season and accumulate throughout the normal freezing period. The air temperature datum for Freezing Index is 32°F and 29°F for Thawing Index. The Thawing Index period for much of the U.S. will start in late February of April

Figure 1. Worksheet for Bituminous Surfaced Pavements

Day	Month/ Year	Measured Daily Air Temperature (^oF)		Average Daily Air Temperature, (^oF)	Daily Freezing Index = 32° F - Avg. Daily Temp. (see note) (^oF - days)	Sum of Daily Freezing Index (^oF-days)	Daily Thawing Index = Avg. Daily Temp. - 29° F. (see note) (^oF - days)	Sum of Daily Thawing Index (^{o} F-days)
		High	Low	(High + Low)/2				
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
1	Dec., '85	17	-8	4	28	471		
2	" "	8	-8	0	31	503		
3	" "	18	2	10	22	525		
4	" "	24	8	16	16	541		
5	" "	34	12	23	9	550		
6	" "	34	12	23	9	559		
7	" "	34	16	25	7	566		
8	" "	34	14	24	8	574		
9	" "	34	20	27	5	579		
10	" "	26	12	19	13	592		
11	" "	20	2	11	21	613		
12	" "	22	12	17	15	628		
13	" "	26	-4	11	21	649		
14	" "	23	-3	10	22	671		
15	" "	22	-2	10	22	693		
16	" "	20	-3	8	24	717		
17	" "	20	2	11	21	738		
18	" "	30	2	16	16	754		
19	" "	25	10	18	14	768		
20	" "	28	18	23	9	777		
21	" "	26	18	22	10	787		
22	" "	24	14	19	13	800		
23	" "	24	18	21	11	811		
24	" "	22	16	19	13	824		
25	" "	22	14	18	14	838		
26	" "	21	12	16	16	854		
27	" "	21	14	18	14	868		
28	" "	18	14	16	16	884		
29	" "	20	12	16	16	900		
30	" "	20	12	16	16	916		
31	" "	20	10	15	17	933		

Note: Calculate Daily Freezing Index starting at the beginning of the freezing season and accumulate throughout the normal freezing period. The air temperature datum for Freezing Index is 32^oF and 29^oF for Thawing Index. The Thawing Index period for much of the U.S. will start in late February of April

Figure 1. Worksheet for Bituminous Surfaced Pavements (Cont.)

Day	Month/ Year	Measured Daily Air Temperature (^oF)		Average Daily Air Temperature, (^oF)	Daily Freezing Index = 32° F - Avg. Daily Temp. (see note) (^oF - days)	Sum of Daily Freezing Index (^oF-days)	Daily Thawing Index = Avg. Daily Temp. - 29° F. (see note) (^oF - days)	Sum of Daily Thawing Index (^{o} F-days)
		High	Low	(High + Low)/2				
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
1	Jan., '86	27	5	16	16	949		
2	" "	34	0	17	15	964		
3	" "	26	6	16	16	980		
4	" "	28	9	18	14	994		
5	" "	28	9	18	14	1008		
6	" "	30	12	31	11	1019		
7	" "	26	8	17	15	1034		
8	" "	30	10	30	12	1046		
9	" "	40	16	39	4	1050		
10	" "	37	28	32	0	1050	3	3
11	" "	39	20	30	2	1052	1	4
12	" "	34	16	25	7	1059	-4	0
13	" "	32	10	21	11	1070		
14	" "	18	18	18	14	1084		
15	" "	26	14	20	12	1096		
16	" "	27	20	24	8	1104		
17	" "	36	26	31	1	1105	2	2
18	" "	46	30	38	-6	1099	9	11
19	" "	40	30	25	-3	1096	6	17
20	" "	40	21	30	2	1098	1	18
21	" "	40	21	30	2	1100	1	19
22	" "	40	12	26	6	1106	-3	16
23	" "	34	22	28	4	1110	-1	15
24	" "	41	20	30	2	1112	1	16
25	" "	32	6	19	13	1125	-10	6
26	" "	26	6	16	16	1141	-13	--
27	" "	28	16	22	10	1151		
28	" "	31	24	28	4	1155		
29	" "	30	20	25	7	1162		
30	" "	34	24	29	3	1165	0	0
31	" "	28	30	34	-2	1163	5	5

Note: Calculate Daily Freezing Index starting at the beginning of the freezing season and accumulate throughout the normal freezing period. The air temperature datum for Freezing Index is 32°F and 29°F for Thawing Index. The Thawing Index period for much of the U.S. will start in late February of April

Figure 1. Worksheet for Bituminous Surfaced Pavements (Cont.)

Day	Month/ Year	Measured Daily Air Temperature (^oF)		Average Daily Air Temperature, (^oF)	Daily Freezing Index = 32° F - Avg. Daily Temp. (see note) (^oF - days)	Sum of Daily Freezing Index (^oF-days)	Daily Thawing Index = Avg. Daily Temp. - 29° F. (see note) (^oF - days)	Sum of Daily Thawing Index (^{o} F-days)
		High	Low	(High + Low)/2				
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
1	Feb., '86	36	32	34	-2	1161	5	10
2	" "	40	26	33	-1	1160	4	14
3	" "	34	30	32	0	1160	3	17
4	" "	38	30	34	-2	1158	5	22
5	" "	37	27	32	0	1158	3	25
6	" "	36	25	30	2	1160	1	26
7	" "	40	18	29	3	1163	0	26
8	" "	32	10	21	11	1174	-8	18
9	" "	32	20	26	6	1180	-3	15
10	" "	28	16	22	10	1190	-7	8
11	" "	30	14	22	10	1200	-7	1
12	" "	32	15	24	8	1208	-5	--
13	" "	32	7	20	12	1220		
14	" "	29	11	20	12	1232		
15	" "	30	15	22	10	1242		
16	" "	29	12	20	12	1254		
17	" "	30	11	20	12	1266		
18	" "	29	15	22	10	1276		
19	" "	30	-8	11	21	1298		
20	" "	23	-10	6	26	1323		
21	" "	21	-6	8	24	1347		
22	" "	28	0	14	18	1365		
23	" "	34	10	22	10	1375		
24	" "	45	32	38	--	--	9	9
25	" "	48	38	43			14	23
26	" "	48	32	40			11	34
27	" "	48	24	36			7	41
28	" "	48	26	37			8	49

Note: Calculate Daily Freezing Index starting at the beginning of the freezing season and accumulate throughout the normal freezing period. The air temperature datum for Freezing Index is 32^oF and 29^oF for Thawing Index. The Thawing Index period for much of the U.S. will start in late February of April

Figure 1. Worksheet for Bituminous Surfaced Pavements (Cont.)

Day	Month/ Year	Measured Daily Air Temperature (^oF)		Average Daily Air Temperature, (^oF)	Daily Freezing Index = 32° F - Avg. Daily Temp. (see note) (^oF - days)	Sum of Daily Freezing Index (^oF-days)	Daily Thawing Index = Avg. Daily Temp. - 29° F. (see note) (^oF - days)	Sum of Daily Thawing Index (^{o} F-days)
		High	Low	(High + Low)/2				
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
1	Mar., '86	43	25	34			5	54
2	" "	48	22	35			6	60
3	" "	40	20	30			1	61
4	" "	46	24	35			6	67
5	" "	47	21	34			5	72
6	" "	46	24	35			6	78
7	" "	47	29	38			9	87
8	" "	49	30	40			11	98
9	" "	50	30	40			11	109
10	" "	52	31	42			13	122
11	" "	40	33	36			7	129
12	" "	52	32	42			13	142
13	" "	52	25	38			9	151
14	" "	45	20	32			3	154
15	" "	53	26	40			11	165
16	" "	53	26	40			11	176
17	" "	54	26	40			11	187
18	" "	54	32	43			14	201
19	" "	56	31	44			15	216
20	" "	57	32	44			15	231
21	" "	62	35	48			19	250
22	" "	50	32	41			12	262
23	" "	47	34	40			11	273
24	" "	54	30	42			13	286
25	" "	50	30	40			11	297
26	" "	52	34	43			14	311
27	" "	52	36	44			15	326
28	" "	58	42	50			21	347
29	" "	58	41	50			21	368
30	" "	59	40	50			21	389
31	" "	60	43	52			23	412

Note: Calculate Daily Freezing Index starting at the beginning of the freezing season and accumulate throughout the normal freezing period. The air temperature datum for Freezing Index is 32°F and 29°F for Thawing Index. The Thawing Index period for much of the U.S. will start in late February of April

Figure 1. Worksheet for Bituminous Surfaced Pavements (Cont.)

SECTION 3.0

PAVEMENT EVALUATION

This SECTION is used to describe the kinds of information currently available to evaluate pavement structures. Specific reference is made to the equipment and data used by WSDOT.

1. INTRODUCTION

1.1 NEED FOR PAVEMENT CONDITION DATA

The following provides a brief overview about the types and uses of pavement condition data for various time periods (after Hicks and Mahoney [3.1]).

1.1.1 THE PAST

Pavement condition data have been used in the past to develop maintenance, rehabilitation, and reconstruction programs, generally on a project-by-project basis. The data were used to determine the projects requiring maintenance and the type of maintenance or rehabilitation required to correct the observed deficiencies.

1.1.2 PERIOD OF 1940S AND 1950S

During this period highway maintenance personnel relied heavily on visual inspections to establish type, extent, and severity of distress, and on experience or judgment to establish maintenance programs. Unfortunately, experience is difficult to transfer from one person to another, and individual decisions made from similar data are often inconsistent.

1.1.3 PERIOD OF LATE 1950S AND EARLY 1960S

During this period the increased use of roughness meters and deflection and skid test equipment permitted objective data to be collected and used both alone and with visual distress surveys to aid in making maintenance and rehabilitation decisions.

1.1.4 PERIOD OF 1970S AND EARLY 1980S

During this period highway personnel could no longer rely on the luxury of managing roadways solely on the basis of field personnel experience. Because of limited resources, it was essential to develop rapid, objective means to establish:

(a) Projects in need of maintenance or rehabilitation.

(b) Types of maintenance or rehabilitation currently required.

(c) Types and schedule of maintenance or rehabilitation to be undertaken in the future to minimize life-cycle costs (construction, maintenance, and user costs) or to maximize the net benefit.

1.1.5 PRESENT

At present, three specific applications for pavement condition data can be identified.

(a) Establish priorities

Condition data such as ride (profile), distress, and deflection are used to establish the projects most in need of maintenance and rehabilitation. Often only ride and/or distress data are used; at other times ride, distress, and deflection data are combined into a single rating. Skid resistance data are often used separately. Once identified, the projects in the poorest condition (low rating) will be more closely evaluated to determine repair strategies. (Note: WSDOT up until 1994 used a "worst first"approach in the WSPMS.)

(b) Establish maintenance and rehabilitation strategies

Data from visual distress surveys are used to develop an action plan on a year-to-year basis; i.e., which strategy (repairs, surface treatments, overlays, recycling, etc.) is most appropriate for a given pavement condition.

(c) Predict pavement performance

Data, such as ride, skid resistance, distress, or a combined rating, are projected into the future to assist in preparing long-range budgets or to estimate the condition of the pavements in a network given a fixed budget.

1.2 BENEFITS OF PAVEMENT EVALUATION

(a) Allocation of maintenance and rehabilitation funds.

(b) Determination of structural adequacy.

(c) Indication of highway network pavement condition and performance (city, county, state).

(d) Measure of "year-to-year" differences in pavement condition and performance.

(e) Overview of current practices.

2. TYPES OF DATA COLLECTED

2.1 GENERAL

(a) Roughness (profile or ride)
(b) Surface distress
(c) Structural evaluation (surface deflection)
(d) Skid resistance

3. ROUGHNESS (PROFILE OR RIDE)

3.1 DEFINITIONS

3.1.1 AASHO ROAD TEST

Pavement roughness is generally defined as irregularities in the pavement surface that adversely affect the ride quality of a vehicle (and thus the user). The activities associated with the AASHO Road Test [3.3] produced a more precise definition of roughness that is still used by some agencies at this time.

(a) Present Serviceability Rating (PSR)

"The judgment of an observer as to the current ability of a pavement to serve the traffic it is meant to serve." The quantitative scale used by the panels (observers) in the AASHO Road Test is shown in Figure 3.1. The subjective scale ranges from 5 (excellent) to 0 (essentially impassable).

(b) Present Serviceability Index (PSI)

"An estimate of the mean serviceability ratings made by a panel of judges." Usually this estimate is obtained with some type of equipment that is correlated to panel ratings.

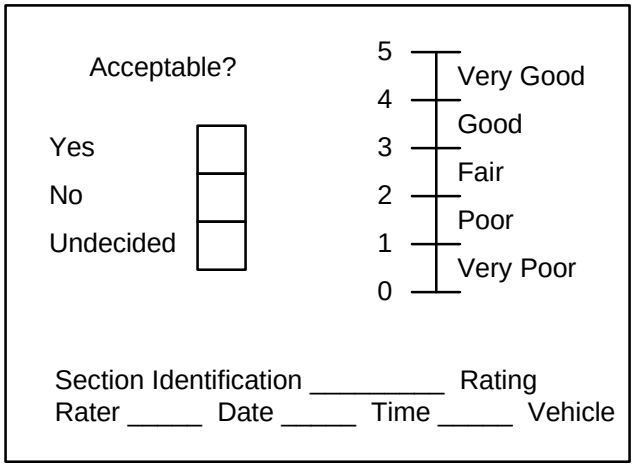


Figure 3.1. Individual Present Serviceability Rating Used for the AASHO Road Test [4.2]

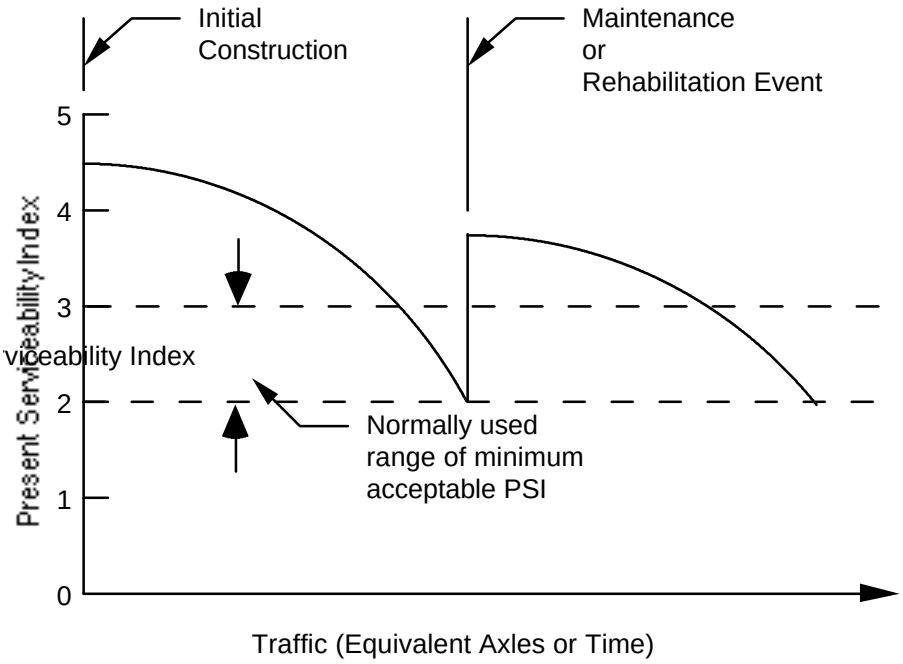


Figure 3.2. Concept of Pavement Performance Using Present Serviceability Index [4.1]

(c) Performance

> "The serviceability trend of a ... [pavement segment] with increasing number of axle applications." Figure 3.2 further demonstrates this concept.

The original development of the pavement serviceability–performance concept was reported by Carey and Irick [3.32]. They noted in their 1960 Highway Research Board paper:

> "… design systems in general use in highway departments do not include consideration of the level of performance desired. Design engineers vary widely in their concepts of desirable performance. By way of example, suppose that two engineers were given the task of designing a pavement of certain materials for certain traffic and environment for 20 years. The first might consider this job to be properly done if not a single crack occurred in 20 years, whereas the second might be satisfied if the last truck that was able to get over the pavement made its trip 20 years from the date of construction."

To transition from a PSR serviceability measure (panel developed) to a PSI serviceability measure (no panel required), a panel of raters during 1958 to 1960 rated various roads in the states of Illinois, Minnesota, and Indiana for PSR. This information was then correlated to various pavement measurements (such as slope variance (profile), cracking, etc.) to develop PSI equations. These equations are shown in SECTION 5.0, Paragraph 1.3.3 for flexible pavements, and SECTION 6.0, Paragraph 1.2 for rigid pavements. Further, the raters were asked to provide an opinion as to whether a specific pavement assessed for PSR was "acceptable" or "unacceptable" as a primary highway (see Figure 3.1). The cumulative frequency plots developed by Carey and Irick reveal the following:

	"Acceptable"		"Unacceptable"	
PSR	Flexible	Rigid	Flexible	Rigid
4.5	100%	100%	0%	0%
4.0	100%	99%	0%	0%
3.5	95%	88%	0%	1%
3.0	55%	55%	10%	15%
2.5	17%	20%	50%	55%
2.0	3%	3%	84%	88%
1.5	0%	1%	100%	98%

From the above, about one-half of the panel of raters found a PSR of 3.0 acceptable and a PSR of 2.5 unacceptable. Such information was useful in selecting a "terminal" (or failure) serviceability (PSI) design input in SECTIONS 5.0 and 6.0.

3.1.2 SUBSEQUENT DEFINITIONS

Since the 1950s and 1960s, the equipment and associated measures of road roughness have continued to evolve. Some of the "newer" roughness measures [3.20] include the following:

(a) RTRRMS

Response-type road roughness measuring system

(b) International Roughness Index (IRI)

The IRI is used to define a characteristic of the longitudinal profile of a traveled wheeltrack and now constitutes a standardized roughness measurement. The commonly recommended units are meters per kilometer (m/km) or millimeters per meter (mm/m). The IRI is equal to "slope" multiplied by 1,000. The "slope" is specifically defined by the average rectified slope (ARS). The open-ended IRI scale is shown in Figure 3.3. The reference RTRRMS used for the IRI is a mathematical model, rather than a mechanical system. Thus, the reference RTRRMS is computed from a measured profile. The computation is referred to as a "quarter-car simulation." Thus, the mathematical model presents a RTRRMS having a single wheel.

Sayers et al. [3.20] noted that IRI levels would present the following "ride" at 80 km/h (49 mph):

IRI Range	Description
1.3–1.8	"Undulation barely perceptible"
4.0–5.3	"Moderately perceptible movements or large undulations may be felt"

If, for example, an IRI of 4.0 is judged to be "unacceptable," then using Equation 3.1 (Paragraph 3.2.5 this SECTION) results in a corresponding PSR of 2.4. This level of PSR would be "acceptable" to about 15 percent of the original pavement raters associated with the development of the serviceability–performance concept used at the AASHO Road Test. About 60 percent of those raters would have rated pavements with a PSR of 2.4 as "unacceptable." Given that trucks are more severely impacted by rough pavements than autos, an "unacceptable" range of IRI is more likely 3.0 to 4.0 (a PSR of 2.4 to 2.9).

(c) Average Rectified Slope (ARS)

The ARS is a ratio of the accumulated suspension of a vehicle (in., mm, etc.) divided by the distance traveled by the vehicle during the measurement (mi, km, etc.).

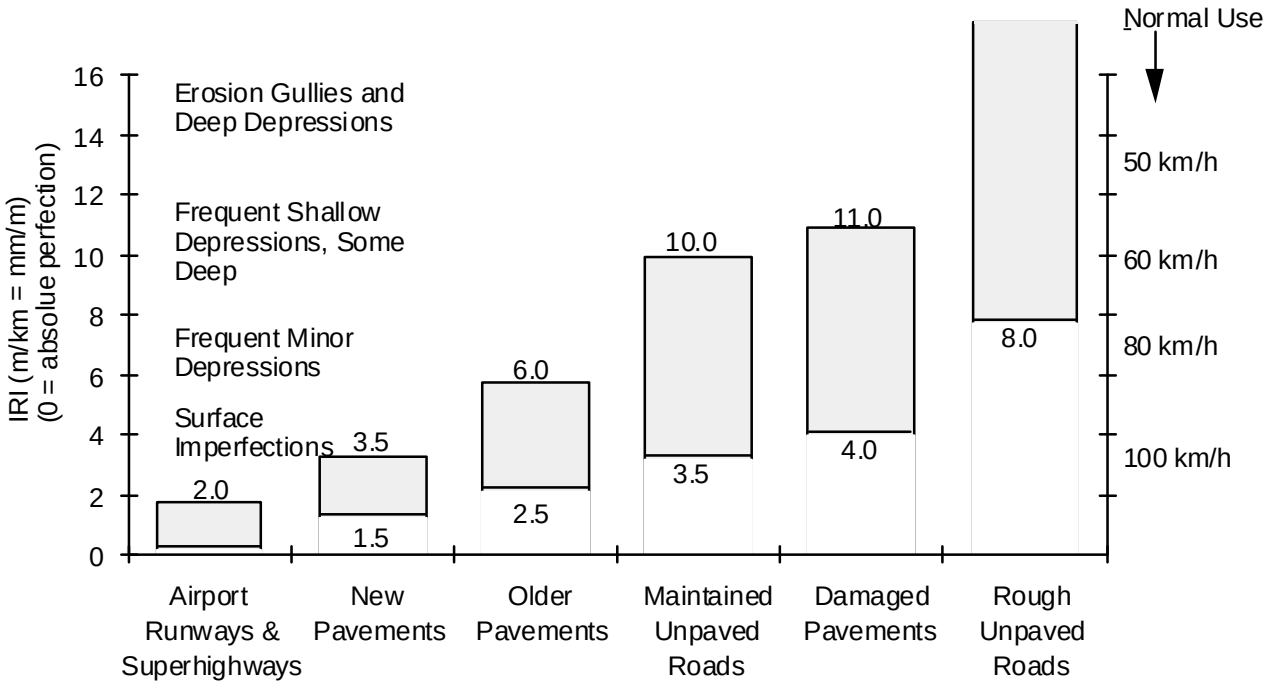


Figure 3.3. IRI Roughness Scale [Replotted from Ref. 3.20]

3.2 GENERAL

The equipment for roughness survey data collection can be categorized into four broad categories:

Equipment/Technique	Complexity
• Rod and level survey (includes Dipstick Profiler) • Profilographs • Response type road roughness meters (RTRRMs) • Profiling devices	(-) ↓ (+)

The following discussion with a few modifications was taken directly from Reference 3.21.

One of the most important considerations in selection of a roughness measuring device is the tradeoff between the relatively low initial and data collection costs of devices such as an RTRRM, versus the frequent need to calibrate the device. Other tradeoffs must also be carefully considered.

3.2.1 ROD AND LEVEL SURVEY AND THE DIPSTICK PROFILER

Rod and level surveys can provide an accurate measurement of the pavement profile. The use of the rod and level survey for network or even large project survey data, however, is impractical and cost prohibitive. A first-step automation of the rod and level survey which may be used to collect a relatively small quantity of pavement profile measurements is through the use of the Dipstick Profiler.

The Dipstick Profiler consists of an inclinometer enclosed in a case supported by two legs separated by 305 mm (12 in.). Two digital displays are provided, one at each end of the instrument. Each display reads the elevation of the leg at its end relative to the elevation of the other leg. The operator then "walks" the dipstick down a premarked pavement section by alternately pivoting the instrument about each leg. Readings are recorded sequentially as the operator traverses the section. The device records 10 to 15 readings per minute. Software analysis provides a profile accurate to ± 0.127 mm (± 0.005 in.).

A common application for the dipstick is to measure the profile for the calibration of RTRRMs. A strip can be surveyed by a single operator in about one-half the time of a traditional survey crew.

3.2.2 PROFILOGRAPHS

3.2.2.1 Introduction

Profilographs have been available for many years and exist in a variety of different forms, configurations, and brands. Due to their design they are not suitable for condition surveys. Their most common use today is for portland cement concrete pavement construction inspection, control, and acceptance. The major differences among the various profilographs involve the configuration of the wheels and the operation and measurement procedures of the various devices.

Profilographs have a sensing wheel, mounted to provide for free vertical movement at the center of the frame. The deviations against a reference plane, established from the profilograph frame, is recorded (automatically on some models) on graph paper from the motion of the sensing wheel. Profilographs can detect very slight surface deviations or undulations up to about 6 m (20 ft) in length.

WSDOT uses the California Profilograph to check construction related profiles (in accordance with WSDOT Test Method No. 807). The profilograph requirement only applies to PCCP. The 1991 WSDOT Standard Specifications requires the following for PCCP surface smoothness:

- A daily profile index will be computed on PCCP paving projects. If the lane is 3.6 m (12 ft) wide (or less), the daily profile index will be the average of two profiles approximately 0.9 m (3 ft) from and parallel to each pavement lane edge. The daily profile index of the finished pavement will be 7 in. per mile, or less.

- All pavement areas with "high point" deviations in excess of 0.3 in. "… shall be reduced by abrasive means until such deviations do not exceed 0.3 in. as determined by reruns of the profilograph."

- For small or irregular PCCP areas, surface smoothness will be measured with a 3 m (10 ft) long straightedge both parallel and transverse to the centerline.

An overview of national profilograph use and the origin of the above specification requirements follows in separate subsections.

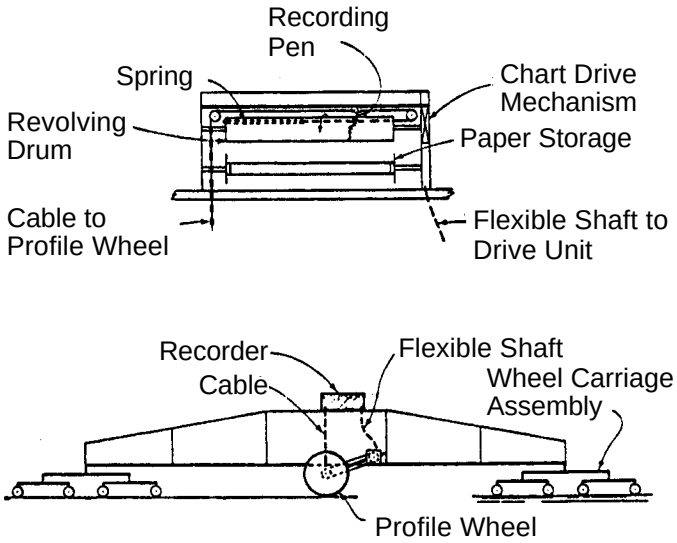
3.2.2.2 National Profilograph Survey

A 1992 survey of SHAs by Scofield [3.31] provides insight into the national use of profilograph equipment. The results are based on the 36 SHAs which responded. The California type profilograph was used by

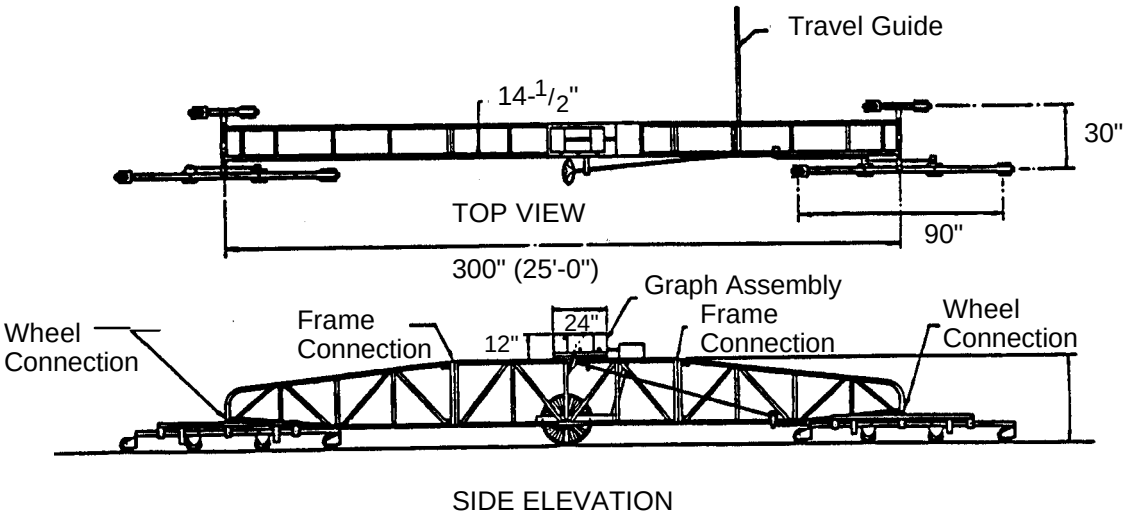
- 15 SHAs (42 percent of responses) for AC pavements,
- 19 SHAs (53 percent of responses) for PCC pavements.

3.2.2.3 WSDOT Profilograph Specification

The WSDOT California Profilograph Specification is essentially the same as originally developed by the California Division of Highways during the late 1950s (see Scofield [3.31]). To understand this more fully, the sketch of the California Profilograph (taken from WSDOT Test Method No. 807) is shown below.



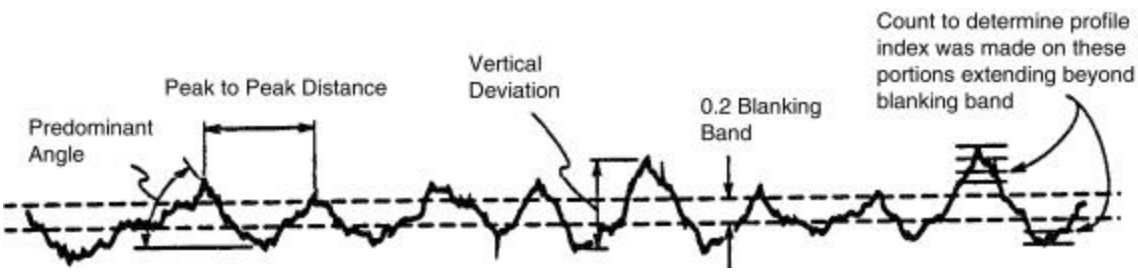
A dimensioned sketch of the California Profilograph (from Scofield [3.31] and shown below) shows a 7.6 m (25 ft) long "reference beam" with the "measuring wheel" located at mid-length. The profile trace is recorded at a horizontal scale of 1:300 and a 1:1 vertical scale.



The California Division of Highways used a 10 ft long profilograph up to 1955, then the 7.6 m (25 ft) length thereafter. It is noted by Scofield [3.31] that the profilograph wheelbase should be longer than PCCP slab lengths so that the results are not influenced by slab tilting.

The Profile Index (used by WSDOT) was developed by the California Division of Highways starting in 1956. The goal was to develop a relationship between a calculated index and a subjectively obtained panel rating of road roughness. Both flexible and rigid pavements were surveyed (total of 100 km (62 miles): PCCP 51 km (32 miles) and AC 49 km (30 miles)) and ranged in roughness from 2 to 90 in. per mile (Profile Index Measure).

The Profile Index was developed after much trial and error and was based on 0.2 in. "blanking band" as illustrated below.



According to Scofield [3.31], the use of a blanking band significantly reduced the trace reduction effort since all "scallops" (deviations or excursions of roughness above or below the blanking band) would not have to be analyzed. The 0.2 in. blanking band was selected (0.1 and 0.3 in. bands were also tried) since it produced an index which identified the rough pavements in the California study and minimized the data reduction effort. Further, during this effort, it was noted that rough roads showed scallops with ordinates over 3/8 in. The blanking zone eliminated those scallops which little influenced perceived pavement roughness. It was noted by Scofield that it

> "… was recognized at this time (1957) that the blanking band precluded faulting up to 0.2 in. Although considered annoying, the vibration caused by this level of faulting did not create accelerations large enough to produce discomfort to the passengers."

Scofield [3.31] notes that the 1956 California study found that pavements with a Profile Index of 10 or less were almost always considered smooth and those above 40 almost always rough. A Profile Index of 7 in. per mile was reached by a consensus of California "… Traffic, District Construction, and Materials Laboratory personnel, as well as construction industry representatives." It was noted then that "… any contractor could construct to a PRI [or Profile Index] of 7 since most already constructed to 5 or less."

California first used the specification in 1958 and it was integrated into their Standard Specifications in 1960.

The 0.3 in. "must grind" requirement also originated in California. Scofield noted that during

> "… development of the ride specification, concern existed that a single large bump could still exist within the newly developed specification requirements. To alleviate this concern the 0.3 in. per mile must grind specification was included."

WSDOT Test Method No. 807 basically describes a manual data reduction process whereby the profilogram trace with deviations above the 0.2 in. blanking band are recorded in 0.1 in. increments. The data reduction is done in units of 0.1 mile, then totaled, if required, for an in. per mile basis. For additional details, consult the WSDOT test method.

As a final note, Scofield's [3.31] 1993 NCHRP report listed profilograph prices. A Cox automated California style profilograph was listed with a cost of $23,910.

3.2.2.4 PSI vs. Profile Index

Scofield [3.31] summarized several studies which related roughness measures such as PSI to Profile Index. Using the California Profilograph and the 0.2 in. blanking band, one study of PCCP showed

Pavement Serviceability Index	Profile Index (in. per mile)
4.32	2.0
4.24	4.0
4.14	6.0
4.04	8.0
3.95	10.0
3.86	12.0
3.77	14.0
3.68	16.0
3.58	18.0
3.48	20.0
3.39	22.0
3.29	24.0

Thus, for every 2 in. per mile increase in Profile Index, the PSI decreases about 0.1. Using the current specification limit of 7 in. per mile following construction results in a PSI of about 4.1. This is likely lower than that assumed for newly constructed PCCP (refer to SECTION 6.0).

3.2.3 RESPONSE TYPE ROAD ROUGHNESS METERS

The third category of roughness data collection equipment is the response type road roughness meters. This category includes such devices as the BPR Roughometer (Figure 3.4), the Mays Ride Meter, and the PCA Road Meter. RTRRMs have been used for a number of years and are currently the most widely used roughness data collection device in the U. S. Their primary use is for network-level roughness data collection.

Road meters or RTRRMs measure the vertical movements of the rear axle of an automobile or the axle of a trailer relative to the vehicle frame. The meters are installed in vehicles with a displacement transducer on the body located between the middle of the axle and the body of a passenger car or trailer. The transducer detects small increments of axle movement relative to the vehicle body. The output data consists of a strip chart plot of the actual axle body movement versus the time of travel.

Several standard calibration procedures have been developed for the RTRRMs which are in use today. Careful operating and maintenance procedures should be followed, including frequent and precise calibrations, in order to improve device accuracy and consistency. The degree of accuracy desired in the calibration of RTRRMs ultimately depends upon the proposed use of the data being collected.

RTRRM systems are adequate for routine monitoring of a pavement network and providing an overall picture of the condition of the network. The output can provide managers with a general indication of the overall network condition and maintenance needs.

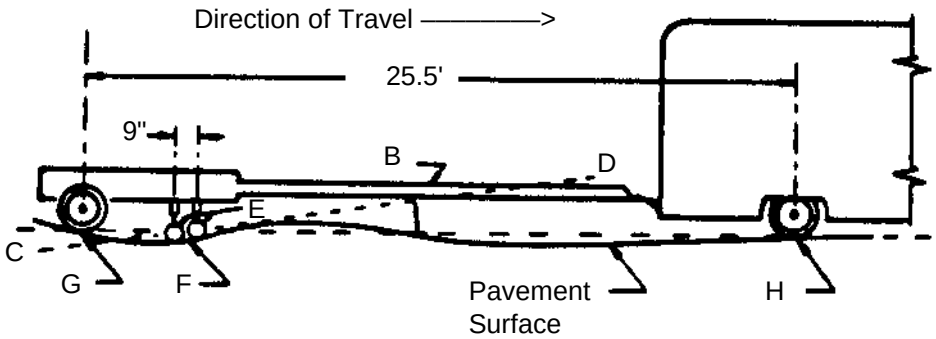


Figure 3.4. CHLOE Profilometer [3.2]

3.2.4 PROFILING DEVICES

Profiling devices are used to provide accurate, scaled, and complete reproductions of the pavement profile within a certain range. They are available in several forms, and can be used for calibration of the RTRRMs. The equipment is fairly expensive, with complexity increasing depending on the types and number of transducer sensors contained on board. Three generic types of profiling systems are in use today:

- straight edge,
- low speed systems, and
- inertial reference systems.

The simplest profiling system is the straight edge. Modifications to the straight edge, such as mounting it on a wheel, results in a profilograph. Low speed systems such as the CHLOE profilometer (Figure 3.4) are moving reference planes that have little or no dynamic effect due to their low speed. The CHLOE is a long trailer that is towed at low speeds of 3 to 8 kph (2 to 5 mph). The slow speed is necessary to prevent any dynamic response measurement during the readings. The device measures the difference in slope between a small arm with two wheels and a trailer frame with two larger wheels. A few agencies still use the CHLOE to calibrate their RTRRMs.

Most sophisticated road profiling equipment uses the inertial reference system. The profiling device measures and computes longitudinal profile through the creation of an inertial reference by using accelerometers placed on the body of the measuring vehicle to measure the vehicle body motion. The relative displacement between the accelerometer and the pavement profile is measured with either a "contact" or a "non-contact" sensor system.

The earliest profiling devices used a measurement system in direct contact with the pavement to measure profile. Several contact systems have been used, and are still in use today. The French Road Research Laboratory developed the Longitudinal Profile Analyzer (APL) in 1968.

Systems used today in the United States are frequently installed in vans which contain microcomputers and other data handling and processing instrumentation. Older profiling devices are usually contact systems, while the more recently manufactured devices use non-contact sensors. The non-contact systems use probes, either acoustic or light, to measure differences in the pavement surface.

Profiling devices are capable of providing highly accurate roughness data and scaled reproductions of the pavement profile.

A summary of the most commonly used roughness data collection devices, their measurement principles, relative costs, relative degrees of accuracy, and current and projected future use is contained in Table 3.1.

Table 3.1. Roughness Data Collection Equipment [from Reference 3.21]

Roughness Data Collection Device	Principle of Measurement	Relative Initial Cost	Relative Data Collection Cost (Network)	Relative Degree of Accuracy	Approximate Number of Years in Use	Extent of Current Use	Projected Extent of Use Year 1990 - 2000
Dipstick	Direct Differential Measurement	Low	Impractical	Very High	8	Limited, Used for Calibration	Same as Current Use
Profilographs	Direct Profile Recordation	Low	Impractical	Medium	30	Extensive for Const. Acceptance	Same as Current Use
BPR Roughometer	Device Response	Low	Low	Medium	50	Limited	None
Mays Meter	Vehicle Response	Low	Low	Medium	30	Extensive	Decreasing Continuously
South Dakota Road Profiler	Direct Profile Recordation	Medium	Low	High	7	Growing	Rapidly Increasing
Contact Profiling Device	Direct Profile Recordation	High	Medium	Very High	20	Limited	Decreasing
Non-Contact Profiling Device	Direct Profile Recordation	Very High	Medium	Very High	8	Medium	Increasing Continuously

3.2.5 CORRELATIONS BETWEEN PSR AND IRI

Various correlations have been developed between PSR and IRI. Two are presented here. The first one was reported in 1987 by Paterson [3.23]:

$$PSR = (5)e^{((-0.18)(IRI))} \qquad \text{(Eq. 3.1)}$$

where PSR = Present Serviceability Rating, and

IRI = International Roughness Index (m/km).

The data used to generate Equation 3.1 came from Texas, Pennsylvania, South Africa, and Brazil. No regression statistics were provided by Paterson [3.23].

The second correlation was reported in a 1992 Illinois funded study performed by Al-Omari and Darter [3.24]:

$$PSR = (5)e^{((-0.26)(IRI))} \quad \text{(Eq. 3.2)}$$

where PSR = Present Serviceability Rating, and

IRI = International Roughness Index (mm/m or m/km).

The Illinois study used data from the states of Indiana, Louisiana, Michigan, New Mexico, and Ohio for both flexible and rigid pavements. The associated regression statistics for Equation 3.2 are $R^2 = 0.73$, SEE = 0.39, and n = 332 sections.

As an illustration of the use of Equation 3.1, a few IRI measures obtained on I-5 and contained in the 1993 version of the WSPMS will be used.

IRI Condition	Calculated PSR (Equation 3.1)
• Minimum IRI = 1.02 (MP 85.51–88.30 (L))	4.2
• Maximum IRI = 3.92 (MP 276.20–276.56 (L))	2.5
"Typical" IRI = 2.0	3.5

Using the opinions provided by the panel of raters originally used to develop the PSR vs. PSI relationship for the AASHO Road Test (refer to Paragraph 3.1.1, this SECTION), the following results:

- About 100 percent would find an IRI of 1.02 (PSR ≃ 4.2) acceptable and 0 percent unacceptable.
- About 90 percent would find an IRI of 2.00 (PSR ≃ 3.5) acceptable and less than 1 percent unacceptable.
- Only 20 percent would find an IRI of 3.92 (PSR ≃ 2.5) acceptable and about 50 percent unacceptable.

The AASHO Road Test panel of raters' opinions "suggests" that a PSR of 3.0 or greater is a minimum level to maintain primary highways (corresponds to an IRI of 2.9). Being as the raters' opinions and the IRI is based on car ride dynamics, it is unclear whether such levels are acceptable for trucks.

3.3 SPECIFIC ROUGHNESS MEASURING EQUIPMENT

3.3.1 RESPONSE-TYPE EQUIPMENT [3.3]

This type of equipment provides a relative measure of road roughness. The mechanical measuring system responds to the road surface and does not provide a precise road profile. Testing is normally conducted at a fixed speed. A partial list of this type of equipment includes:

3.3.1.1 BPR Roughmeter

(a) First developed in 1920s

(b) Simulates one wheel of a passenger car

(c) Data recorded at 32 kph (20 mph)

(d) Records inches of vertical movement of wheel axle relative to suspension frame

(e) Apparatus: trailer and car

(f) Output: Roughness Index (inches per mile)

(g) Disadvantages

(i) Low operating speed
(ii) Repeatability problems
(iii) Traffic hazard

3.3.1.2 Car Ride Meters

(a) Various instruments have been available, including those from PCA, Mays, and Cox. The Cox Ride Meter was used by WSDOT for many years up to the early 1990s.

(b) Devices primarily developed during the 1960s and 1970s.

(c) Commonly used at this time.

(d) Output: vertical deviations between the body of automobile (or trailer) and the rear axle.

(e) Advantages

(i) Relatively low cost, simplicity and ease of operation.
(ii) Operating speed generally 80 kph (50 mph).
(iii) Acquire large amounts of data.
(iv) Adequate repeatability (short term).
(v) Output of inches per mile can be correlated with PSI.

(f) Disadvantages

(i) Inability to measure profile or long-wave lengths (expansive clay).
(ii) Relatively frequent calibration for PSI determination.
(iii) Operating characteristics of auto (for auto mounted instruments)

- Tire pressure
- Shocks
- Temperature and wind
- Tire wear
- Auto cargo and distribution

3.3.2 PROFILING EQUIPMENT

This kind of equipment is used to obtain a direct measure of the pavement profile (as compared to response-type equipment which is a measure of a "vehicle's" response to the road profile).

3.3.2.1 Straight Edges or Rod and Level Surveys

3.3.2.2 CHLOE Profilometer

(a) Developed at the AASHO Road Test.

(b) Roughness measured by change in angle between two reference lines (refer to Figure 3.4). The short reference line is defined by the two small wheels (E) which are 229 mm (9 in.) apart. The long reference line between wheels G and H is 7.8 m (25.5 ft.) and assumed to be parallel to pavement surface. Thus, the angle A between CD and GH represents the deviation of pavement profile (line CD) from the "reference" (line GH).

(c) Output: slope variance (after Reference 3.2)

$$SV = \frac{\sum (x_i - \bar{x})^2}{n-1}$$

where SV = slope variance,

x_i = i^{th} slope measurement as measured by angle A (see Figure 3.4)

$\bar{x}$ = mean (or average),

n = total number of measurements.

(d) Disadvantages

(i) Low operating speed (5 to 8 kph (3 to 5 mph))
(ii) High maintenance
(iii) Traffic hazard

3.3.2.3 Surface Dynamics Profilometer

(a) Originally called GM Profilometer.

(b) Developed in early 1960s

(c) Substantial development and evaluation work performed in State of Texas during 1960s and 1970s.

(d) Apparatus mounted in van

(e) Output: direct measurement of road profile.

(f) Advantages

(i) Measurement of actual road profile
(ii) Operating speed up to 32 kph (20 mph)
(iii) Capable of measuring long roughness wave lengths
(iv) Has been used to calibrate car ride meters (Texas)

(g) Disadvantages

(i) High capital cost ($200,000 ±)
(ii) High operating costs
(iii) Requirement for skilled personnel due to complexity
(iv) Traffic hazard

3.3.2.4 South Dakota Road Profiler [3.14]

(a) Originally developed in 1981 by the South Dakota Department of Transportation, Pierre, South Dakota. Currently manufactured by a private corporation in Florida (January 1992). This device is used by WSDOT as of 1992.

(b) Can be used to measure rutting, road profile, and associated roughness.

(c) Uses ultrasonic sensors and accelerometer to measure the pavement profile. Instrumentation installed in standard sized van or auto.

(d) Advantages

(i) Measurement of actual road profile and rutting at normal highway speeds. Can survey over 170 km (100 mi.) or more per day.

(ii) Modest operating costs

(e) Disadvantages

(i) Current accuracy may not provide adequate measures of PCC slab faulting
(ii) Initial cost ($60,000 (±))

WSDOT uses the South Dakota Profilometer for collection of longitudinal profile (IRI) and pavement rutting. WSDOT's Profilometer incorporates three ultrasonic sensors and two accelerometers. Interstate pavements are tested annually and all remaining pavements every two years. The Northwest, Olympic and South Central Regions are tested in odd numbered years, while North Central, Southwest and Eastern Regions are tested in even numbered years. Divided highways are tested in both directions.

3.4 STATE USAGE OF ROUGHNESS MEASURING EQUIPMENT

As shown in Table 3.2, it is clear that most SHAs are converting to profile measuring equipment (likely due to FHWA requirements for measurement of IRI for HPMS sites).

Table 3.2 State Usage of Roughness Measuring Equipment [after Reference 3.14]

Device	Number of SHAs		
	Mid 1980s	1990	Estimated 1995
BPR Roughometer	5	1	0
Mays Meter	23	16	5-10
Cox RTRRM	6	7	2-5
ARAN	0	7	5-10
Other RTRRM	4	8	0-5
KJ Law 69ODNC/8300	4	7	3-8
SD Road Profiler	1	12	20-30
Laser or Own Profilometer	2	4	3-5
TOTAL — RTRRM	38	38	15-20
TOTAL — PROFILE	7	23	25-35

4. SURFACE DISTRESS

4.1 DEFINITION OF SURFACE DISTRESS

"Any indication of poor or unfavorable pavement performance or signs of impending failure; any unsatisfactory performance of a pavement short of failure" [3.5].

4.2 COMPONENTS OF SURFACE DISTRESS

Table 3.3 shows typical distres modes and causative mechanisms. For example, cracking can be caused by at least six different mechanisms ranging from excessive loading to shrinkage. More broadly, all distress modes (or types) fall into three groups: fracture, distortion, and disintegration.

4.3 HOW IS SURFACE DISTRESS MEASURED?

4.3.1 SUBJECTIVE MEASURES

At this time, measures of distress are usually subjectively obtained; however, distress detection equipment is being more commonly used. For these subjective measurements, pavement, in lengths ranging from 30 to 150 m (100 to 500 ft.) is usually surveyed at 1.6 km (1 mi.) intervals. Research conducted in Texas verifies this approach [3.7].

4.3.2 WSDOT PRACTICE

The recording form currently used by the Washington State Department of Transportation (WSDOT) is shown in Figure 3.5. The form is primarily used to record the location, extent, and severity of pavement distress (the specifics for conducting a WSDOT surface condition survey are contained in the document "Pavement Surface Condition Rating Manual" [3.29]). All types of distress measurements used for both flexible (bituminous) and rigid (portland cement concrete) pavements are listed in this single form. A column for recording roughness counts obtained from the ride meter formerly used in Washington is also included in the form. Other agencies have also developed forms to accommodate specific evaluation procedures. The agencies use various methods to condense distress data into useful information. One common procedure is to associate deduct (penalty) points with specific distress type, severity, and extent combinations. These points can then be summed and subtracted from some upper limit or maximum value (usually 100). A generalized relationship for this concept was described by Shahin and Darter [3.8]:

Table 3.3. Distress Groups [3.6]

Distress Group	Distress Mode	Examples of Distress Mechanism
Fracture	Cracking	Excessive loading
		Repeated loading (i.e., fatigue)
		Thermal changes
		Moisture changes
		Slippage (horizontal forces)
		Shrinkage
	Spalling	Excessive loading
		Repeated loading (i.e., fatigue)
		Thermal changes
		Moisture changes
Distortion	Permanent Deformation	Excessive loading
		Time-dependent deformation (e.g., creep)
		Densification (i.e., compaction)
		Consolidation
		Swelling
		Frost
	Faulting	Excessive loading
		Densification (i.e., compaction)
		Consolidation
		Swelling
Disintegration	Stripping	Adhesion (i.e. loss of bond)
		Chemical reactivity
		Abrasion by traffic
	Raveling and Scaling	Adhesion (i.e. loss of bond)
		Chemical reactivity
		Abrasion by traffic
		Degradation of aggregate
		Durability of binder

AR 4

Washington State Department of Transportation

PAVEMENT CONDITION RATING

DATE | RATER | SHEET OF

CONTROL DATA									BITUMINOUS PAVEMENTS						PORTLAND CEMENT CONCRETE								
STATE ROUTE	REL. RDWY TYPE	REL. RDWY QUAL.	CONTROL SECTION	DISTRICT	ENDING ACCUMULATED STATE ROUTE MILEAGE FOR RATED SECTION	PAVEMENT TYPE "A", "B", "C"	*PAVEMENT EXEMPTION	MULTI-LANE (L), (R), (B)	RUTTING PAVT. WEAR **	ALLIGATOR CRACKING (1) Hairline (2) Spalling (3) Spalling & Pumping: 1-9% WHL. TRK. LEN. / 10-24% WHL. TRK. LEN. / 25-49% WHL. TRK. LEN. / 50-100% WHL. TRK. LEN.	RAVELING OR FLUSHING (1) Slight (2) Moderate (3) Severe: LOCALIZED / WHEEL PATHS / ENTIRE LANE / R - RAVELING F - FLUSHING	LONGITUDINAL CRACKING Percent of Sect. Len. (1) 1-99% (2) 100-199% (3) 200% or More: LESS THAN 1/4" / OVER 1/4 WIDE / SPALLED	TRANSVERSE CRACKING Count per 100 FL (1) 1-4 (2) 5-9 (3) 10 or More: LESS THAN 1/4" / OVER 1/4 WIDE / SPALLED	PATCHING Percent of WHL. TRK. LEN. (1) 1-9% (2) 10-24% (3) 25% or More: BST (Chir Seal) / Blade (or Cold) / ACP (Full Depth)	CRACKING Cracks Per Panel (1) 1 (2) 2-3 (3) 4 or More: 1-9% PANELS / 10-24% PANELS / 25% or More PANELS	RAVELING SCALING (1) Slight (2) Moderate (3) Severe: 1-9% AREA / 10-24% AREA / 25% or More AREA	JOINT/CRACK SPALLING Width Inches (1) 1/8" - 1" (2) 1-3" (3) 3" or More: 1-9% JOINTS / 10-24% JOINTS / 25% or More JOINTS	PUMPING BLOWING (1) Slight depression (2) Slight stain (3) Depression & stain: 1-9% OF PANELS / 10-24% OF PANELS / 25% or More PANELS	BLOWUPS: COUNT	FAULTING, SETTLEMENT (1) 1/8" - 1/4" (2) 1/4" - 1/2" (3) 1/2" or More: 1-9% OF PANELS / 10-24% OF PANELS / 25% or More PANELS	PATCHING No. Panels (1) 1-9% (2) 10-24% (3) 25% or More: 1-9% AREA/PANEL / 10-24% AREA/PANEL / 25% or More AREA/PANEL	** WEAR	REMARKS
5 7	8 9	10 15	16 19	20	21 .	26	27	28	29	30	34	38	41	44	47	50	53	56	59	60	63	66	
					.																		
					.																		
DUPLICATE					.																		
					.																		
COLUMNS					.																		
					.																		
1 THRU 20					21 .	26	27	28	29	30	34	38	41	44	47	50	53	56	59	60	63	66	
					.																		
					.																		
					.																		
					.																		
					.																		
					.																		
					21 .	26	27	28	29	30	34	38	41	44	47	50	53	56	59	60	63	66	
SR	RRT	RRQ	C.S.	D	ENDING ARM	TY	EX	LRB	BITUMINOUS PAVEMENTS						PORTLAND CEMENT CONCRETE								

REMARKS:

*CODE = (1) Unpaved Roadways, (2) Roadways under Construction (3) Bridge, (4) Road Impassable, (5) Other

** Rutting - Pavt. Wear
(1) 1/4" to 1/2"
(2) 1/2" to 3/4"
(3) Over 3/4"

Figure 3.5. Pavement Distress Recording Form (Washington State Department of Transportation)

$$\text{Rating (distress) score} = C - \left[\sum_{i=1}^{n} \sum_{j=1}^{m} a(T_i, S_j, E_{ij}) \right]$$

where C = initial rating (distress) number (usually 100), and

a ( ) = weighting factor or deduct points, which is a function of distress type T_i, severity of distress S_j, and extent of distress E_{ij}

WSDOT is in the process of upgrading its Pavement Management System. The condition rating is now termed Pavement Structural Condition (PSC) and is calculated separately for flexible and rigid pavements. The PSC has an upper limit of 100 (no distress) and a lower limit of 0 (extensive distress). The deduct points are now continuous functions as opposed to the previously used deduct tables for the prior Pavement Condition Rating (PCR). The specifics on how to calculate PSC for both flexible and rigid pavements can be found in WSDOT Research Report WA-RD 274.1 [3.30].

The PSC is calculated based on the amount and severity of the following distress types:

- Flexible pavements
  - Alligator cracking (fatigue cracking)
  - Longitudinal cracking
  - Transverse cracking
  - Patching
- Rigid pavements
  - Slab cracking
  - Joint and crack spalling
  - Pumping and blowing
  - Faulting and settlement
  - Patching
  - Raveling and scaling

The calculated PSC can be described by four broad pavement condtion categories:

PSC Range	Pavement Condition Category
100-75	Good
74-50	Fair
49-25	Poor
24-0	Very Poor

Typically, WSDOT attempts to program rehabilitation for pavement segments when they are projected to reach a PSC of 50. A PSC of 50 can occur due to various amounts and severities of distress. For example, a flexible pavement PSC of 50 is calculated when the wheeltrack length has 25 percent alligator cracking ("hairline" crack severity). For rigid pavement, a PSC of 50 represents 50 percent of PCC slabs exhibiting joint faulting with a severity of 1/8 to 1/4 in. A PSC of 50 can also be obtained if 25 percent of PCC slabs exhibit 2 to 3 cracks per panel.

A recent survey (1997) of the WSDOT route system for all functional classes revealed the following percentages for the four basic PSC ranges [3.30]:

PSC Range	Percentage of WSDOT Route System
100-75	66
74-50	24
49-25	9
24-0	1

Contrast the above to the PSCs calculated from the condition survey done about 20 years earlier (1973):

PSC Range	Percentage of WSDOT Route System
100-75	38
74-50	27
49-25	19
24-0	16

5. STRUCTURAL EVALUATION (SURFACE DEFLECTION)

5.1 PURPOSE OF DEFLECTION MEASUREMENTS

(a) A tolerable level of deflection is a function of traffic (type and volume) and the pavement structural section.

(b) Overlaying a pavement will reduce its deflection. The thickness needed to reduce the deflection to a tolerable level can be established.

(c) The deflections experienced by a pavement varies throughout the year due to the effects of temperature and moisture changes (including frost and thaw effects).

(d) For a given flexible pavement structure, the magnitude of surface deflections increases with an increase in the temperature of the bituminous surfacing material (due to decreasing stiffness of bituminous binder with increasing temperature).

5.2 CATEGORIES OF NONDESTRUCTIVE TESTING EQUIPMENT

(a) Static deflections

(b) Steady state deflections

(c) Impact load deflections (FWD)

(d) State (SHA) usage of deflection equipment in U.S.:

The usage of static, steady state, and impact NDT equipment by the 50 state highway agencies (SHAs) in the U.S. is shown in Table 3.4.

A 1988 SHA survey [3.22] revealed the following concerning use of NDT equipment.

Device	No. of SHAs
FWD20	
Dynaflect	18
Road Rater	7
Benkelman Beam	4

Eleven SHAs reported using more than one type of device. Most of the agencies reporting on how they used the NDT data can be categorized into the following categories:

Use of Data	No. of SHAs
• Void Detection	3
• Mechanistic - Empirical Design	2
• Pavement Management	2
• Load Transfer Estimates	1
• Backcalculation of Layer Moduli	1

WSDOT has used the FWD (Dynatest) since 1983.

Table 3.4. SHA Deflection Equipment [after Ref. 3.14]

Device	Number of SHAs Using Device for Various Time Periods		
	mid-1980s	1990	mid-1990s (estimated)
Benkelman Beam	18	3	0-3
Dynaflect	18	11	5-10
Road Rater	5	4	2-6
FWD	5	30	30-40

5.2.1 STATIC DEFLECTION EQUIPMENT

5.2.1.1 Benkelman Beam

(a) Most widely used device (developed at WASHO Road Test — 1952).

(b) Operates on a lever arm principle (refer to Figure 3.6).

(c) Must be used with loaded truck or aircraft. Truck weight normally 80 kN (18,000 lb) on a single axle with dual tires inflated to 480 to 550 kPa (70 to 80 psi).

(d) Measurements made by placing tip of beam between dual tires and measuring deflection as the vehicle (truck) is moved away. Measurements made with a dial gage.

(i) Asphalt Institute procedure [3.9] requires placement of tip of beam between dual tires even with the centerline of the rear axle prior to movement of the vehicle.

(ii) AASHTO T 256-77 (Pavement Deflection Measurements) requires that the tip of the beam be placed between the dual tires 1.4 m (4.5 ft) forward of the rear axle prior to movement of the vehicle.

(iii) ASTM D4695-87 (Standard Guide for General Pavement Deflection Measurements) recommends that the standard load for Benkelman Beam measurements be 80 kN (18,000 lb) on a single axle with dual 279 x 572 mm (11.00 x 22.5 in) 12-ply tires inflated to 480 kPa (70 psi). Pavement deflection to be measured with a dial gage or LVDT to within 0.025 mm (0.001 in.).

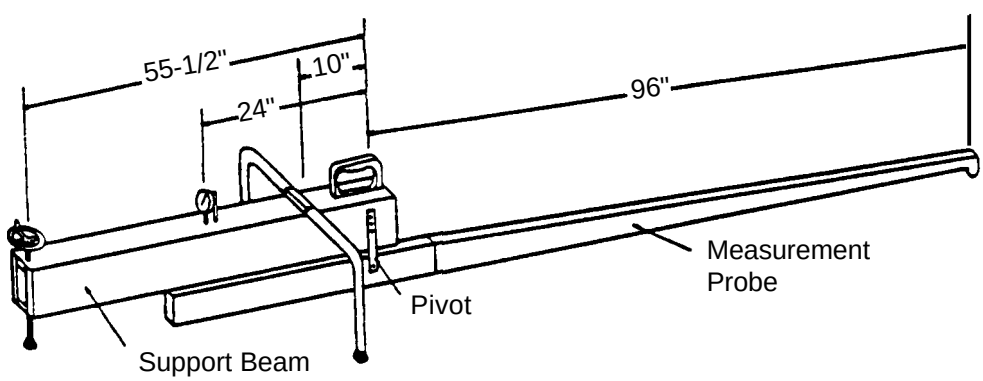


Figure 3.6. Sketch of Basic Components of Benkelman Beam [3.10]

(e) Manufactured by:

Soiltest Inc.
Materials Testing Division
2205 Lee Street
Evanston, Illinois 60202

(f) Disadvantages

(i) Slow, requires traffic control
(ii) Labor intensive
(iii) Does not provide deflection basin

(g) Advantages

(i) Widely used and hence numerous analysis procedures available to use with such data.

(ii) Instrument cost low (about $1,000).

5.2.1.2 Plate Bearing Test

(a) Standard test methods

(i) AASHTO T222-81 (Nonrepetitive Static Plate Load for Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements)

(ii) ASTM D1196-77 (same title as AASHTO T222)

(b) Uses

(i) To determine modulus of subgrade reaction (k value).
(ii) Typical uses of static (or near static) pavement surface deflections.
(iii) Asphalt Institute: overlay design and/or determination of remaining life.
(iv) California DOT: overlay design.

5.2.2 STEADY STATE DEFLECTIONS

5.2.2.1 General

(a) Several types of steady state deflection equipment are available. Primarily includes:

(i) Dynaflect (electro-mechanical)
(ii) Road Rater (electro-hydraulic)

(b) Equipment induces a steady state (nonchanging vibration to the pavement with a dynamic force generator).

(c) Pavement deflections measured with velocity transducers.

5.2.2.2 Dynaflect

(a) Standard test methods

(i) AASHTO T256-77
(ii) ASTM D4695-87

(b) Manufactured by:

Geolog Inc.
103 Industrial Boulevard
Granbury, Texas

(c) Mounted on a two-wheel trailer.

(d) Dynaflect is stationary when measurements are taken. Force generator (counter rotating weights) started and deflection sensors (velocity transducers) lowered to the pavement surface. Refer to Figure 3.7 (plot of typical force output) and Figure 3.8 (location of Dynaflect loading wheels and five velocity transducers). The peak-to-peak dynamic force is 4.4 kN (1,000 lb) at a fixed frequency of 8 Hz. This load is applied through two 102 mm (4 inch) wide, 406 mm (16 inch) diameter rubber-coated steel wheels which are placed 508 mm (20 inches) apart.

(e) Disadvantages

(i) Requires traffic control
(ii) Dynamic load significantly less than normal truck traffic.

(f) Advantages

(i) High reliability (low maintenance)
(ii) Can be used to obtain a deflection basin.

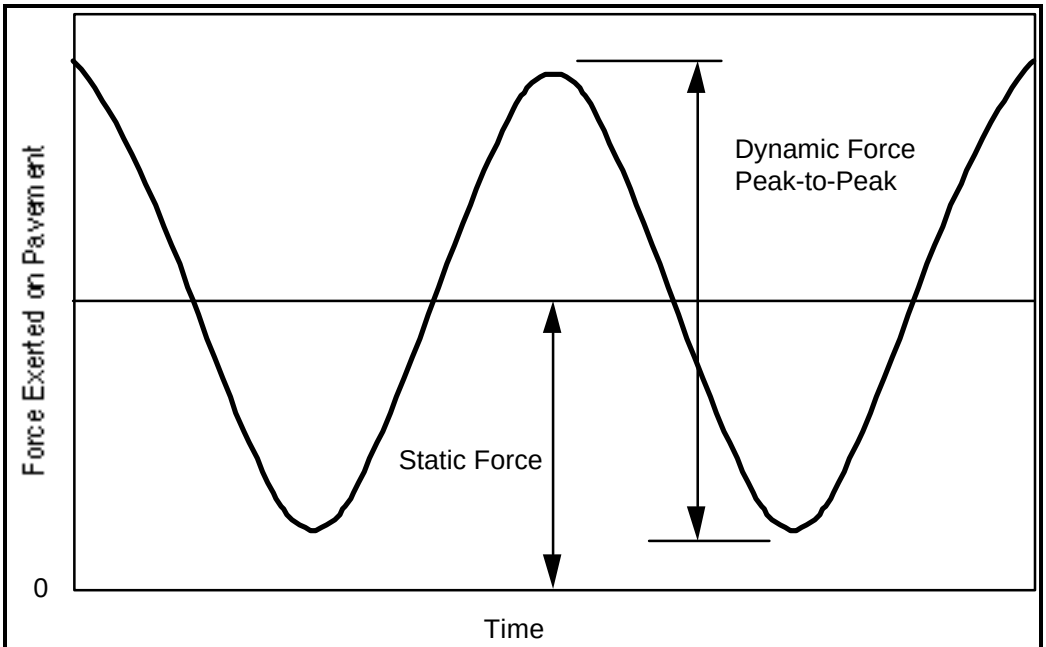


Figure 3.7. Typical Force Output of Steady State Dynamic Deflection Devices [3.10]

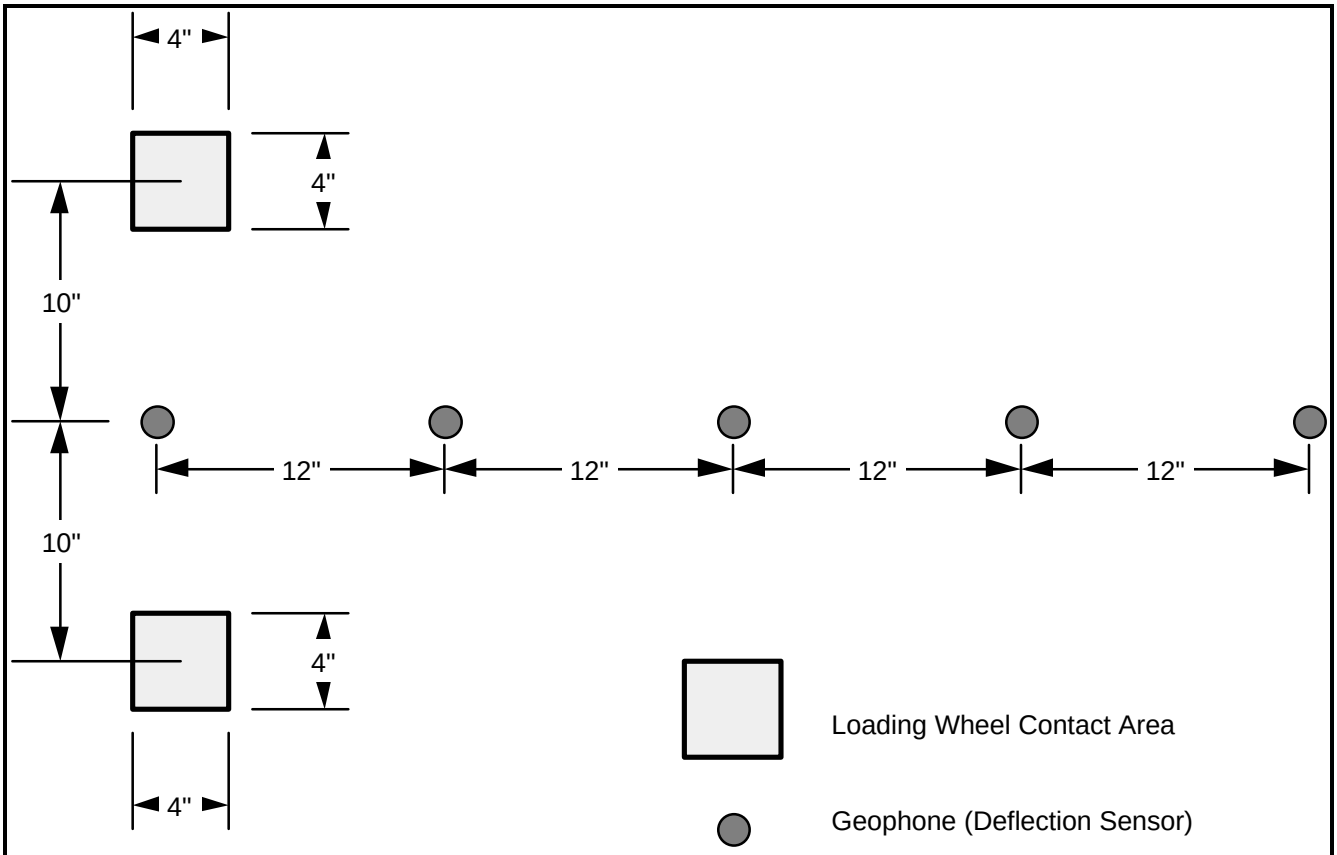


Figure 3.8. Standard Location of Dynaflect Loading Wheels and Geophones [3.10]

5.2.2.3 Road Rater

(a) Standard test methods

(i) AASHTO T256-77 (for Model 400 only)
(ii) ASTM D4695-87

(b) Manufactured by:

Foundation Mechanics, Inc.
421 East El Segundo Boulevard
El Segundo, California 90245

(c) Two production models available as of November 1991:

(i) Model 400 B-1
(ii) Model 2000 A-1

(d) Force generator consists of a steel mass, hydraulic actuated vibrator. Driving frequencies range between 5 and 60 Hz. Load ranges for various models:

(i) Model 400 B: 2.2 to 13.3 kN (500 to 3,000 lb)
(ii) Model 2000: 2.2 to 22.2 kN (500 to 5,000 lb)

The loading footprints for the two models are shown in Figure 3.9 (due to differences, one must be careful in comparing data between the models).

(e) Deflections are measured with four velocity transducers.

(f) Disadvantages

(i) Requires traffic control
(ii) Low load level relative to truck traffic (Models 400 B and 2000).
(iii) Large static preload for larger devices [3.10]

(g) Advantage

(i) Can measure deflection basin

5.2.2.4 Typical uses of steady state pavement surface deflections

(a) Correlation with static deflections (Benkelman Beam).
(b) Estimation of layer elastic modulus values.
(c) Overlay design and/or determination of remaining life.

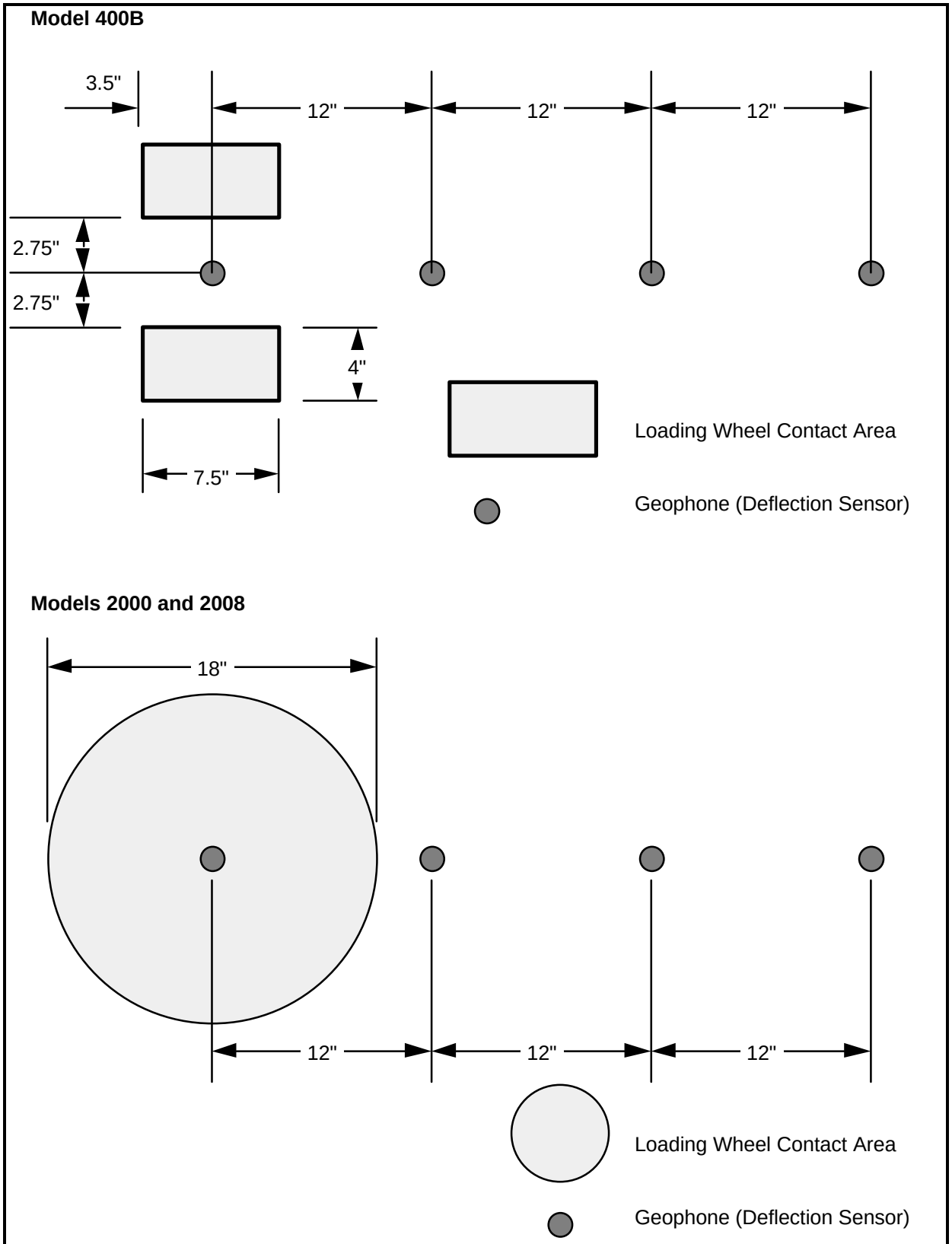


Figure 3.9. Standard Location of Loading Plate(s) and Geophones for the Road Rater Model 400 B and Models 2000 and 2008 [3.10]

5.2.3 IMPACT (IMPULSE) LOAD RESPONSE

5.2.3.1 General

(a) All impact load NDT devices deliver a transient impulse load to the pavement surface. The subsequent pavement response (deflection) is measured.

(b) Standard test methods

(i) ASTM 4694-87: Standard Test Method for Deflections with a Falling Weight Type Impulse Load Device

(ii) Related test method

ASTM D4695-87: Standard Guide for General Pavement Deflection Measurement

(iii) Significant features of ASTM D4694

- Falling Weight ("force-generating device")
  - Force pulse will approximate a haversine or half-sine wave
  - Peak force at least 50 kN (11,000 lb)
  - Force-pulse duration should be within range of 20 to 60 ms. Rise time in range of 10 to 30 ms.
- Loading plates

  Standard sizes are 300 mm (12 in.) and 450 mm (18 in.)
- Deflection transducers

  Can be seismometers, velocity transducers, or accelerometers. Used to measure the <u>maximum</u> vertical movement of the pavement.
- Signal conditioning and recorder system
  - Load measurements

    Accurate to at least ± 2 percent or ± 160 N (± 36 lb), whichever is greater.
  - Deflection measurements

    Accurate to at least ± 2 percent or ± 2 µm (± 0.08 mils), whichever is greater. Recall that 0.08 mils = 0.00008 inch and 2 µm = 0.002 mm.

- Precision and bias
  - Precision guide

    When a device is operated by a single operator in repetitive tests at the same location, the test results are questionable if the difference in the measured center deflection (D_0) between two consecutive tests at the same drop height (or force level) is greater than 5 percent. For example, if D_0 = 0.254 mm (10 mils) then the next load must result in a D_0 range less than 0.241 mm to 0.267 mm (9.5 to 10.5 mils).

(c) Measurements obtained very rapidly.

(d) Impact load easily varied.

(e) Pavement response usually measured with velocity transducers (Dynatest) or LVDT/accelerator meter combination (KUAB).

(f) The primary impact deflection equipment currently marketed in the U.S. include:

(i) Dynatest
Dynatest Consulting
Ojai, California
U.S.A.

(ii) KUAB
KUAB Konsult and
Utveckling AB
Box 10
79500 Rittvik, Sweden

or KUAB America
1401 Regency Drive East
Savoy, Illinois 61874
U.S.A.

(iii) Foundation Mechanics, Inc.
421 East El Segundo Boulevard
El Segundo, California 90245

5.2.3.2 Dynatest Falling Weight Deflectometer (FWD)

(a) Discussion will be for the Dynatest Model 8000.

(b) Most widely used FWD in U.S. (as of 1991)

(c) FWD is trailer mounted.

(d) By use of different drop weights and heights can vary the impulse load to the pavement structure from about 6.7 to 120 kN (1,500 to 27,000 lb). (The WSDOT FWD uses a weight system which results in an impulse load range of 24 to 71 kN (5,500 to 16,000 lb)). The weights are dropped onto a rubber buffer system resulting in a load pulse of 0.025 to 0.030 seconds (refer to Figure 3.10). The standard load plate has an 300 mm (11.8 in.) diameter.

(e) Typical location of the loading plate and seven velocity transducers is shown in Figure 3.11.

The WSDOT sensor spacings with the 300 mm (11.8 in.) load plate are:

inches	mm
0	0
8	203
12	305
24	610
36	914
48	1,219

The Strategic Highway Research Program (SHRP) sensor spacings with the 300 mm (11.8 in.) load plate are:

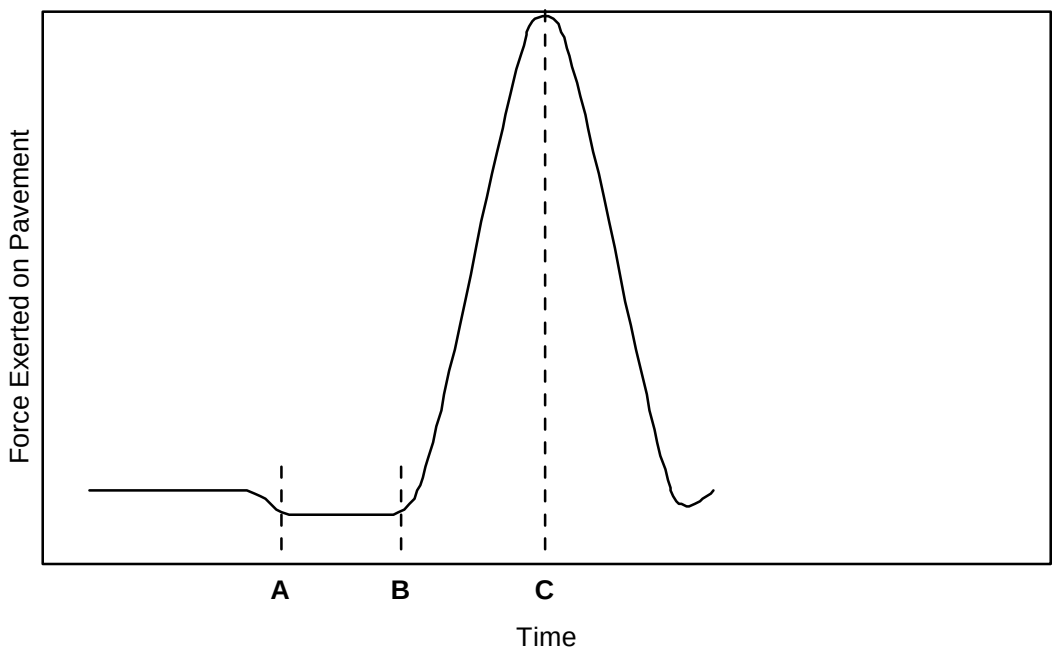
inches	mm
0	0
8	203
12	305
18	457
24	610
36	914
60	1,524

(f) SHRP FWDs [3.18, 3.19]: Dynatest Model 8000E

(i) Loading plate: 300 mm (11.8 in.)

(ii) Loads (flexible placements): Drops result in loads of approximately 27 kN (6,000 lb), 40 kN (9,000 lb), 53 kN (12,000 lb), and 71 kN (16,000 lb).

(iii) Peak deflections recorded for all four drops. A complete time — load and time — deflection "history" is recorded for the last drop at each of the four load levels.



A Time at which weights are released
B Time at which weight package makes first contact with load plate
C Peak load reached

Figure 3.10. Typical Force Output of Falling Weight Deflectometer [3.10]

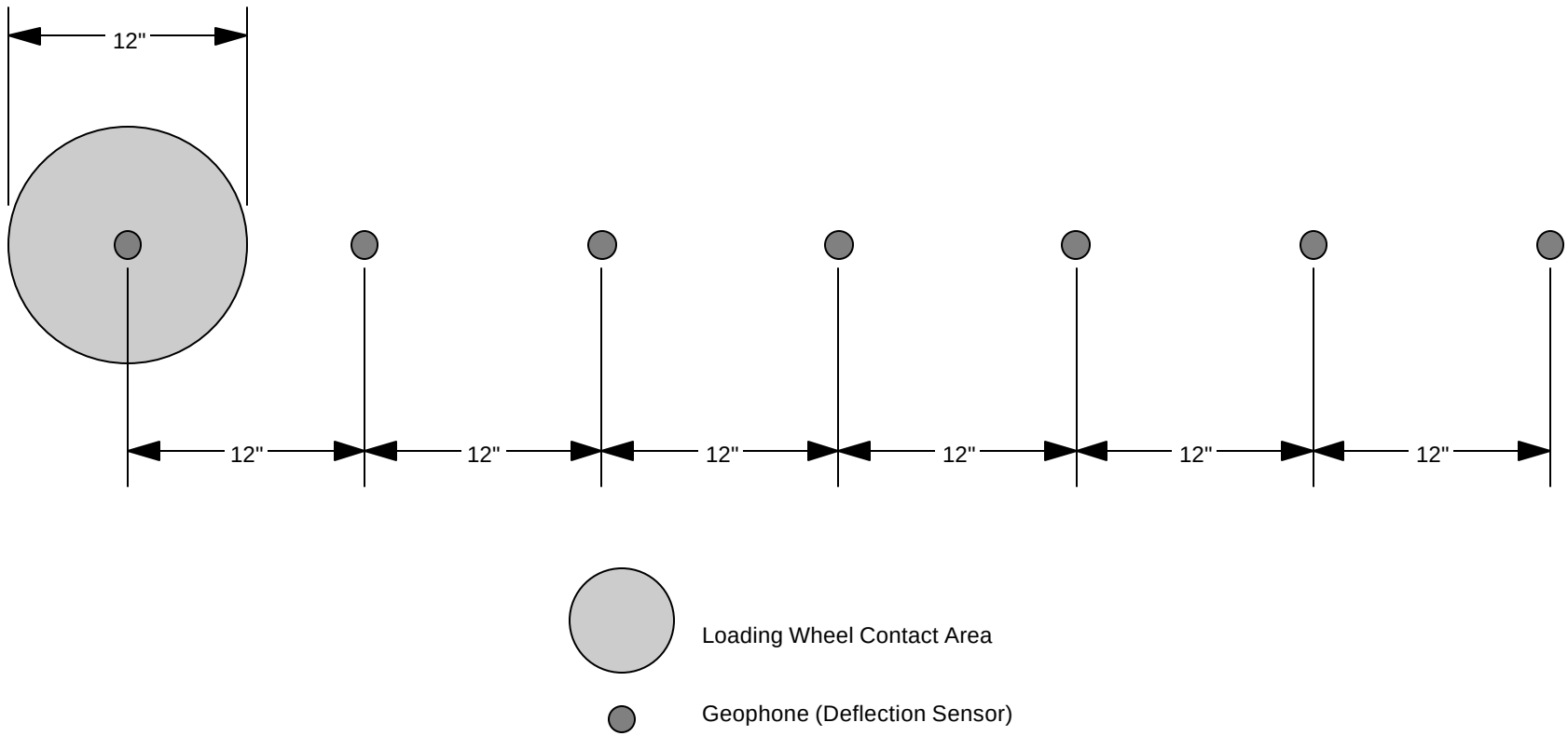


Figure 3.11. Typical Location of Loading Plate and Geophones for Falling Weight Deflectometers

(iv) SHRP Regional LTPP contractors use two computer programs to check FWD data [from Ref. 3.18]:

- FWDSCAN: checks FWD data files for completeness and readability.
- FWDCHECK: checks for section uniformity based on subgrade and pavement strength.

5.2.3.3 KUAB Falling Weight Deflectometer [3.15, 3.16, 3.17]

(a) Model 50

Load range: 7 to 65 kN (1,500 to 15,000 lb)

(b) Model 150

Load range: 14 to 150 kN (3,000 to 34,000 lbs)

(c) Total of five models are available (as of 1991). The heaviest load model has a range of 14 to 290 kN (3,000 to 66,000 lb).

(d) KUAB models are completely enclosed for protection during towing. The impulse force is the result of a unique two-mass system. The deflection sensors are called seismometers and use LVDTs along with a mass-spring reference system (standard KUAB FWD equipped with seven deflection sensors). Each sensor has micrometer making static field calibrations possible. The seismometers have three ranges: low (0-50 mils), medium (0-100 mils), and high (0-200 mils).

(e) Worldwide distribution of the KUAB FWD began in 1976 and subsequently over 60 units have been sold (as of 1991) [after Ref. 3.17].

(f) The basic weight of a KUAB FWD and associated trailer is about 1 800 kg (4,000 lb). A load of about 320 kg (700 lb) is applied to the pavement by the loading plate prior to testing [3.17].

(g) An original feature of the KUAB FWD is the segmented load plate (four quarter - circle segments). This provides a more uniform pressure distribution to the pavement surface [3.17].

(h) Accuracy and precision of KUAB 50 (based on KUAB product literature mostly from Ref. 3.17)

(i) Accuracy

- Deflection sensors: ± 2 µm (± 0.08 mils), ± 2 percent
- Load cell: ± 20 kg (± 44 lb), ± 2 percent

(ii) Precision

- Deflection sensors: ± 1 µm (± 0.04 mils), ± 1 percent
- Load cell: ± 10 kg (± 22 lb), ± 1 percent

(iii) Range

- Deflection sensors range 0 - 200 mils (0 - 5.08 mm)

5.2.3.4 Foundation Mechanics Falling Weight Deflectometer [3.12]

(a) Current model is the JILS-20-FWD.

(b) As of November 1991, three of these models have been in service for two years.

(c) Uses seven velocity transducers to measure the deflection basin (location of sensors is variable). Load is measured with a transducer.

(d) Load range: Approximately 7 to 107 kN (1,500 to 24,000 lbs)

(e) JILS-FWD is trailer mounted (tandem axle trailer) and includes a 16 horsepower gasoline engine to provide all necessary hydraulic and electrical power for operation. The gross weight of the unit is about 1 300 kg (2,800 lbs). The FWD unit can be enclosed with an available cover.

5.2.3.5 Comparisons between KUAB 150 and Dynatest 8000

(a) Refer to Table 3.5 for a comparison of model specifications.

(b) Disadvantages

(i) High initial cost
(ii) Traffic control required

(c) Advantages

(i) Gaining worldwide use
(ii) Best simulates actual wheel loads
(iii) Can measure deflection basin

Table 3.5. Equipment Specifications [3.16]

Characteristic	KUAB	Dynatest
Load range	7 - 150 kN	7 - 125 kN
Load rise time	Variable	Variable
Load duration	56 ms	25 - 30 ms
Load generator	Two-mass system	One-mass system
Load plate	Segmented or nonsegmented with rubberized pads (300 and 450 mm diameter)	Rigid with rubberized pad, tilts 6° (300 and 450 mm diameter)
Deflection sensors	Seismometer with static field calibration device	Geophones with or without dynamic calibration device
Deflection sensor position	0 - 1.8 m	0 - 2.25 m
Number of sensors	7 (all available positions)	Same
Deflection sensor range	5 mm (200 mils)	2 mm (80 mils) or 2.5 mm (100 mils)
Deflection resolution	1 µm (0.04 mils)	Same
Relative accuracy	2 µm ± 2%	Same
Test sequence	Unlimited user selected	8 drops
Test time sequence (4 loads)	35 s	25 s
Computer	HP 85B or IBM compatible. MS DOS	Same

5.2.3.6 Selected Comparisons for Seven NDT Devices [after Ref. 3.15]

(a) The U.S. Army Corps of Engineers, Waterways Experiment Station (WES) in Vicksburg, Mississippi, evaluated the following NDT devices:

- KUAB Model 50 FWD
- Dynatest 8000 FWD
- Dynatest HWD (Heavy Weight Deflectometer)
- Phonix FWD
- Dynaflect
- Road Rater 2008
- WES 16 kip vibrator

The basic equipment characteristics are shown in Table 3.6.

(b) Selected results from this evaluation included measurement of typical load pulses for the FWDs evaluated:

Measured loading times for the load cell (standard) experiment performed by WES:

Device	Loading Time (ms)
KUAB FWD	79.8
Dynatest FWD	30.4
Dynatest HWD	28.1
Phonix FWD	40.7

5.2.3.7 Typical Uses of Impact Pavement Surface Deflection

(a) Correlation with static deflections
(b) Estimation of layer elastic moduli
(c) Overlay design and/or determination of remaining life

5.3 CORRELATIONS BETWEEN DEFLECTION MEASURING EQUIPMENT

5.3.1 INTRODUCTION

In general, correlations between deflection devices should be used with caution. Too often, a correlation is developed for a specific set of conditions that may not be present for those using the correlation. It appears that the best approach is to obtain pavement parameters (such as layer moduli) from the specific NDT device being used. However, that said, a few of many such correlations that have been developed follow.

Table 3.6. NDT Device Characteristics (WES Evaluation) [3.15]

Device Name	Dynamic Force Range, lbf*	Load Transmitted by	Number and Type of Deflection Sensors	Deflection Sensor Spacing
Kuab FWD	3000 to 15 000	Sectionalized circular plate 11.8 in. diameter	7 seismometers	fixed at 0, 8, 12, 18, 24, 36, 48 in.
Dynatest HWD	10 000 to 55 000	Circular plate 11.8 or 17.7 in. dia	7 geophones	Variable, 12 to 96 in.
Dynaflect	1000 peak to peak	Two 16 in. dia by 2 in. width urethane-coated steel wheels	5 geophones	Variable, 0 to 48 in.
Dynatest FWD	1500 to 27 000	Circular plate 11.8 or 17.7 in. dia	7 geophones	Variable, 12 to 96 in.
Road Rater 2008	500 to 7000 peak to peak	Circular plate 18 in. dia	4 geophones	Variable, 24 to 48 in.
WES 16-Kip	500 to 30 000 peak to peak	Circular plate 18 in. dia	5 geophones	Variable, 12 to 60 in.
Phonix FWD	2300 to 23 000	Circular plate 11.8 in. dia	6 geophones	Variable, 8.3 to 58 in.

*1 lbf = 4.448 N
*1 in. = 2.54 cm

5.3.2 BENKELMAN BEAM TO FALLING WEIGHT DEFLECTOMETER (BASED ON UNPUBLISHED DATA COLLECTED BY WSDOT MATERIALS LABORATORY IN 1982-1983)

BB = 1.33269 + 0.93748 (FWD)

R^2 = 0.86

Std Error = 3.20 mils

Sample Size = 713 matched deflection points

where BB = Benkelman Beam deflection (in. x 10^{-3}),

FWD = FWD deflection (in. x 10^{-3}) corrected to a 9,000 lb load applied on a 11.8 inch diameter plate

5.3.3 BENKELMAN BEAM TO DYNAFLECT

(a) Arizona [after Reference 3.10]

BB = 22.5 (DMD)

where BB = Benkelman Beam deflection (in. x 10^{-3}),

DMD = Dynaflect Maximum Deflection (in. x 10^{-3}).

(b) Asphalt Institute [after Reference 3.10]

BB = 22.30 (D) - 2.73

where BB = Benkelman Beam deflection (in. x 10^{-3}),

D = Dynaflect center deflection (in. x 10^{-3}), same as DMD (Arizona).

(c) Louisiana [after Reference 3.10]

BB = 20.63(D)

R^2 = 0.72

where BB = Benkelman Beam deflection (in. x 10^{-3}),

D = Dynaflect deflection (in. x 10^{-3}), same as DMD (Arizona).

5.3.4 BENKELMAN BEAM TO ROAD RATER [FROM REFERENCE 3.11]

(a) Stabilized pavements: for Benkelman Beam load at 9,000 pounds on dual tires with 70-80 psi inflated tires and Road Rater at 8,000 pound peak-to-peak load at 15 Hz on a 12 inch diameter plate

$$BB = 2.57 + 1.27\ RR$$

$$R^2 = 0.66$$

where BB = Benkelman Beam deflection (in x 10^{-3}),

RR = Road Rater (Model 2008) deflection at 8,000 pounds and 15 Hz (in x 10^{-3}).

(b) Asphalt Institute [3.9]

Recommends that correlation between Benkelman Beam and Dynaflect be used to correlate Benkelman Beam to Road Rater Model 400 (with caution).

(c) Western Direct Federal Division, Federal Highway Administration, Vancouver, Washington

Correlation for Benkelman Beam to Road Rater Model 400

$$BB = 8.0 + 9.1026\ (D_0)$$

where BB = Benkelman Beam deflection (in. x 10^{-3}),

D_0 = Maximum deflection from Road Rater Model 400 (deflection location between load pads) at a load of 1,300 pounds at 25 Hz.

6. SKID RESISTANCE

6.1 DEFINITIONS

(a) Skid Resistance. Force developed when a tire that is prevented from rotating slides along the pavement surface [3.12].

(b) Hydroplaning. Separation of wheel from pavement surface by water [3.13]. Tends to occur when there is an abnormally thick water layer on the pavement or the vehicle speed is high.

(c) Pavement Texture. Generates resistance to sliding and facilitates expulsion of water from the tire - pavement interference.

(d) Pavement Microtexture. Controls contact between tire rubber and pavement surface. An individual aggregate particles surface texture. The fine texture of a bituminous concrete surface [3.13].

(e) Pavement Macrotexture. Controls the escape of water from under the tire hence loss of skid resistance with increased speed. Coarse texture of a bituminous surface [3.13].

6.2 PURPOSE OF FRICTION MEASUREMENTS

(a) To prevent or reduce skid related accidents.

(b) Most agencies have obligation to provide users of any transportation medium a roadway that is "reasonably" safe.

(c) Pavement management

- Identify areas of low skid resistance.
- Planning maintenance.
- Evaluate various types of materials and construction practice.

6.3 SEVERAL FRICTION RESISTANCE CONCEPTS

6.3.1 QUANTIFICATION OF SKID RESISTANCE

Skid resistance can be stated several ways. Some of those more commonly used in the U.S. include:

(a) Coefficient of friction

$\mu = F/L$

F = frictional resistance to motion in plane of interface

L = load perpendicular to interface

(b) Friction factor (better term for pavement friction)

$f = F/L$

Note: Not correct to say pavement has a certain friction factor (or coefficient of friction) because friction involves two bodies. Bodies (tires and pavement) extremely variable due to pavement wetness, vehicle speed, temperature, tire wear, tire type, etc.

(c) Friction number (FN), or Skid Number (SN), ASTM E 274 or AASHTO T242

FN or SN = 100(f) = 100(F/L)

F = obtained with a locked wheel standardized tire at constant speed (usually 40 mph) along artificially wetted pavement.

L = vertical load on locked wheel

Note: Friction resistance as measured in accordance with ASTM E 274 is an imitation of an emergency stop

6.3.2 DRY VS. WET PAVEMENTS

The friction resistance of most dry pavements (as determined by FN or SN) is high. Wet pavements are the problem. The number of accidents on wet pavements are twice as high as dry pavements (but other factors are involved other than skid resistance).

6.4 FRICTION REQUIREMENTS

6.4.1 FRICTION DEMAND VS. FRICTION SUPPLY (AFTER CORSELLO [3.25])

Skid resistance can be thought of in terms of the margin of safety between the friction supply available and the friction demand generated at any particular time and for a particular driving maneuver. Table 3.7 lists the major factors that affect friction supply and demand. Note that certain factors affect supply and demand and can therefore have a significant influence on skid resistance. Obvious examples are speed and weather related factors.

For a vehicle to follow a desired course the tires must be capable of developing definite friction factors. Friction demand factors vary with speed, tire type, weather conditions, roadway geometrics and, most importantly, with the maneuvers that drivers impose on their vehicles (changes in direction and rate of acceleration).

The large dependence of available friction on speed arises from the fact that on wet pavements the tire-pavement contact area is determined by the efficiency with which the tire can expel water from that area. Water viscosity and other effects cause this efficiency to degrade at higher speeds—consequently, the tire is unable to maintain a dry contact area [3.26].

Whether enough friction supply is produced in the tire-pavement interface under certain conditions, for example, high speed, depends on the properties of the tire, the pavement surface, and weather related factors.

Table 3.7. The Factors of Skid Resistance (from Byrd, et al. [3.26])

Friction Demand	Friction Supply
• Speed • Acceleration and braking • Vehicle characteristics • Weather related Wind Spray Sight • Roadway geometrics Curvature Superelevation Tracking	• Speed • Tire • Pavement surface Microtexture Macrotexture Drainage Cross slope • Weather related Rainfall Temperature Cyclic Effects Seasonal

The two principal factors responsible for rubber tire friction are adhesion and hysteresis; a high level of each is desirable for skid resistance. Adhesion is the product of interface shear strength and contact area. Hysteresis is caused by damping losses within the rubber when it is "flowing" over and around the mineral particle. Both are illustrated in Figure 3.12.

The two principal factors responsible for pavement surface friction are microtexture and macrotexture; both are needed to provide the pavement surface with an adequate level of friction. The microtexture, provided by the small surface asperities, affects the level of friction in the tire-pavement contact area. The macrotexture, provided by the larger surface asperities, provides escape channels for the surface water from the tire-pavement contact area. Microtexture varies from harsh to polished, and macrotexture varies from rough to polished, as shown in Figure 3.13. A high drainage potential is also desirable for the pavement surface, as it allows water to escape by gravity; this potential reduces the water film thickness at the interface and increases available friction [3.26].

Identifying the major factors of skid resistance highlights an important point—a significant number of wet weather accidents are related to factors other than a slick pavement. Because of the number of contributing factors, wet-pavement skid resistance is said to have a large degree of variability [3.26].

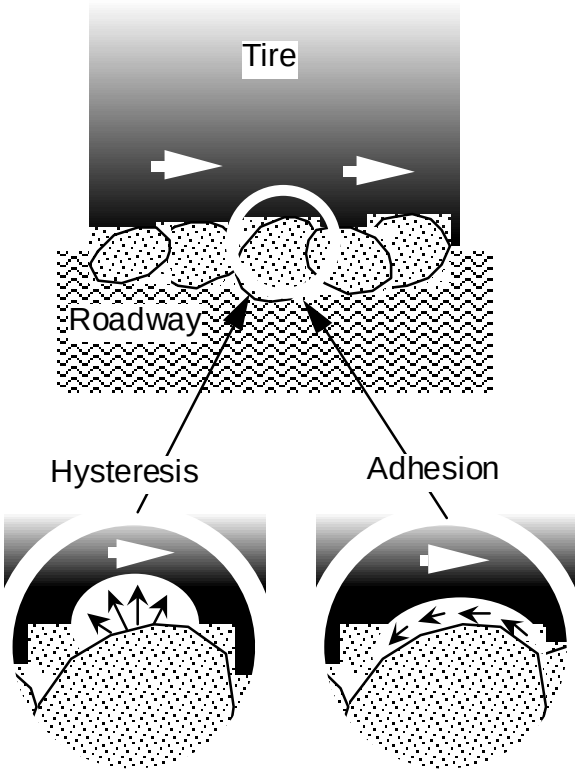


Figure 3.12 Rubber Tire Friction (Byrd et al. [3.26])

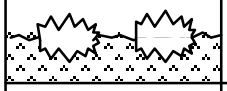
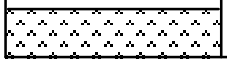
Surface	Macro (large)	Micro (fine)
	Rough	Harsh
	Rough	Polished
	Smooth	Harsh
	Smooth	Polished

Figure 3.13 Pavement Surface Friction, Scale of Texture (Byrd et al. [3.26])

Wet-pavement skid resistance is a major concern of highway departments. However, it is a relatively small factor in relation to all accidents:

> Reported studies have indicated that most accidents, perhaps more than 80 percent, may be attributed to human factors, and only 3 to 5 percent may be attributed to vehicular characteristics or failures, with the exception of tire condition. The remaining 10 to 15 percent of accidents may be related to the roadway and its environment [3.27].

To prevent skidding accidents, most states have initiated skid resistance programs that include means of measuring wet-pavement skid resistance.

6.4.2 MINIMUM FRICTION NUMBERS (SKID NUMBERS)

The initial research on a recommended minimum SN was reported in NCHRP Report 37 [3.28]. The tentative minimum was set at a SN = 37. This value was based on a number of assumptions including potential measurement errors and SN changes with pavement temperature changes. An updated guideline was developed by Corsello [3.25]. Adjustments were made to the NCHRP Report 37 methodology for current knowledge in the areas of pavement temperature and machine errors and their effect on pavement skid resistance. The resulting minimum skid number guideline was estimated by Corsello to be 26.

6.5 METHODS OF FRICTION NUMBER MEASUREMENT

6.5.1 INTRODUCTION

Skid testing in the U.S. may occur in a number of ways. Two of the most commonly used equipment types are the locked wheel tester (used by WSDOT) and the Mu-Meter (mostly used at airports). These equipment are briefly overviewed in Paragraph 6.5.2.

The dominant type of skid tester used by state DOTs uses a locked-wheel mechanism (refer to Table 3.8) to measure the brake-force coefficient under near steady-state conditions (ASTM E-274 or AASHTO T-242). The measurement represents a steady state friction force on a locked test wheel as the tire is dragged over a wetted pavement surface under constant speed. The locked test wheel's major plane is parallel to its direction of motion and perpendicular to the pavement.

The vast majority of skid measurement systems consist of a towing vehicle and a two-wheel trailer. Most commonly, the left wheel is locked during testing. The skid measurement system must have the following: (a) a transducer associated with each test wheel that senses the force developed between the sliding wheel and the pavement during testing, (b) electronic signal conditioning equipment to receive the transducer

Table 3.8. State Usage of Friction Measuring Equipment [after Reference 3.14]

Device	Number of SHAs		
	Mid-1980s	1990	Estimated for 1990s
Locked Wheel Tester	38	41	35-45
Mu-Meter	4	2	0-2
Spin-Up Tester	0	0	0-10?
Laser or Image Processing	0	0	0-20?

output signal and modify it as required, and (c) suitable analog and/or digital readout equipment to record either the magnitude of the developed force or the calculated value of the resulting Skid Number (SN).

The system must include a means to transport a supply of water — usually 200 to 500 gallons — and the necessary apparatus to deliver approximately 4 gallons of water per minute, per wetted inch of pavement at 40 mph within specified limits in front of the test wheel. The system must be able to measure the speed at which the test is conducted.

A standard tire is used in the test so that the skid resistance of different pavements can be compared. The standardized skid-test tire, a tubeless, bias-ply G78x15 tire with seven circumferential grooves, is defined by ASTM E-501 (AASHTO M-261). The standard rigidly prescribes the rubber composition. Thus, the tire type and design are eliminated as variables in the measurement of pavement skid resistance.

To take a measurement, the trailer is towed at a speed of 40 mph over the dry pavement, while water is applied in front of the test wheel. The test wheel is then locked up by a suitable brake. After the test wheel has been sliding on the pavement for a certain distance, the force that the friction in the tire contact patch produces and transmits as torque on the test wheel is measured and recorded for a specified length of time. The result of such a test is reported as the Skid Number (SN).

6.5.2 SPECIFIC SKID MEASUREMENT EQUIPMENT

6.5.2.1 Locked-Wheel Trailers (ASTM E-274, AASHTO T-242)

(a) Use standard tire.

(b) Test speed normally 40 mph.

(c) Two-wheel trailer - test wheel locked and force measured. Pavement surface artificially wetted.

(d) Most commonly used skid testing device in U.S. A locked-wheel trailer is used by WSDOT (manufactured by Cox and Sons).

6.5.2.2 YAW MODE TRAILER

(a) Commonly used device is "Mu-Meter".

(b) Two wheels turned in opposite directions to create transverse forces.

(c) Trailer travels in straight line without restraining mechanism.

(d) Both wheels cannot be in wheelpaths hence could measure higher friction than locked-wheel trailer.

6.5.2.3 AUTOMOBILE METHODS

(a) Potentially hazardous

(b) Vehicles often use diagonally braked wheels

(c) Primarily used on airfield pavements

(d) Pavement surface condition variable (wet, dry, etc.)

(e) Measurements:

(i) Distance vehicle travels after locking braking (using standard speed)
(ii) Time to decelerate from one speed to another
(iii) Deceleration measured directly

6.5.2.4 BRITISH PORTABLE TESTER

(a) Can be used in laboratory or field.

(b) Uses a pendulum with a spring-loaded rubber shoe.

(c) Pendulum drops and shoe slides over surface to be tested. Determines a measure of friction.

(d) Results reported as British Pendulum Number (BPN), ASTM E 303.

SECTION 3.0

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SECTION 4.0

FLEXIBLE AND RIGID PAVEMENT RESPONSES AND RELATED DESIGN PROCESSES

This SECTION is used to overview how flexible and rigid pavements respond to traffic loads and environmental changes (such as temperature). The concepts presented are of value in understanding the design procedures described in SECTIONS 5.0, 6.0, and 7.0.

1. INTRODUCTION

1.1 MOTIVATION FOR SWITCHING FROM EMPIRICAL DESIGN PROCESSES TO MECHANISTIC-EMPIRICAL PROCESS

Mechanics is the science of motion and the action of forces on bodies. When we refer to a mechanistic approach in engineering, we are talking about the application of elementary physics to determine the reaction of structures to loading. The primary concern in pavements is how the structure distributes vehicle loads to the underlying soil layers. Weak pavements will allow more of the load to be transmitted to the subgrade than strong pavements as shown in Figure 4.1. In order to quantify how the load is being distributed, certain fundamental properties of the materials must be known along with the thicknesses of the pavement layers and the load characteristics. These will be discussed later.

An empirical approach is one which is based on the results of experiments or experience. Generally, it requires a number of observations to be made in order to ascertain the relationships between the variables and outcomes of trials. It is not necessary to firmly establish the scientific basis for the relationships as long as the limitations are recognized. In some cases, it is much more expedient to rely on experience than to try to quantify the exact cause and effect of certain phenomena.

Most of the pavement design procedures used in the past have been empirical in that their failure criteria were based on a set of given set of conditions, i.e., traffic, materials, layer configurations, and environment.

The equation for the thickness of cover for asphalt pavements developed by Hveem and Carmany [4.1] for California highways is an example of empirical pavement design:

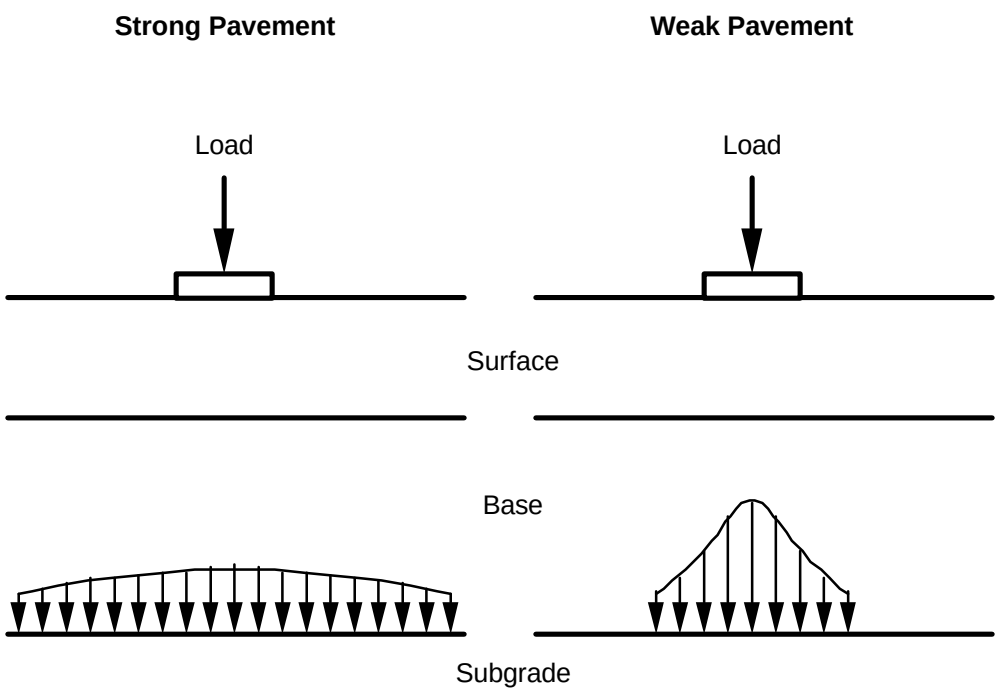


Figure 4.1. Load Distribution Characteristics of Strong versus Weak Pavement

$$T = K'(TI)(90 - R)/(c)^{0.2}$$

where T = thickness of cover,

K' = 0.095 (coefficient depending on design wheel load and tire pressure with a factor of safety),

TI = traffic index,

R = resistance value, and

c = cohesiometer value.

Although the above equation encompasses parameters for the bound materials (c-value) and the underlying unbound materials (R-value) as well as the traffic volume (TI), it is based on a 5000 lb (22 kN) wheel load with a tire pressure of 70 psi (480 kPa). Yoder [4.2] noted that it is unlikely that this design procedure could be successfully adapted to a region with severe frost problems or different rainfall characteristics.

Another illustration of an empirical design procedure is the AASHTO process. The fundamental information for developing the design procedure came from the AASHO Road Test which was constructed and tested during the late 1950s and early 1960s (Figure 4.2 shows a layout of the test section loops at the Road Test). The most recent version of this flexible pavement design process is illustrated in Figure 4.3. This figure shows the basic design nomograph which was developed from the following empirically derived performance equation:

$$\log_{10} W_{18} = (Z_R)(S_0) + (9.36)(\log(SN + 1)) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + (2.32)(\log_{10} M_R) - 8.07$$

where W_{18} = 18,000 lb (80 kN) equivalent single axle loads predicted to p_t,

Z_R = Z-statistic associated with the selected level of design reliability,

S_0 = overall standard deviation of normal distribution of errors associated with traffic prediction and pavement performance,

SN = Structural Number (essentially a "Thickness Index"),

ΔPSI = overall serviceability loss = $p_0 - p_t$,

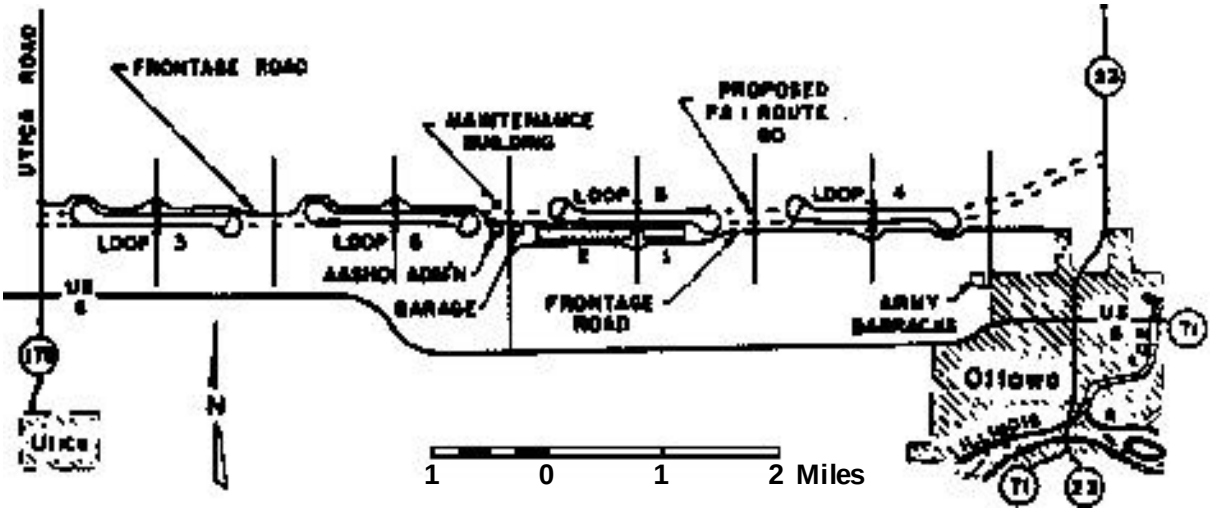
p_0 = initial serviceability index following construction,

p_t = terminal serviceability index, and

M_R = resilient modulus of the roadbed soil(s).

The various constants in the above equation were obtained from regression analysis of the AASHO Road Test data — hence this too is an empirical design procedure as opposed to a mechanistic-empirical approach.

A mechanistic-empirical approach to pavement design incorporates elements of both approaches. The mechanistic component is the determination of pavement reactions such as stresses, strains, and deflections within the pavement layers through the use of mathematical models. The empirical portion relates these reactions to the performance of the pavement structure. For instance, it is possible to calculate the amount of deflection at the surface of the pavement using some of the tools discussed later. If these deflections are related to the life of the pavement, then an empirical relationship has been established between the mechanistic response of the pavement and its expected performance.



Layout of AASHO Road Test

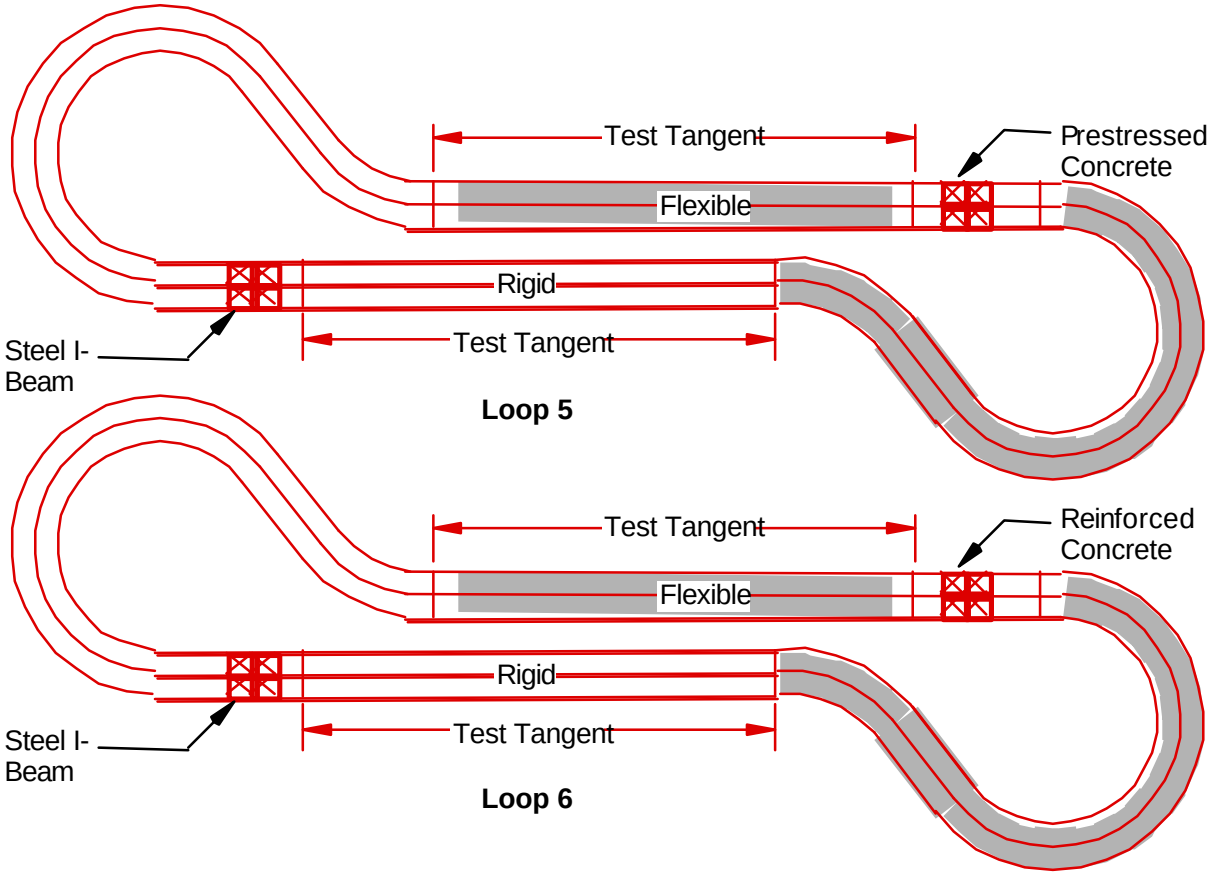


Figure 4.2. Layout of the AASHO Road Test [2.11]

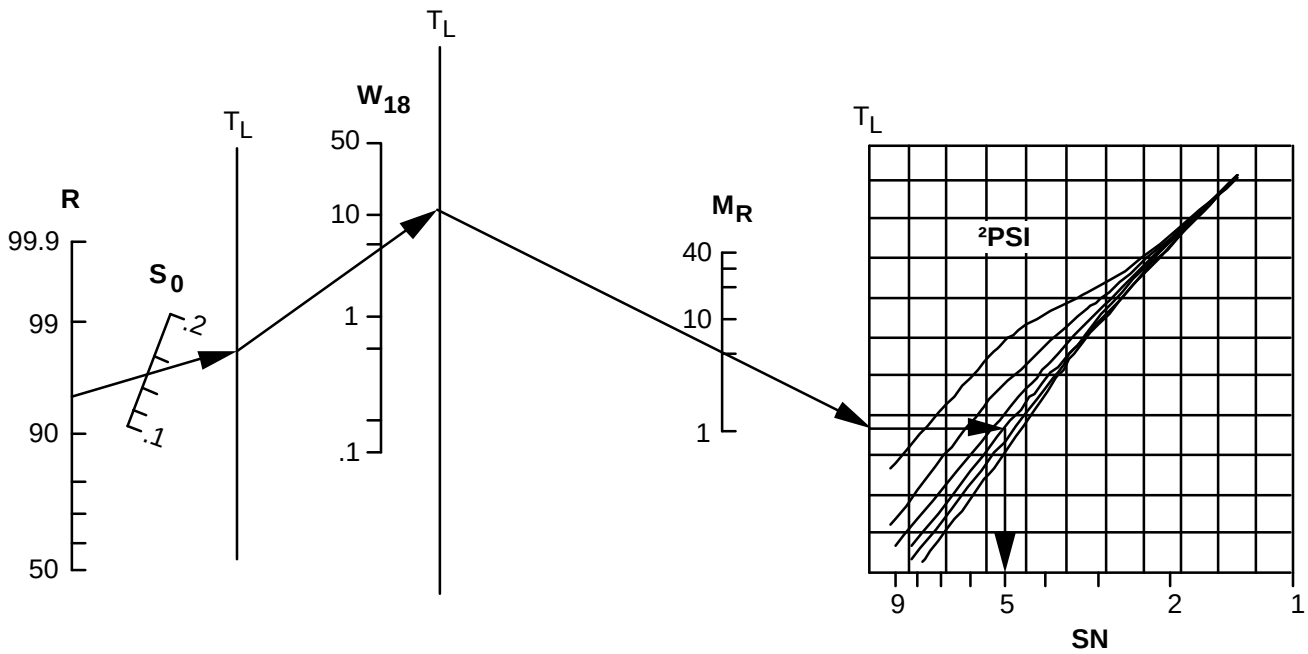


Figure 4.3. Illustration of AASHTO Design Nomograph for Flexible Pavements

The basic advantages of a mechanistic-empirical pavement design procedure are:

(a) The accommodation of changing load types.

(b) A better utilization of available materials.

(c) The ability to accommodate new materials.

(d) The improvement of reliability of performance predictions.

(e) A better definition of the role of construction.

(f) Material properties which relate better to actual pavement behavior and performance.

(g) An improved definition of existing pavement layer properties.

(h) The accommodation of environmental and aging effects on materials.

The primary means of mathematically modeling a pavement is through the use of layered elastic analysis. Although more complicated techniques are available (e.g., dynamic, viscoelastic models), we will restrict the discussion to basic linear elastic models subjected to static loading. Layered elastic analysis computer programs can easily be run on personal computers and do not require data which may not be realistically obtained.

1.2 ELASTICITY

The modulus of elasticity and Poisson's ratio of each layer define the material properties required for computing the stresses, strains, and deflections in a pavement structure. Typical values for the moduli and Poisson's ratios of pavement materials were given in SECTION 2.0 of this volume.

In addition to the material properties of the layers, the thickness of each pavement layer must be described. For computation purposes, the layers are assumed to extend infinitely in the horizontal direction, and the bottom layer (usually the subgrade) is assumed to extend infinitely downward. Given the general geometry of pavements, these assumptions are considered to be representative of actual conditions.

The loading conditions must be specified in terms of the magnitude of the load, the geometry of the load, and the number of loads to be applied to the structure. The magnitude of the load is simply the total force (P) applied to the pavement surface. In pavement analysis, the load geometry is usually specified as being a circle of a given radius (r or a), or the radius computed knowing the contact pressure of the load (p) and the magnitude of the load (P). Although most actual loads more closely represent an ellipse, the effect of the differences in geometry become negligible at a very shallow depth in the pavement. Multiple loads on a pavement surface can be accommodated by summing the effects of individual loads. This can be done because we are assuming that the materials are not being stressed beyond their elastic ranges.

To summarize, the following information must be available to compute the response of a pavement to loading:

(a) Material properties of each layer

  (i) Modulus of elasticity
  (ii) Poisson's ratio

(b) Thickness of each pavement layer

(c) Loading conditions

(i) Magnitude of load
(ii) Geometry of load
(iii) Number of loads

Figure 4.4 shows how these inputs relate to a layered elastic model of a pavement system.

The outcome of a layered elastic analysis is the computation of stresses, strains, and deflections in the pavement. The stress is the intensity of internally distributed forces experienced within the pavement structure at various points, and has units of force per unit area (psi or N/m^2(Pa)). Strain is the unit displacement due to stress, usually expressed as a ratio of the change in dimension to the original dimension (in/in or mm/mm). Since the strains in pavements are very small, they are normally expressed in terms of microstrain (10^{-6}). Deflection is the linear change in a dimension and is expressed in units of length (inches or mils or mm or µm).

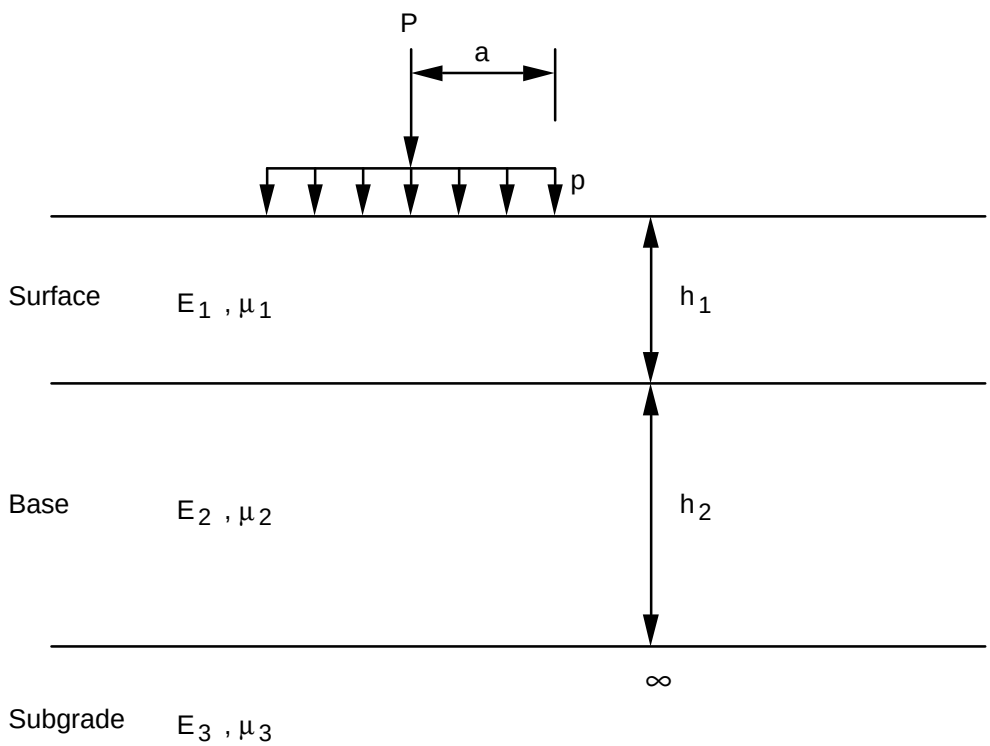


Figure 4.4. Layered Elastic Pavement Model

The use of a layered elastic analysis computer program will allow one to calculate the theoretical stresses, strains, and deflections anywhere in a pavement structure. However, there are only a few locations in which we are generally interested for the calculation of critical responses. These are:

Location	Response
Pavement Surface	Deflection
Bottom of Asphalt Concrete	Horizontal Tensile Strain
Top of Intermediate Layer (Base or Subbase)	Vertical Compressive Strain
Top of Subgrade	Vertical Compressive Strain

The locations of these responses relative to a pavement structure and load are illustrated in Figure 4.5. The horizontal tensile strain at the bottom of the asphalt concrete layer is used in fatigue failure criteria. Rutting failure in the subgrade can be predicted using the vertical compressive strain at the top of the subgrade. Deflections under load at the pavement surface are used in imposing load restrictions during spring thaw and overlay design (for example).

In the next three sections, we will discuss the evolution of layered elastic analysis and try to get a qualitative understanding of the process.

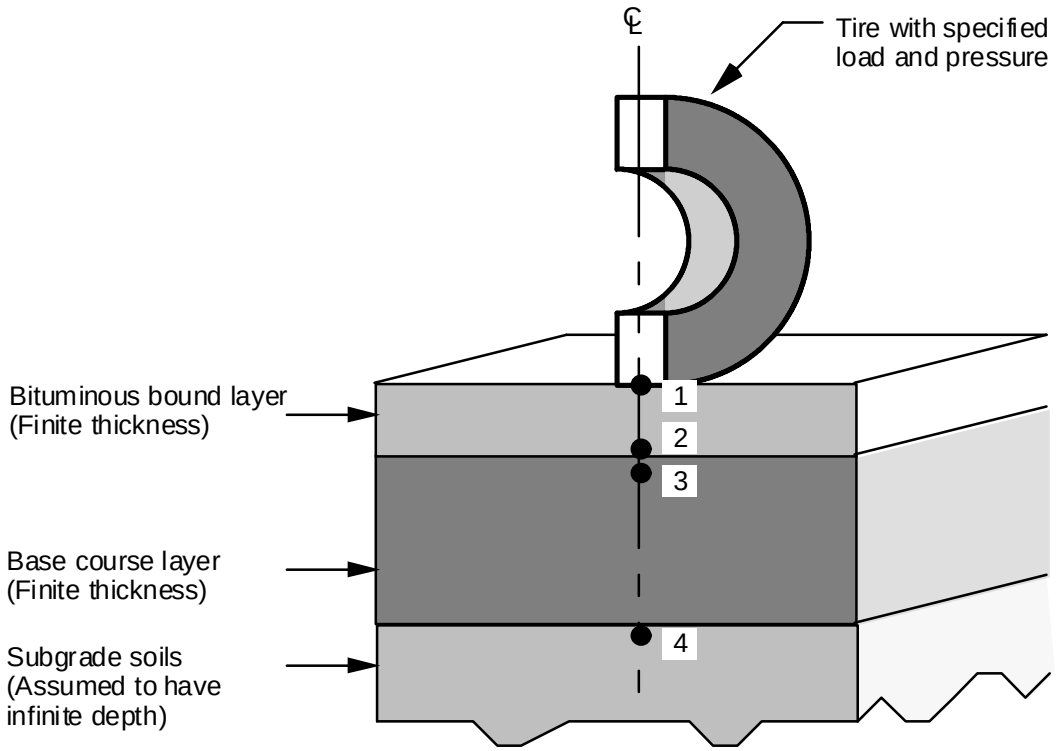
1.3 ONE-LAYER SYSTEM (BOUSSINESQ)

The origin of layered elastic theory is credited to V.J. Boussinesq [4.3] who published his classical work in 1885. He developed solutions for computing stresses and deflections in a halfspace (soil) composed of homogeneous, isotropic, and linearly elastic material. Indeed, Boussinesq influence charts are still widely used in soil mechanics and foundation design.

In this approach, the stresses and deflections are calculated for a point load applied to the surface of a deep soil mass. Distance variables are expressed in terms of cylindrical coordinates, in which the distance from a point on the surface may be expressed as:

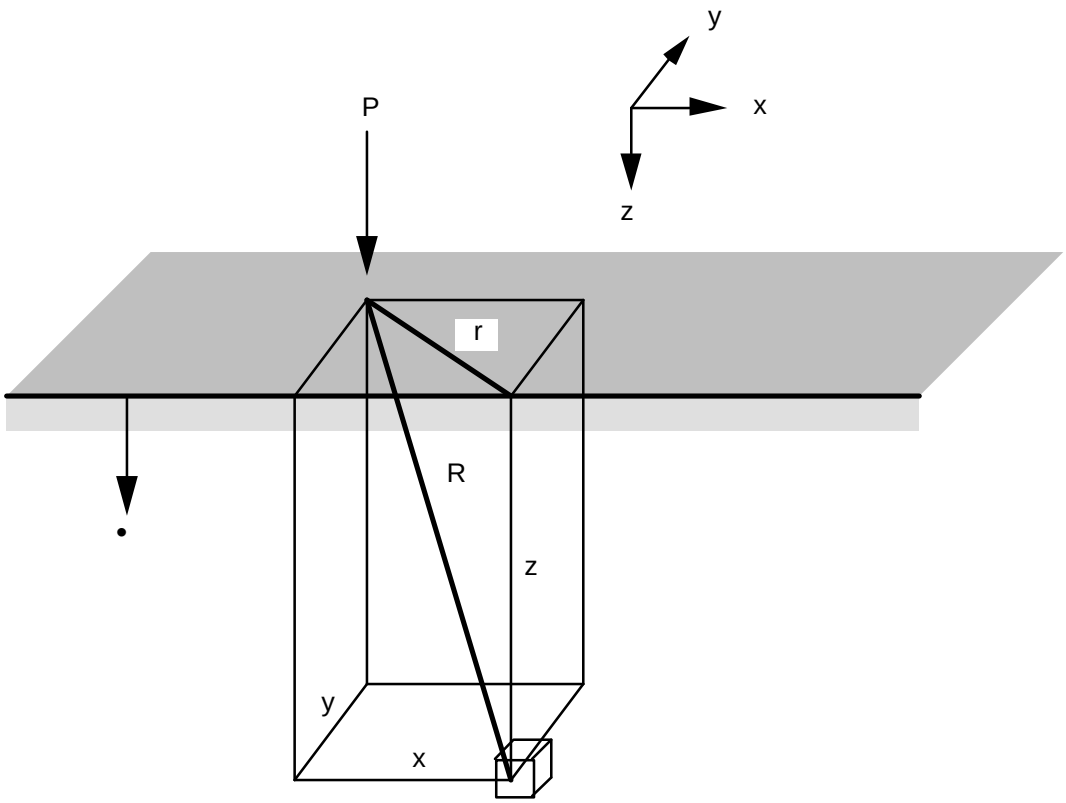
$$R^2 = r^2 + z^2 = x^2 + y^2 + z^2$$

as shown in Figure 4.6.

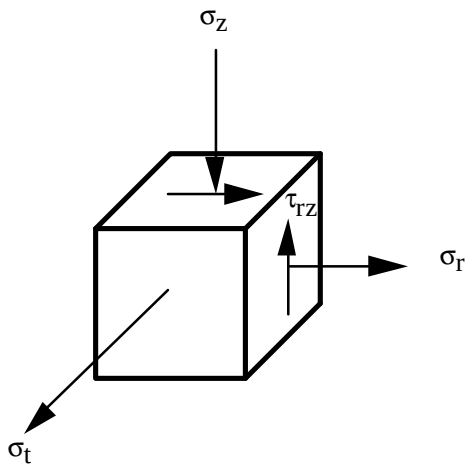


1. Pavement surface deflection
2. Horizontal tensile strain at bottom of bituminous layer
3. Vertical compressive strain at top of base
4. Vertical compressive strain at top of subgrade

Figure 4.5. Pavement Response Locations Used in Evaluating Load Effects



(a) Cylindrical Coordinates



(b) Stresses Acting on Soil

Figure 4.6. One-Layer System

The vertical stress (σ_z), radial stress (σ_r), tangential stress (σ_t), and shear (τ_{rz}) can be calculated using the formulas given below (after Taylor [4.15]):

Vertical stress: $$\sigma_z = \frac{P}{2\pi}\,\frac{3z^3}{(r^2+z^2)^{5/2}}$$

Radial stress: $$\sigma_r = \frac{P}{2\pi}\left[\frac{3r^2z}{(r^2+z^2)^{5/2}} - \frac{1-2\mu}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

Tangential stress: $$\sigma_t = -\frac{P}{2\pi}(1-2\mu)\left[\frac{z}{(r^2+z^2)^{3/2}} - \frac{1}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

Shear stress: $$\tau_{rz} = \frac{P}{2\pi}\,\frac{3rz^2}{(r^{2+}z^2)^{5/2}}$$

<u>Example 1:</u>

Let P = 1,000 lb (4.4kN), depth (z) = 10 in. (254 mm), radial distance from point load (r) = 0 in. (0 mm), and μ = 0.4. Calculate σ_z and σ_r.

$$\sigma_z = \frac{(1000)}{2\pi}\,\frac{3(10)^3}{(0^{2+}10^2)^{5/2}} = 4.8 \text{ psi } (33.1 \text{ kPa})$$

$$\sigma_r = \frac{1000}{2\pi}\left[\frac{3(0)^2(10)}{(0^2+10^2)^{5/2}} - \frac{1-2(0.4)}{0^2+10^2+10\sqrt{0^2+10^2}}\right]$$

$$= 0.2 \text{ psi } (1.4 \text{ kPa})$$

If the load is doubled, then the vertical stress is doubled.

1.4 TWO-LAYER SYSTEM (BURMISTER)

Burmister [4.4] extended the one-layer solutions to two and three layers. We will restrict the discussion in this section to two layers, analogous to a full-depth asphalt layer over subgrade. In his work, Burmister assumed that the layers have full frictional contact (no slip) at the interface, and that there are no shear or normal forces on the

surface outside of the loaded area. He also assumed that Poisson's ratio for each of the two layers is 0.5 in order to simplify the mathematics. The exact equations are rather long and complicated even with this simplification, but the important parameters in the solution are listed below:

- p — load distributed over a circular plate
- a — radius of the flexible plate
- h — thickness of the surface layer
- E_1 — modulus of elasticity for the surface layer
- E_2 — modulus of elasticity for the subgrade

In the equations, the geometry of the load and surface layer are specified by the ratio a/h, and the moduli of the layers by E_2/E_1. Graphs based on these ratios were developed to aid in determining the surface deflections and stresses in pavements. Figure 4.7 is a graph which can be used to find the displacement coefficient $I_{\Delta z}$, for calculating the deflection on the surface at the center of the loaded area using the equation below:

$$\Delta_z = 1.5 \frac{(p)(a)}{E_2} I_{\Delta z}$$

Figure 4.8 can be used to find the ratio of the vertical stress in the pavement structure to the applied stress. Please note that Figure 4.8 is only valid when the <u>radius of the load is equal to the surface thickness</u>. Other graphs would be needed for different values of a/h.

<u>Example 1:</u>

A 12 inch (300 mm) diameter plate is loaded to 9,000 lbs (40 kN) on an asphalt concrete pavement over a subgrade. The asphalt concrete is 6 inches (15 cm) thick and has a modulus of elasticity of 500,000 psi (3450 MPa); the subgrade modulus is 10,000 psi (69 MPa). What is the deflection at the center of the loaded area? What is the vertical stress at the top of the subgrade?

$$\frac{E_2}{E_1} = \frac{10{,}000 \text{ psi}}{500{,}000 \text{ psi}} = \frac{1}{50} \qquad \text{Area of Plate} = a^2 = 113 \text{ in}^2$$

$$\frac{a}{h} = \frac{6}{6} = 1 \qquad p = P/A = \frac{9{,}000 \text{ lb}}{113 \text{ in}^2} = 80 \text{ psi}$$

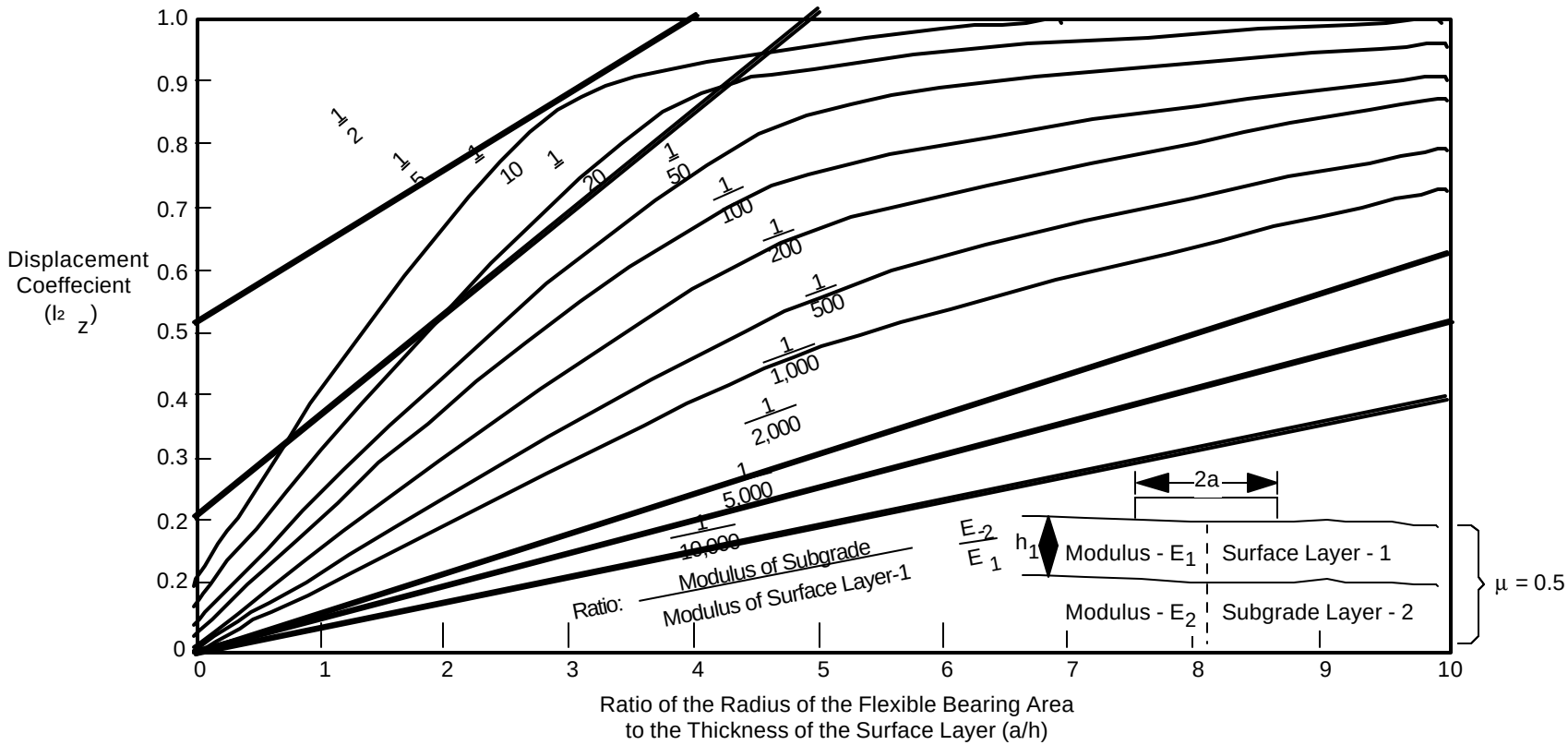


Figure 4.7. Sketch of Graph for Determining Displacement Coeffecient for Two-Layer System

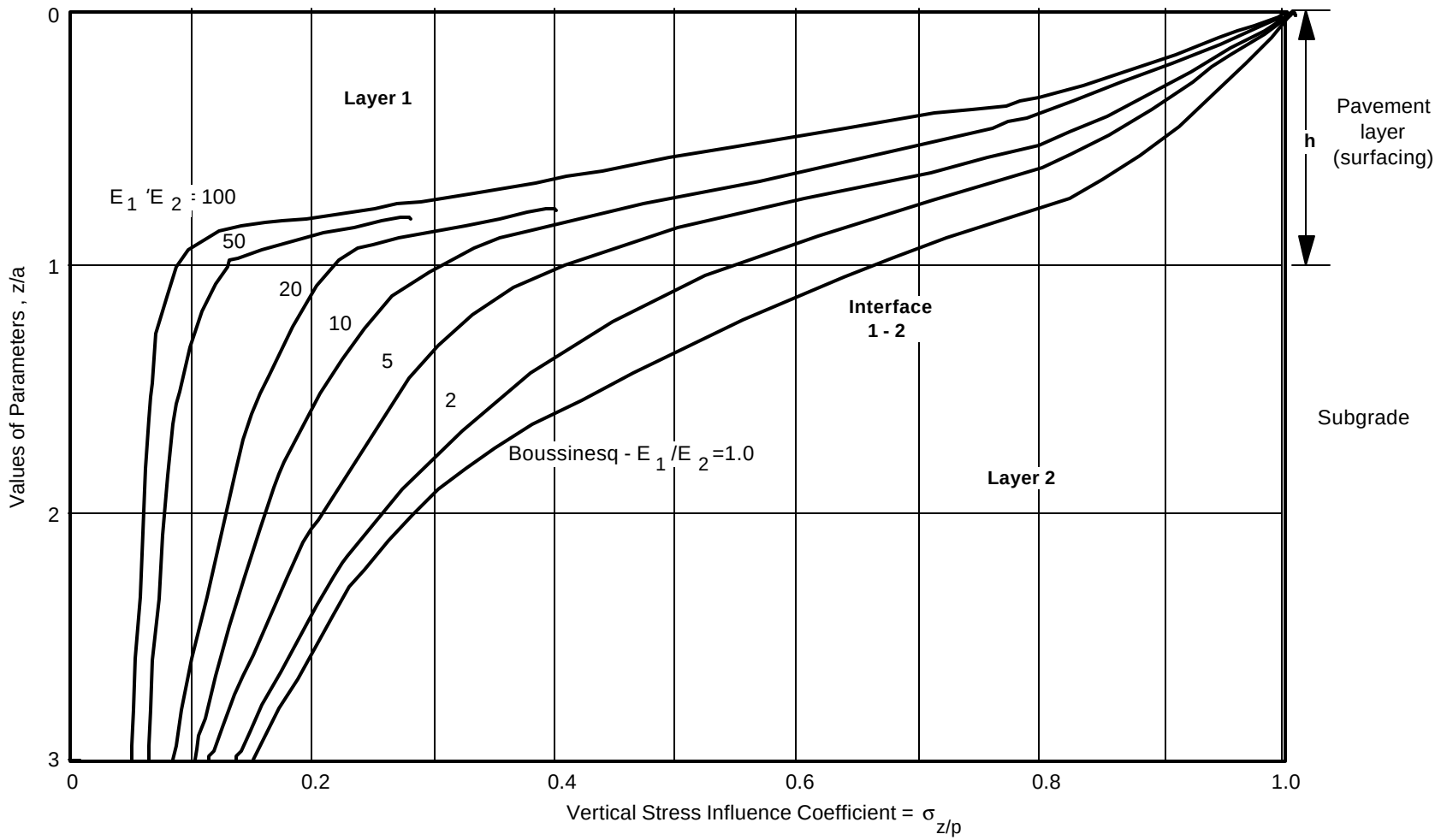


Figure 4.8. Sketch of Vertical Stress as a Function of Depth in a Two-Layered System, for a = h

Deflection at center of load:

From Figure 4.7: $I_{\Delta z} = 0.29$

$$\Delta_z = 1.5 \frac{(80)(6)}{10{,}000} (0.29) = 0.02 \text{ in. } (0.5 \text{ mm})$$

Vertical stress at top of subgrade:

From Figure 4.8: $\sigma_z/p = 0.12$

$$\sigma_z = 0.12(80) = 9.6 \text{ psi } (66 \text{ kPa})$$

1.5 MULTI-LAYER SYSTEM

The majority of pavement structures are more complicated than the two-layer system discussed above. Usually a base layer with or without an underlying subbase layer are placed between the asphalt concrete and subgrade. These layers may be either unbound or stabilized granular materials which have distinctly different properties from the surface and subgrade layers. One can imagine that the solutions for systems with three or more layers become increasingly more complex.

For a three-layer pavement, several charts and tables have been developed by Peattie, Jones and Fox [4.5] to determine the stresses, strains, and deflections. Peattie [4.6] developed graphical solutions for vertical stress in three-layer systems. Jones [4.7] presented solutions for horizontal stresses in a tabular form. Both of these solutions were based upon a Poisson's ratio of 0.5 for all layers.

The logical extension of these solutions was the development of computer programs in order to expedite analysis and allow greater flexibility in accommodating material properties and multiple loads. Even the most elementary of these allow for materials with Poisson's ratios other than 0.5. Some are capable of ascertaining the effects of multiple wheel configurations and/or nonlinear material behavior. During the 1980s, as these mainframe computer programs were converted to run on microcomputers, a greater potential was developed to use elastic analysis tools in pavement design and rehabilitation decision making.

Typical input required for using these computer programs include:

(a) Material properties of each layer

  (i) Modulus of elasticity
  (ii) Poisson's ratio

(b) Thickness of each pavement layer

(c) Loading conditions (2 of 3 listed below)

(i) Magnitude of load
(ii) Radius of load
(iii) Contact pressure

(d) Number of loads

(e) Location of load(s) on the surface (x,y coordinates)

(f) Location of analysis points for output (x,y,z coordinates)

For the purposes of illustration, three typical pavement cross-sections have been selected as shown in Figure 4.9. The initial conditions we will examine are:

(a) Material properties of each layer

Layer	Elastic Modulus		Poisson's Ratio
	psi	MPa	
Asphalt Concrete	500,000	3450	0.35
Crushed Stone Base	25,000	172	0.40
Fine-grained Subgrade	7,500	52	0.45

(b) Layer Thicknesses

Section	AC Thickness		Base Thickness	
	in	mm	in	mm
A	2	50	6	150
B	5	125	8	200
C	9	230	6	150

(c) Loading Conditions

Magnitude of Tire Load = 9,000 lbs (40 kN)
Tire Pressure = 80 psi (552 kPa)

(d) Number of Loads = 1

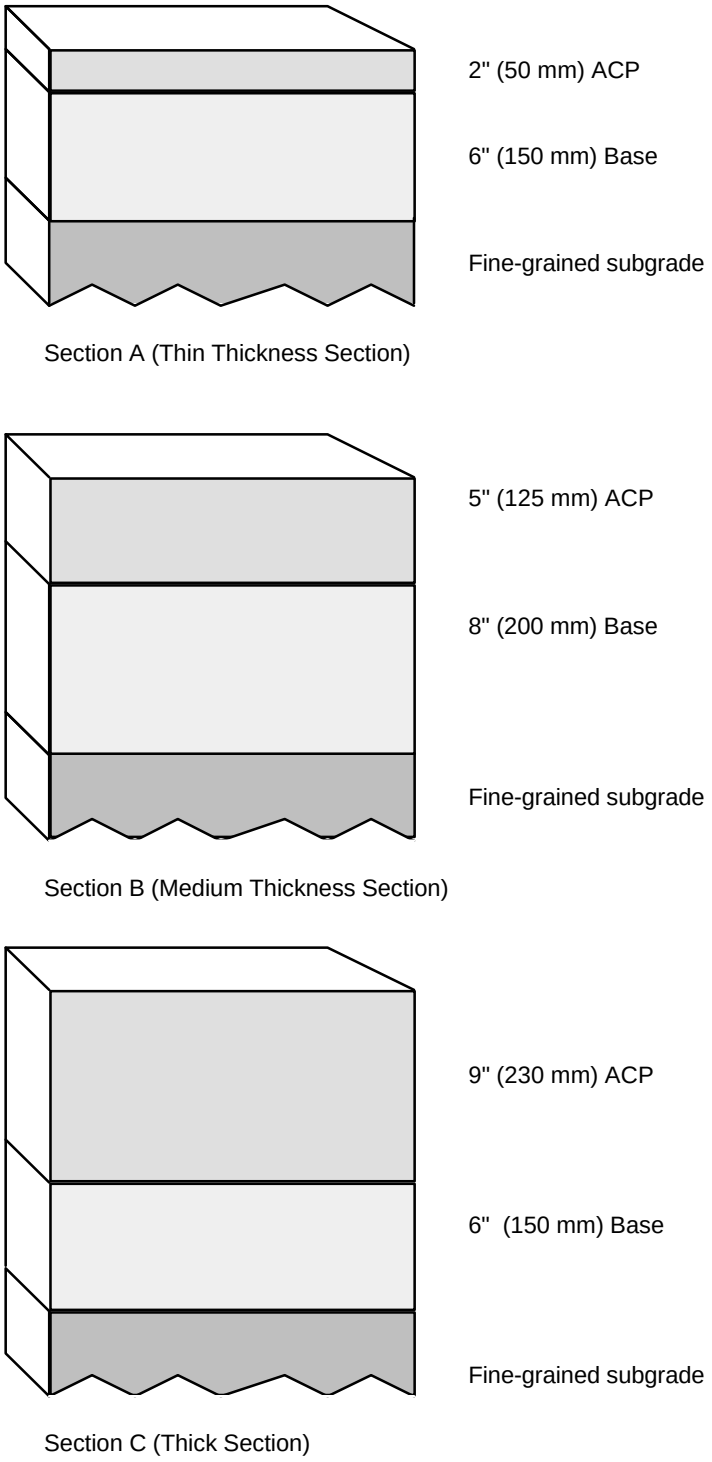


Figure 4.9. "Typical" Pavement Sections

(e) Location of Load: x = 0
y = 0

(f) Location of Analysis Points (all at x=0, y=0)

(i) Surface Deflection (all at z=0)

(ii)	Horizontal Tensile Strain at Bottom of ACP	Thin Pavement:	z = 2 in (50 mm)
		Medium Pavement:	z = 5 in (125 mm)
		Thick Pavement:	z = 9 in (230 mm)
(iii)	Vertical Compressive Strain at Top of Subgrade	Thin Pavement:	z = 8 in (200 mm)
		Medium Pavement:	z = 13 in (330 mm)
		Thick Pavement:	z = 15 in (380 mm)

A layered elastic analysis was performed using the inputs as defined above, and the results are listed in Table 4.1 under the heading "Standard Pavement". As would be expected, the surface deflection, the horizontal strain at the bottom of the AC, and the vertical compressive strain at the top of the subgrade all decrease with increasing pavement thickness. A comparison of FWD basins for the three standard sections is given in Figure 4.10.

In order to demonstrate the sensitivity of the analysis to load and material changes, several other cases were run and compared to the Standard Pavement. These included:

- Low Tire Load: Tire load decreased from 9,000 lb (40 kN) at 80 psi (552 kPa) to 900 lb (4 kN) at 30 psi (207 kPa). This is a comparison of the effects of a truck load versus a passenger vehicle.

- High Tire Pressure: Increase from 80 psi (552 kPa) to 140 psi (965 kPa). This demonstrates the difference in pavement response from a standard truck tire pressure to one which is an extreme.

- Stabilized Subgrade: Increase subgrade modulus from 7,500 psi (52 MPa) to 50,000 psi (345 MPa) in the top 6 in (150 mm) of soil. This case shows the effect of subgrade improvement.

- Asphalt Treated Base: Increase base course modulus from 25,000 psi (172 MPa) to 500,000 psi (3450 MPa). This represents an increase in the structural capacity of the pavement.

- Moisture Sensitive: Decrease AC modulus from 500,000 psi (3450 MPa) to 200,000 psi (1380 MPa). This illustrates a weakening of the AC layer due to stripping in the mixture.

Table 4.1. Sensitivity Analysis of Various Input Parameters

	Pavement Response Parameter	Standard Pavement		Low Tire Load		High Tire Pressure		Stabilized Subgrade		Asphalt Treated Base		Moisture Sensitive	
1.	Surface deflection top of AC	inches	mm	inches	mm	inches	mm	inches	mm	inches	mm	inches	mm
1.1	Section A (thin)	0.048	1.219	0.006	0.152	0.052	1.321	0.036	0.914	0.021	0.533	0.053	1.346
1.2	Section B (med)	0.027	0.686	0.003	0.076	0.028	0.711	0.023	0.584	0.014	0.356	0.033	0.838
1.3	Section C (thick)	0.018	0.457	0.002	0.051	0.019	0.483	0.016	0.406	0.012	0.305	0.024	0.610
2.	Horizontal tensile strain bottom of AC or ATB ($x\ 10^{-6}$)												
2.1	Section A (thin)	469		121		735		369		193		482	
2.2	Section B (med)	279		44		352		246		88		433	
2.3	Section C (thick)	145		17		161		128		67		258	
3.	Vertical compressive strain top of subgrade ($x\ 10^{-6}$)												
3.1	Section A (thin)	-2,239		-284		-2,554		-956		-508		-2,604	
3.2	Section B (med)	-755		-79		-790		-431		-222		-1,037	
3.3	Section C (thick)	-371		-38		-375		-245		-169		-608	

Note: Tension (+)
Compression (-)

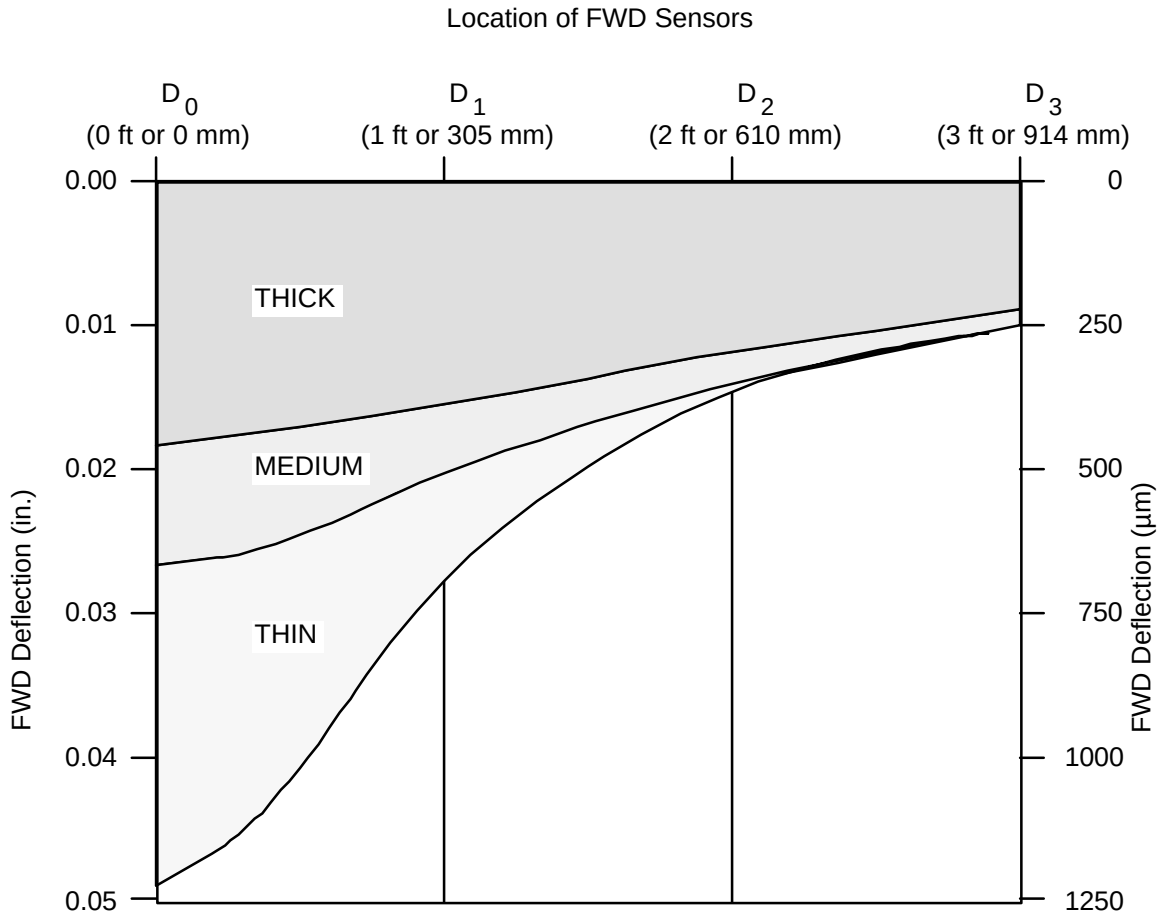


Figure 4.10. Comparison of Deflection Basins for the "Standard" Typical Pavements

Comparisons between the "Standard Pavement" case and the modified input just described are given in Table 4.2.

As would be expected, there is a dramatic decrease (on the order of 75 to 90 percent) in all of the pavement response parameters when a passenger vehicle loading is compared to that of a truck load. These decreases become even more meaningful when they are discussed in the context of pavement life later on in these notes.

Table 4.2. Comparisons of Changes in Pavement Responses from Standard Pavement

	Section	Percent Change from Standard Pavement		
		Surface Deflection	Tensile Strain in AC	Compr. Strain in Subgrade
Low Tire Load	A	-88	-74	-87
	B	-89	-84	-90
	C	-89	-88	-90
High Tire Pressure	A	+8	+57	+14
	B	+4	+26	+5
	C	+6	+11	+1
Stabilized Subgrade	A	-25	-21	-57
	B	-15	-12	-43
	C	-11	-12	-34
Asphalt Treated Base	A	-56	-59	-77
	B	-48	-68	-71
	C	-33	-54	-54
Moisture Sensitive	A	+10	+3	+16
	B	+22	+55	+37
	C	+33	+78	+64

The increase in tire contact pressure from 80 to 140 psi (552 to 965 KPa) results primarily in an increase in the tensile strain in the asphalt concrete. This parameter increased 11 percent for the thick pavement and 57 percent in the thin pavement. It is interesting to note that very little change in the surface deflection or compressive strain in the subgrade resulted from the higher tire pressure. In general, the surface layer of the pavement is sensitive to changes in tire pressure, particularly "thin" surface courses. The lower layers are most sensitive to changes in load.

The incorporation of a stabilized subgrade layer affected the vertical compressive strain in that layer most effectively, although significant improvement in the surface deflection and tensile strain in the asphalt surface can be noted for the thin pavement.

The use of an asphalt treated base greatly influenced the response parameters in all three pavement sections.

The thick pavement, Section C, demonstrated the greatest increase in pavement responses to the presence of a moisture susceptible asphalt mixture. Section A was the least affected of the three. Further illustration of the effects of a weaker surface layer is presented for all three sections in the FWD basins shown Figure 4.11.

2. DESIGN PROCESS

Designing a pavement using a mechanistic-empirical approach is an iterative process which can include the several steps shown in Figure 4.12. These include:

(a) Analyzing traffic for the design period to determine the total number of traffic loads in each season (n).

(b) Computation of strains at critical points in the pavement for each season under consideration.

(c) Calculating the number of cycles to failure (N_f) for each season.

(d) Calculating the damage ratio (n/N_f) for each season.

(e) Summing the damage ratios for all seasons (D).

(f) Increasing or reducing layer thicknesses if D is not close to 1.

(g) Determining final cross-section design.

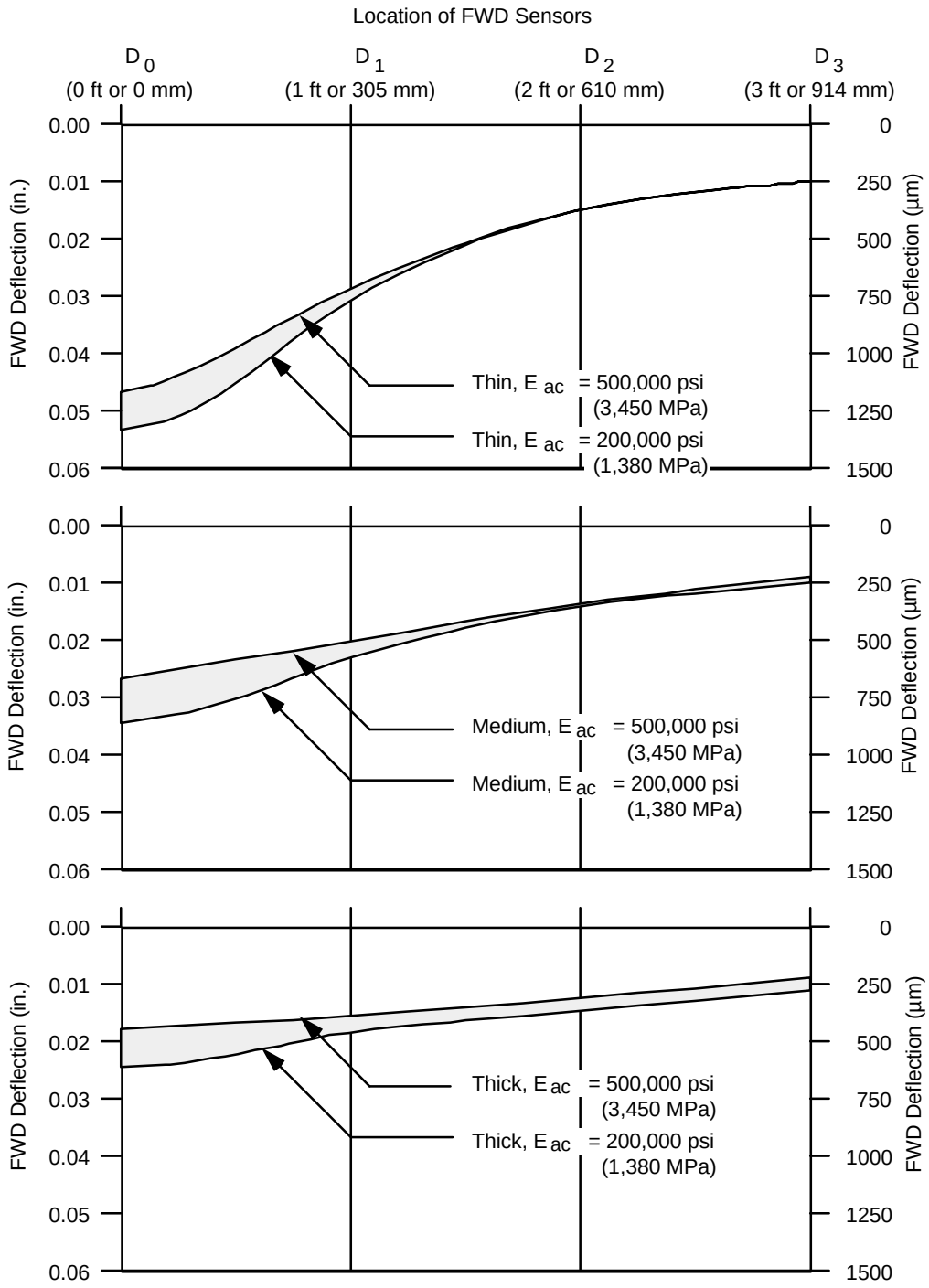


Figure 4.11. Differences in Deflection Basins for the "Standard" vs. "Moisture Sensitive" Pavement Sections

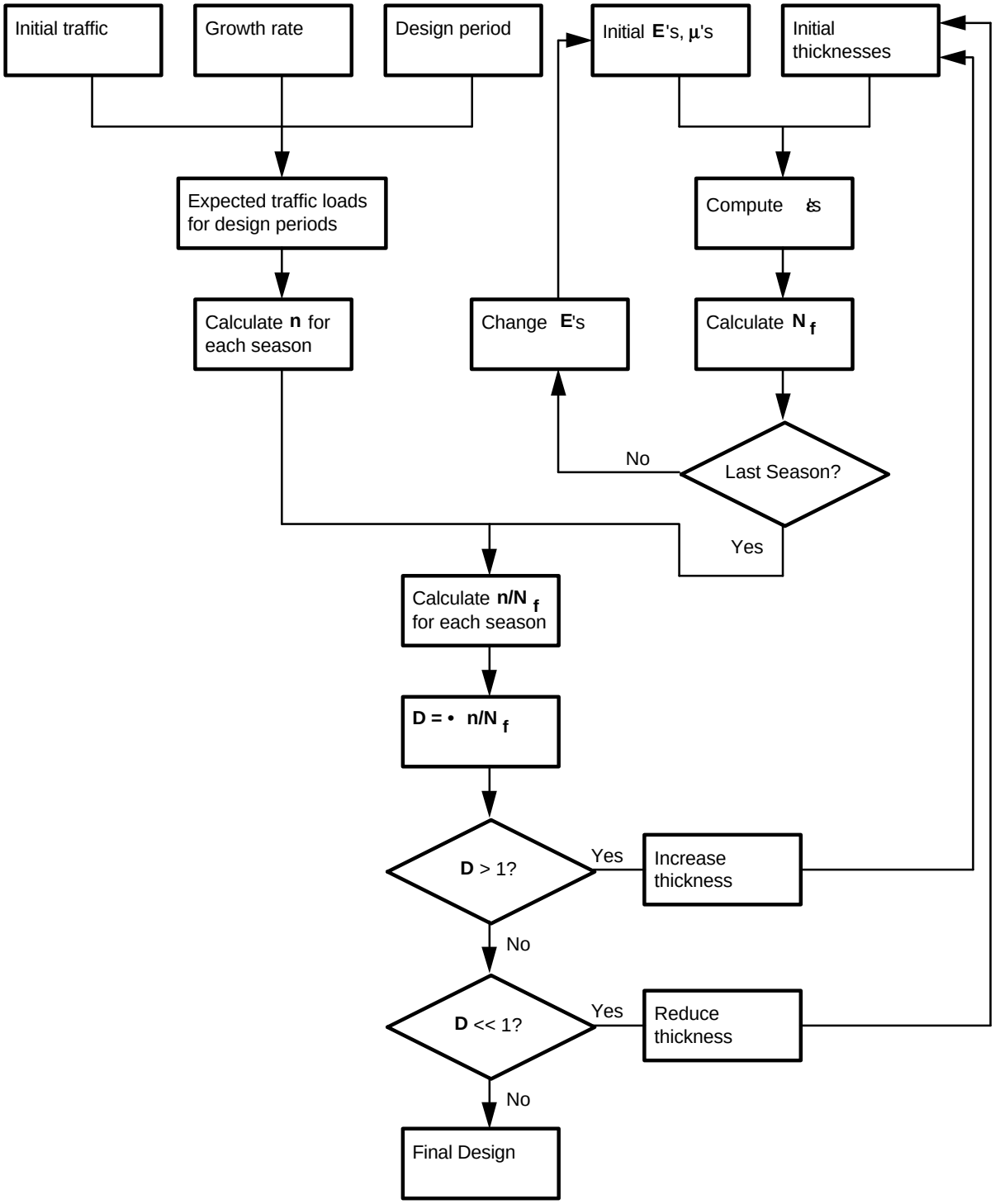


Figure 4.12. Flow Diagram of a Mechanistic-Empirical Design Procedure

At the outset, one should establish the lengths of the seasons to be considered in the design process. This selection should reflect time periods during which the material properties do not change substantially. For instance, if an area has three months of freezing weather, say from December through February, the assumption could be that the base and subgrade materials are frozen (high moduli) and that the asphalt concrete is very stiff (high modulus). During the spring, March and April, the moduli of the underlying layers may be very low, and the asphalt concrete modulus will be an intermediate value. The asphalt concrete modulus will be at its lowest during the summer (May through August) and the base and subgrade will have intermediate moduli. In the fall (September through November), the lower layers will probably maintain intermediate stiffnesses, and the asphalt concrete will again have an intermediate modulus. An example of this kind of information is shown below:

Season	Season Length	Relative Modulus of Elasticity		
		AC	Base	Subgrade
Winter	3 months	high	high	high
Spring	2 months	inter	low	low
Summer	4 months	low	inter	inter
Fall	3 months	inter	inter	inter

The traffic analysis is normally accomplished by using the initial traffic volume, an assumed growth rate, and the design period to calculate the number of traffic loads on the pavement over its life. For a highway pavement, this normally results in a number of 18,000 lb (80 kN) equivalent single axle loads (ESALs). However, it should be noted that since any realistic, mixed traffic loading condition can be used in layered elastic analysis, one is not restricted solely to an ESAL configuration. For the purposes of illustration, we will use the loading condition in the previous example for the "Standard Pavement". Recall that this was a 9,000 lb (40 kN) wheel with 80 psi (552 kPa) contact pressure. If n_T represents the total number of this type of load expected over the design life, then the seasonal distribution can be estimated:

Winter: $n_w = (3/12)\ n_T$
Spring: $n_{sp} = (2/12)\ n_T$
Summer: $n_s = (4/12)\ n_T$
Fall: $n_f = (3/12)\ n_T$

In this example, we are assuming that the expected traffic loads will be distributed evenly throughout the year. If there are obvious seasonal patterns in the traffic, spring load restrictions for instance, then an adjustment would have to be made in the calculation to account for this.

A layered elastic analysis would need to be performed for the assumed loading condition for each season being considered. The loading condition as it would appear in the layered elastic analysis is illustrated in Figure 4.5.

The initial thicknesses could be selected as the minimum allowable thicknesses for given classes of pavement. The critical strains in the pavement structure are determined at the locations noted in Figure 4.5. These strains are calculated for each seasonal change in modulus values for the various layers.

Next, the number of load cycles to failure (N_f) for fatigue cracking and rutting are computed using failure criteria. These will be subsequently discussed in detail. For the fatigue criterion, the number of cycles to failure is a function of the horizontal tensile strain at the bottom of the asphalt concrete and the modulus of the asphalt concrete. For rutting, it is a function of the vertical compressive strain at the top of the subgrade. The number of cycles to failure is computed for each season.

The damage for each season is defined as the ratio of the number of loads expected for that time period, n, to the number of cycles to failure, N_f, for that condition in the pavement. The total damage, D, over the design life is then computed by summing the damage in each season:

$$D = n_w/N_{fw} + n_{sp}/N_{fsp} + n_s/N_{fs} + n_f/N_{ff}$$

If the total damage is close to 1, it means that close to 100 percent of the pavement life has been expended. If D is greater than 1, then the pavement has been underdesigned, and the layer thicknesses should be increased. If the damage is much less than one, then the opposite is true.

The more general form to sum pavement damage due to either seasonal moduli changes and/or various loads is:

$$D = \sum_{i=1}^{g} n_i/N_i$$

where

n_i = <u>actual</u> loads due to condition i,

N_i = <u>allowable</u> loads due to condition i, and

g = number of moduli and load conditions evaluated.

Thus, each condition of seasonal moduli (hence different pavement response) and/or load can be summed. When D approaches 1, failure is estimated to occur. D is calculated <u>separately</u> for each type of distress evaluated. Thus N is separately estimated for distress types such as fatigue cracking and rutting. An illustration of how this might work for two

AC stiffnesses (say 400 ksi and 1000 ksi) and two loads conditions (say dual tires with P = 4500 lb per tire and a single tire with P = 6000 lb per tire):

- Actual Loads
  - n_1 for AC with 400 ksi and dual tires with P = 4500 lb
  - n_2 for AC with 1000 ksi and dual tires with P = 4500 lb
  - n_3 for AC with 400 ksi and single tire with P = 6000 lb
  - n_4 for AC with 1000 ksi and single tire with P = 6000 lb
- Allowable Loads
  - N_1 based on critical strain for n_1 condition
  - N_2 based on critical strain for n_2 condition
  - N_3 based on critical strain for n_3 condition
  - N_4 based on critical strain for n_4 condition

Thus, $D = {}^{n_1}/_{N_1} + {}^{n_2}/_{N_2} + {}^{n_3}/_{N_3} + {}^{n_4}/_{N_4}$

Further, the process is repeated for each distress type evaluated via separate failure criteria. Some of these failure criteria are presented in Paragraph 3 which follows.

3. FAILURE CRITERIA

The main empirical portions of the design process are the equations used to compute the number of loading cycles to failure. These are derived by observing the performance of pavements and relating the type and extent of observed failure to an initial strain under various loads. Currently, two types of failure criteria are widely recognized, one relating to fatigue cracking and the other to rutting initiating in the subgrade. A third criterion is deflection based and may be of use in special applications.

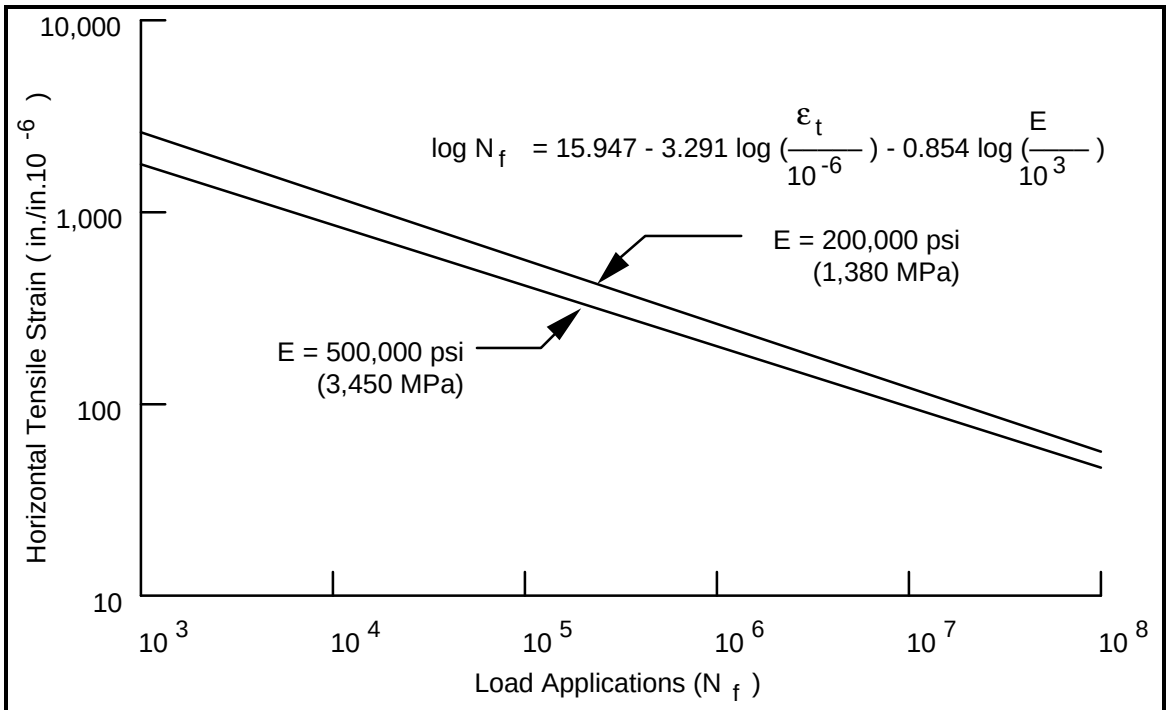


Figure 4.13. Limiting Horizontal Strain Criterion for Asphalt Concrete Fatigue Cracking

3.1 TYPES OF CRITERIA

Many equations have been developed to estimate the number of repetitions to failure in the fatigue mode for asphalt concrete. All of these rely on the horizontal tensile strain at the bottom of the asphalt layer, ε_t , and the modulus of the asphalt mix, E. One commonly accepted criterion was suggested by Finn et al. [4.8]:

$$\log N_f = 15.947 - 3.291 \log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log\left(\frac{E}{10^3}\right)$$

The above equation assumes that failure is defined as fatigue cracking over 10 percent of the wheelpath area. Figure 4.13 shows the relationship between tensile strain in the asphalt concrete and the number of cycles to failure for two levels of asphalt concrete modulus.

Rutting can initiate in any layer of the structure, making it more difficult to predict than fatigue cracking. Current failure criteria are intended for rutting which can be attributed mostly to an overstressed subgrade. This is typically expressed in terms of the vertical compressive strain, ε_v, at the top of the subgrade layer:

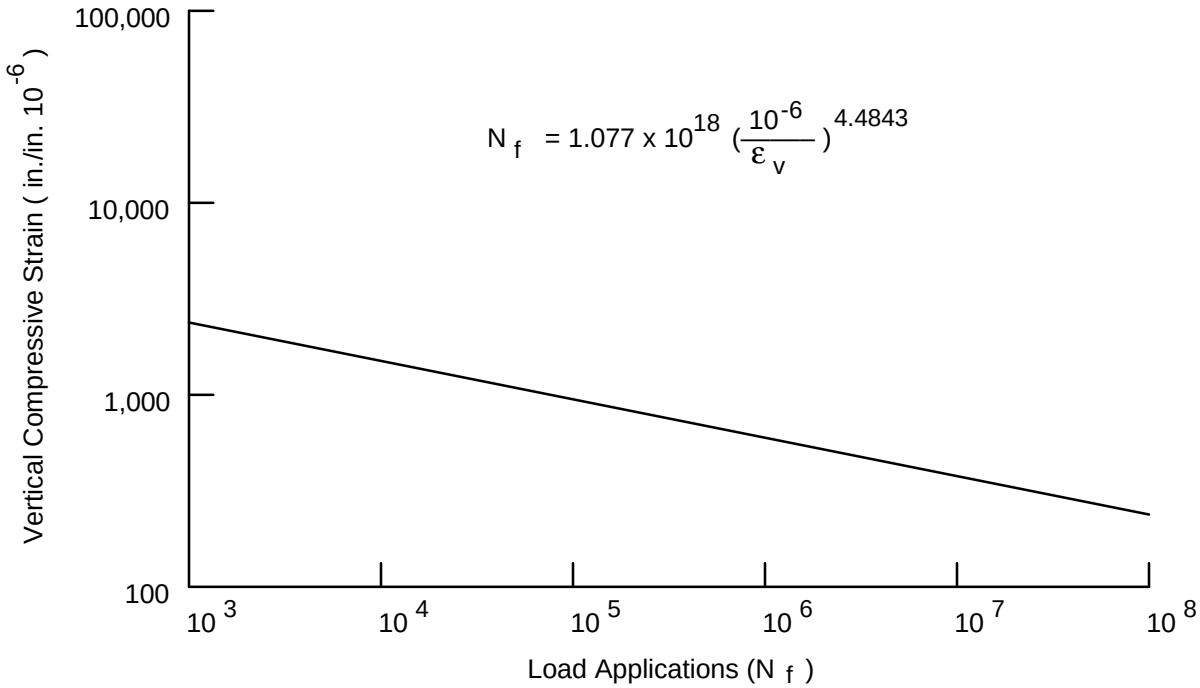


Figure 4.14. Limiting Subgrade Strain Criterion for Rutting

$$N_f = 1.077 \times 10^{18} \left(\frac{10^{-6}}{\varepsilon_v}\right)^{4.4843}$$

Failure in this case is defined as 0.5 in. (13 mm) depressions in the wheelpaths of the pavement. Figure 4.14 illustrates how the vertical compressive strain relates to the number of cycles to failure.

The number of cycles to failure for our example pavement sections are given in Table 4.3 for both fatigue and rutting. The lower number of the two criteria indicates the expected mode of failure. The "Standard Pavement" with the thin cross-section for example is expected to fail after about 1,000 cycles in rutting. The medium section, B, of the same case would show rutting after 130,000 load repetitions. Section C of the "Standard Pavement" would be expected to fail after about 3.3 million cycles in either fatigue or rutting. The "Low Tire Load" results indicate that structural failure of the pavements is unlikely for any of the pavement sections. Increased tire pressure primarily affects the fatigue life of all three sections, although sections A and B will still fail in rutting first. Subgrade stabilization is a better aid to rutting failure than to fatigue distress. The use of an asphalt treated base dramatically benefits all three sections from a structural viewpoint. The weakening of the pavement due to the presence of a moisture sensitive asphalt mixture has its most severe effect on the rutting failure. The basic conclusion that can be drawn from Table 4.3 is that structural failure for thin (Section A) pavements and weak ("Moisture Sensitive") pavements will be in rutting according to these criteria.

A third criterion relates pavement surface deflections to loads to failure. Although WSDOT does not normally use such a failure criterion, it may be of use in special applications. A number of deflection based criteria have been developed by various agencies over the last 40 years or so. Three such criteria will be shown (AASHO Road Test, Asphalt Institute, and Roads and Transportation Association of Canada (RTAC)). All three of these criteria were developed based on spring seasonal deflections.

The AASHO Road Test results were used to develop the following relations [4.11]:

$$\log W_{2.5} = 9.40 + 1.32 \log L_1 - 3.25 \log d_{sn}$$

where $\log W_{2.5}$ = number of applications of axle load L_1 sustained by the pavement to a terminal serviceability index of 2.5,

L_1 = single axle load (kips), and

d_{sn} = Benkelman Beam springtime measured pavement surface deflection (0.001 in.) measured at the AASHO Road Test (Spring 1959) after "disappearance of frost."

This criterion was based on data from Loops 2 through 6 and single axle loads of 6, 12, 18, 22.4, and 30 kips (refer to SECTION 5.0, Paragraph 1 for more details on the AASHO Road Test). The following equation is obtained if L_1 = 18 kips (a standard ESAL):

$$\log W_{2.5} = 11.06 - 3.25 \log d_{sn}$$

Table 4.3. Failure Criteria Applied to Typical Pavements

Millions of Cycles to Failure							
Section	Criterion	Standard Pavement	Low Tire Load	High Tire Pressure	Stabilized Subgrade	Asphalt Treated Base	Moisture Sensitive
A	Fatigue	0.07	6.10	0.02	0.15	1.31	0.14
	Rutting	0.001	10.73	0.001	0.05	0.79	0.001
B	Fatigue	0.39	170.37	0.18	0.59	17.41	0.20
	Rutting	0.13	3,331.86	0.11	1.65	32.39	0.03
C	Fatigue	3.36	3,895.76	2.38	5.07	42.70	1.10
	Rutting	3.24	88,714.00	3.09	20.82	110.08	0.35

Two other deflection criteria will be shown. These were developed by the Asphalt Institute [4.12] and RTAC [4.37] for various 80 kN (18,000 lb) loads to failure, the associated limiting deflections for all three criteria are:

	Limiting "Spring" Deflection (in.)		
Loads to Failure	AASHO Road Test	Asphalt Institute	RTAC
10,000	0.148	0.110	0.100
100,000	0.072	0.062	0.080
1,000,000	0.036	0.034	0.040
10,000,000	0.018	0.020	0.020

The results are similar for the three criteria. The Asphalt Institute and RTAC use spring rebound deflections calculated as the mean plus two standard deviations ($\bar{x}$ + 2s) at a mean asphalt temperature of 21 °C (70° F). (Using the mean plus two standard deviations implies that a bit over two percent of the "spring period" deflections would be larger.) The AASHO results were obtained at an asphalt concrete surface temperature of 15° C (59°F). The three criteria plot as straight lines on a log-log plot except for the RTAC criterion between 10,000 and 100,000 load repetitions. More specifically, the RTAC criterion can be calculated as follows (after RTAC [4.31] and Haas et al. [4.33]):

$$BB = 10^{[0.40824 - 0.30103 (\log ESAL)]}$$

where BB = maximum rebound deflection (in.) ($\bar{x}$ + 2s) at a standard temperature of 21° C (70° F),

ESAL = 80 kN (18000 lb) single axle loads,

BB = 0.100 in. for ESAL 47,651, and

BB = 0.02 in. for ESAL > 10,000,000.

3.2 SENSITIVITY OF DESIGN TO FAILURE CRITERIA

To illustrate the impact of failure criteria on design, Figures 4.15 through 4.17 were developed using the material parameters and loading conditions in the previous example for the "Standard Pavement". Various thicknesses of bituminous surface were plotted against the computed number of repetitions to failure for 6 in. (150 mm), 10 in. (250 mm), and 14 in. (350 mm) thick granular bases. The controlling criterion in the design is the one requiring a greater thickness of surface material for a given traffic level. Keep in mind that these charts were developed holding the material properties constant; they are not to be used in actual design.

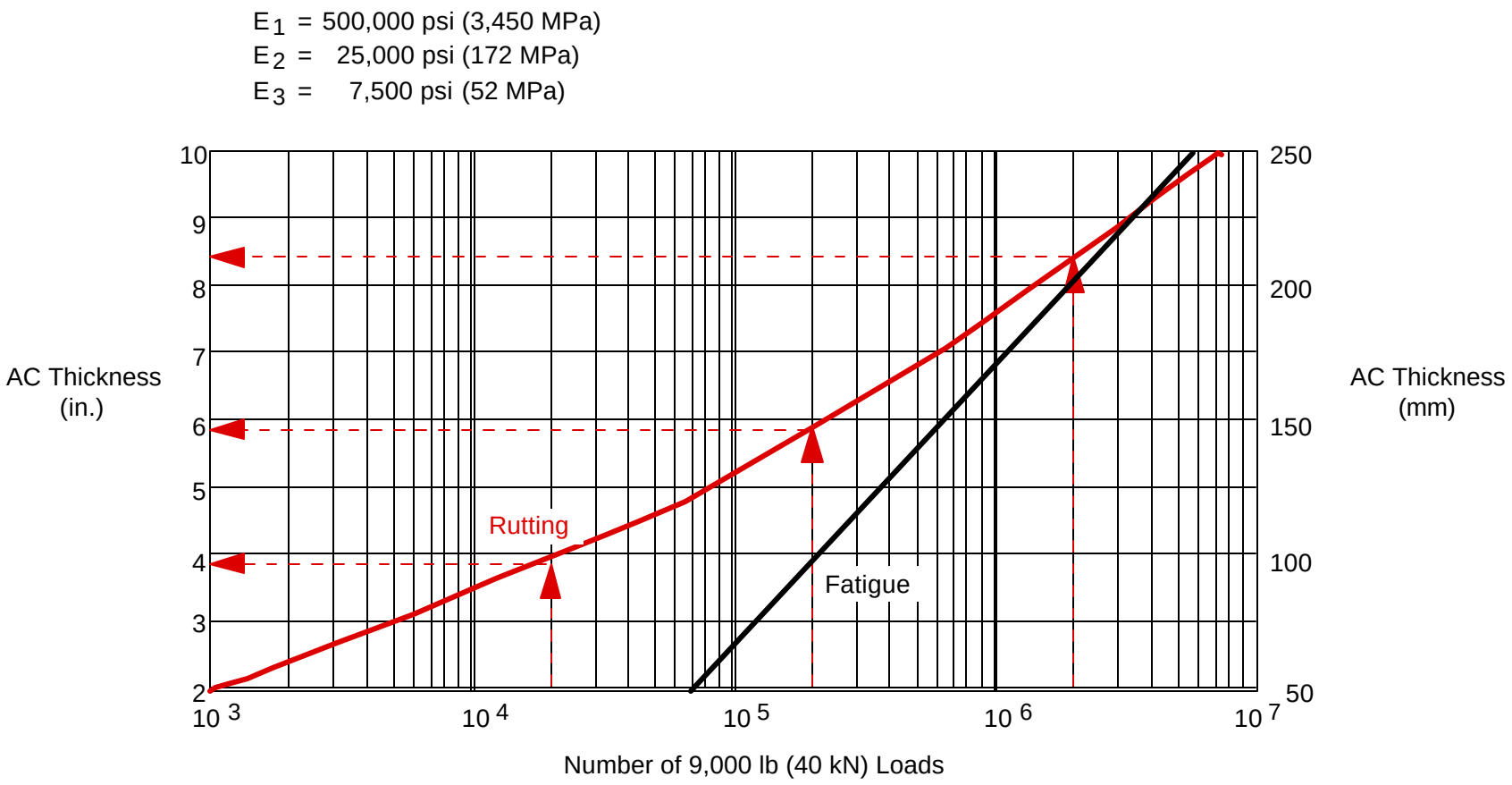


Figure 4.15. Design Chart of Example Pavement (6 in (150 mm) Base)

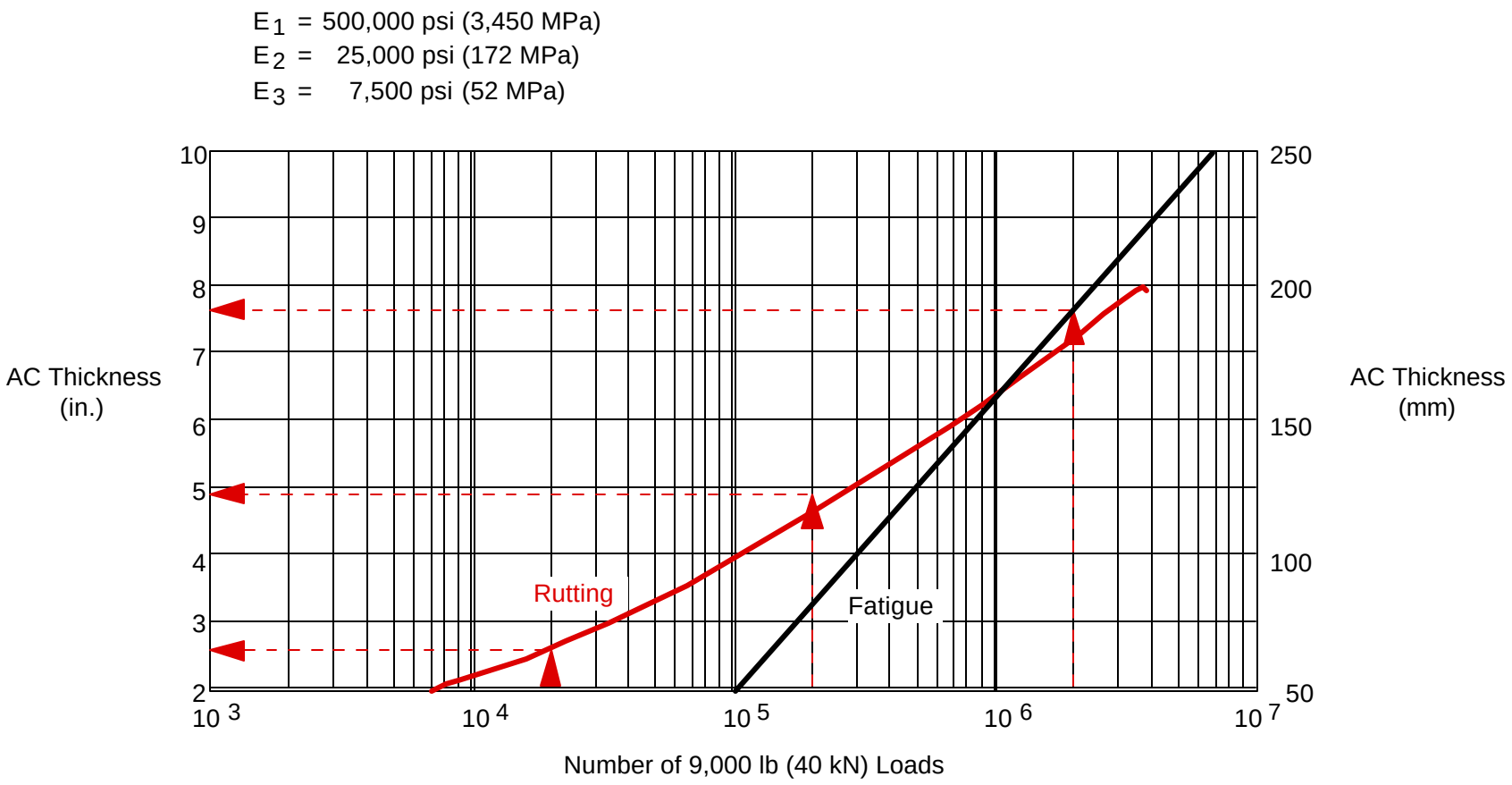


Figure 4.16. Design Chart of Example Pavement (10 in (250 mm) Base)

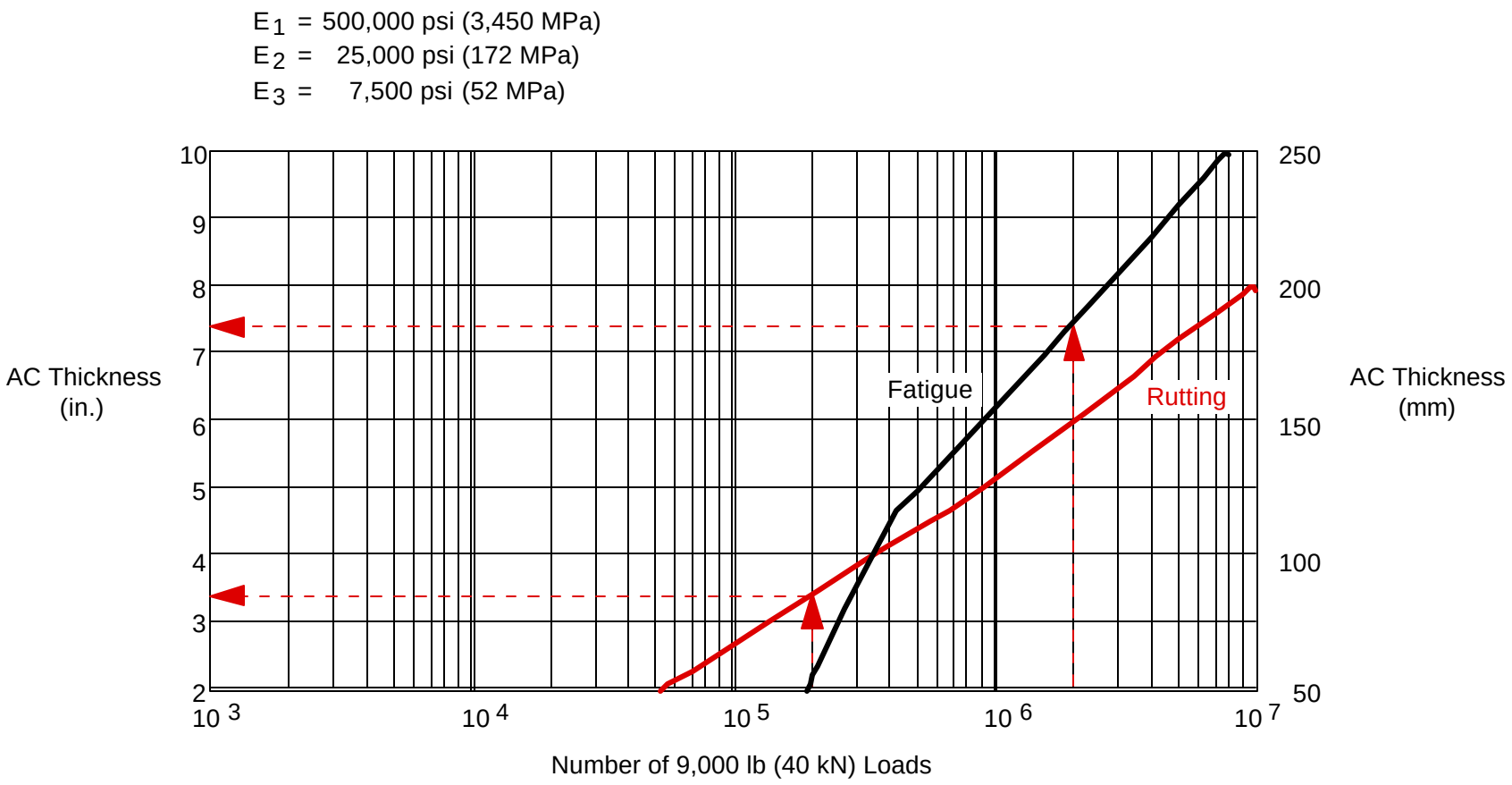


Figure 4.17. Design Chart of Example Pavement (14 in (350 mm) Base)

Table 4.4. Comparison of Example Designs

9,000 lb (40 kN) Wheel Loads	Base Thickness		AC Thickness		Controlling Criterion
	in.	mm	in.	mm	
2×10^4	6	150	4.0	100	Rutting
	10	250	2.5	60	Rutting
	14	360	2.0*	50	Rutting
2×10^5	6	150	6.0	150	Rutting
	10	250	5.0	130	Rutting
	14	360	3.5	90	Rutting
2×10^6	6	150	8.5	220	Rutting
	10	250	8.0	200	Fatigue
	14	360	7.5	190	Fatigue

* Assumed minimum thickness

The pavement designs resulting from three traffic levels are given in Table 4.4. In this table, the resulting thickness of asphalt concrete was rounded to the nearest 0.5 in (12.5 mm), and the assumed minimum thickness of the bituminous layer was 2 in. (50 mm). It is interesting to note that the rutting criterion controlled the design in all cases except those involving the thicker two pavement sections at 2,000,000 load repetitions. The choice of which of the three equivalent pavement designs to use at any of the traffic levels would depend on a life-cycle cost analysis.

4. MECHANISTIC-EMPIRICAL DESIGN PROCEDURES

4.1 INTRODUCTION

Several pavement design methods will be briefly overviewed. The pavement design procedures for all-new or reconstructed pavements includes the Asphalt Institute MS-1 (Section 4.2) and the Shell method (Section 4.3). The WSDOT mechanistic-empirical procedure (Section 4.4) is briefly overviewed (more detail about the WSDOT procedure is provided in SECTION 7.0).

All of the design procedures which follow use layer moduli in some manner.

4.2 ASPHALT INSTITUTE MS-1 [4.9]

The Asphalt Institute presents a mechanistic based design procedure for streets and highways. The following steps are used in this approach:

(a) Determine the initial inputs:

(i) Compute the expected number of ESALs for the design period.

(ii) Determine the design resilient modulus of the subgrade. This is where the conservatism is built into the design. One selects a resilient modulus value which falls below a specified percentile of test results for that section of road.

(iii) Select combinations of layer materials. These may include:

- Full-Depth Asphalt Concrete
- AC over Emulsified Asphalt Base
- AC over Untreated Aggregate Base

(b) Find layer thickness combinations for the materials selected.

(c) Account for stage construction, if used. This is done on the basis of remaining life in the pavement.

(d) Perform an economic analysis of the various pavement sections.

(e) Determine the final design.

An example design chart from MS-1 is shown in Figure 4.18 which can be used to determine the asphalt concrete thickness for a 6 in. (150 mm) untreated granular base course. In order to use it, one finds the intersection of the design subgrade modulus and the number of ESALs for the design period. A brief description of the two principal failure criteria used in the AI procedure follow.

4.2.1 SUBGRADE STRAIN CRITERION [4.17]

The subgrade strain criterion used in the AI procedure was stated in Paragraph 3.1 (this section). The criterion was originally developed by the Chevron Co. (Santucci [4.18]). The basic relationship is:

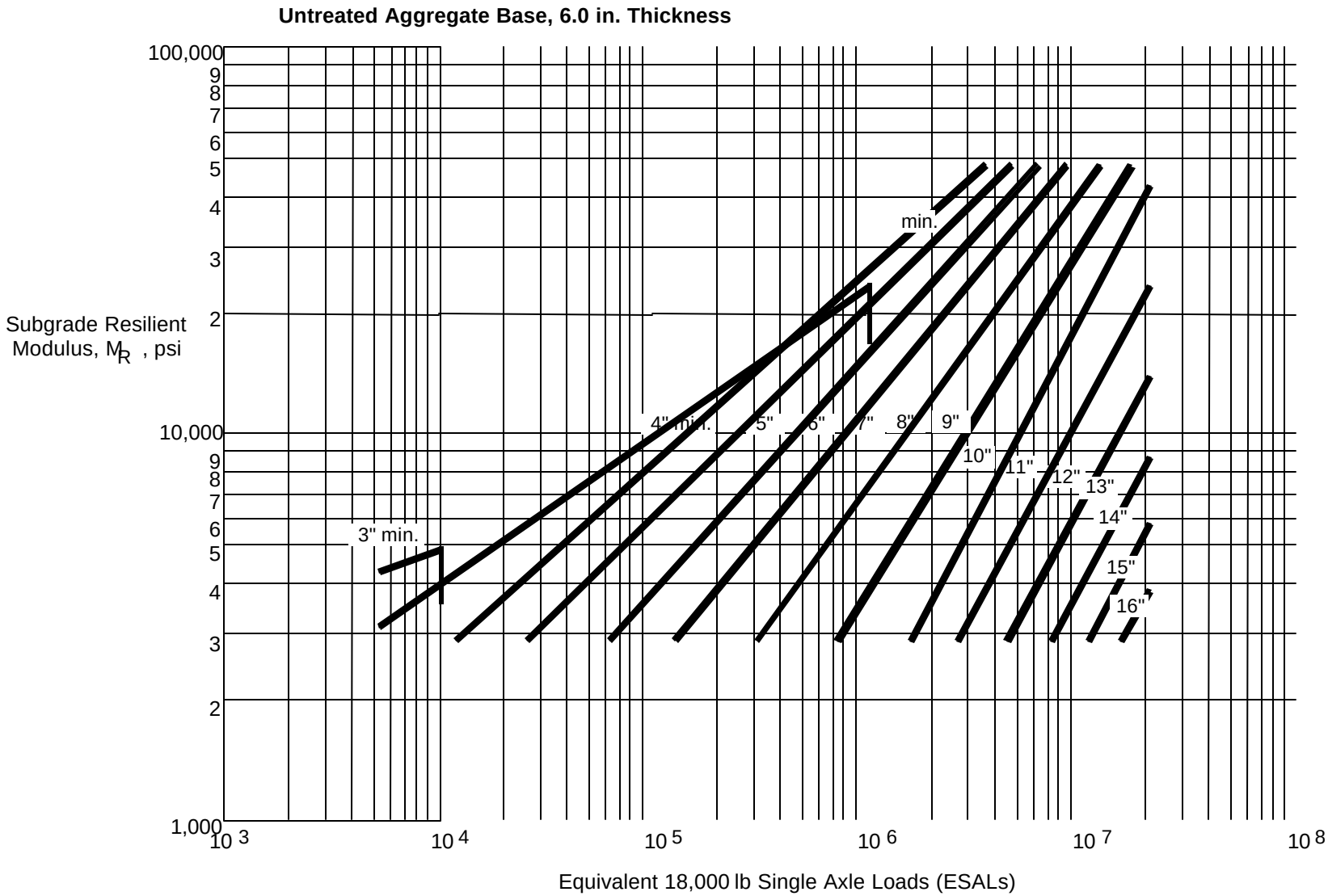


Figure 4.18. Example of an Asphalt Institute Design Chart
[Redrawn from Ref. 4.9]

$$\varepsilon_v = 1.05 \times 10^{-2} (1/N)^{0.223} \qquad \text{(Eq. 4.1)}$$

where ε_v = vertical compressive strain at the subgrade surface, and

N = number of load applications which should not result in more than 0.5 in. (13 mm) of rutting at the pavement surface.

Rewriting Equation 4.1 results in

$$N_f = \left[\frac{1.05 \times 10^{-2}}{\varepsilon_v}\right]^{4.4843}$$

where N_f = allowable number of load applications so that rutting at the pavement surface should not exceed 0.5 in. (13 mm), and

ε_v = vertical compressive strain at the top of the subgrade layer.

4.2.2 ASPHALT CONCRETE TENSILE STRAIN [4.17]

The permissible number of load applications in the asphalt layer is a function of horizontal tensile strain and asphalt mixture characteristics (stiffness, volume of asphalt and volume of air voids). The basic fatigue relationship which is used was developed by Finn, et al. [4.8]. This model is based on Monismith's laboratory work [4.14] and the results of the AASHO Road Test.

$$N_f = 10^{\left[15.947 - 3.291 \log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log\left(\frac{E_{ac}}{10^3}\right)\right]} \qquad \text{(Eq. 4.2)}$$

where N_f = number of axle applications to result in 10 percent or less fatigue cracking in the wheelpath area,

ε_t = horizontal tensile strain at the bottom of the AC layer, and

E_{ac} = modulus of the AC layer (psi)

Equation 4.2 is then multiplied by the following factor to reflect differences in asphalt and air void contents [4.17].

$$C = 10^M$$

where C = function of air voids (V_v) and asphalt volume (V_b)

$$M = 4.84\left[\frac{V_b}{V_v + V_b} - 0.69\right]$$

V_b = volume of asphalt, and

V_v = volume of air voids

The term "M" was obtained from laboratory fatigue data. (C = 1 when V_b = 11% and V_v = 5% (low-normal air voids); C = 0.29 when V_b = 11% and V_v = 8% (medium-acceptable voids); C = 0.16 when V_b = 11% and Vv = 10% (high-unacceptable voids).)

4.3 THE SHELL METHOD [4.10, 4.16]

This flexible pavement mechanistic-empirical approach (released for use in 1978) is based on a linear elastic multilayered system. The layer materials are characterized by E and μ. The limiting criteria for design include (as shown in Figure 4.19):

- horizontal tensile in the asphalt concrete layer,
- vertical compressive strain at the top of the subgrade,
    - tensile stress in cementitious base course layers (if used in the pavement structure), and
- permanent deformation of the asphalt concrete.

Further, the design procedure can be used to examine the effects of horizontal loads and layer interfaces with variable friction. A brief description of two of the principal failure criteria used in the Shell method follow.

4.3.1 SUBGRADE STRAIN CRITERION [4.16]

The Shell subgrade strain criterion was developed by use of the BISAR layered elastic computer program, the standard design load (18,000 lb (80kN) single axle with dual tires) and some of the test sections at the AASHO Road Test. The criterion below assumes the number of axle loads associated with a Present Serviceability Index (PSI) = 2.5.

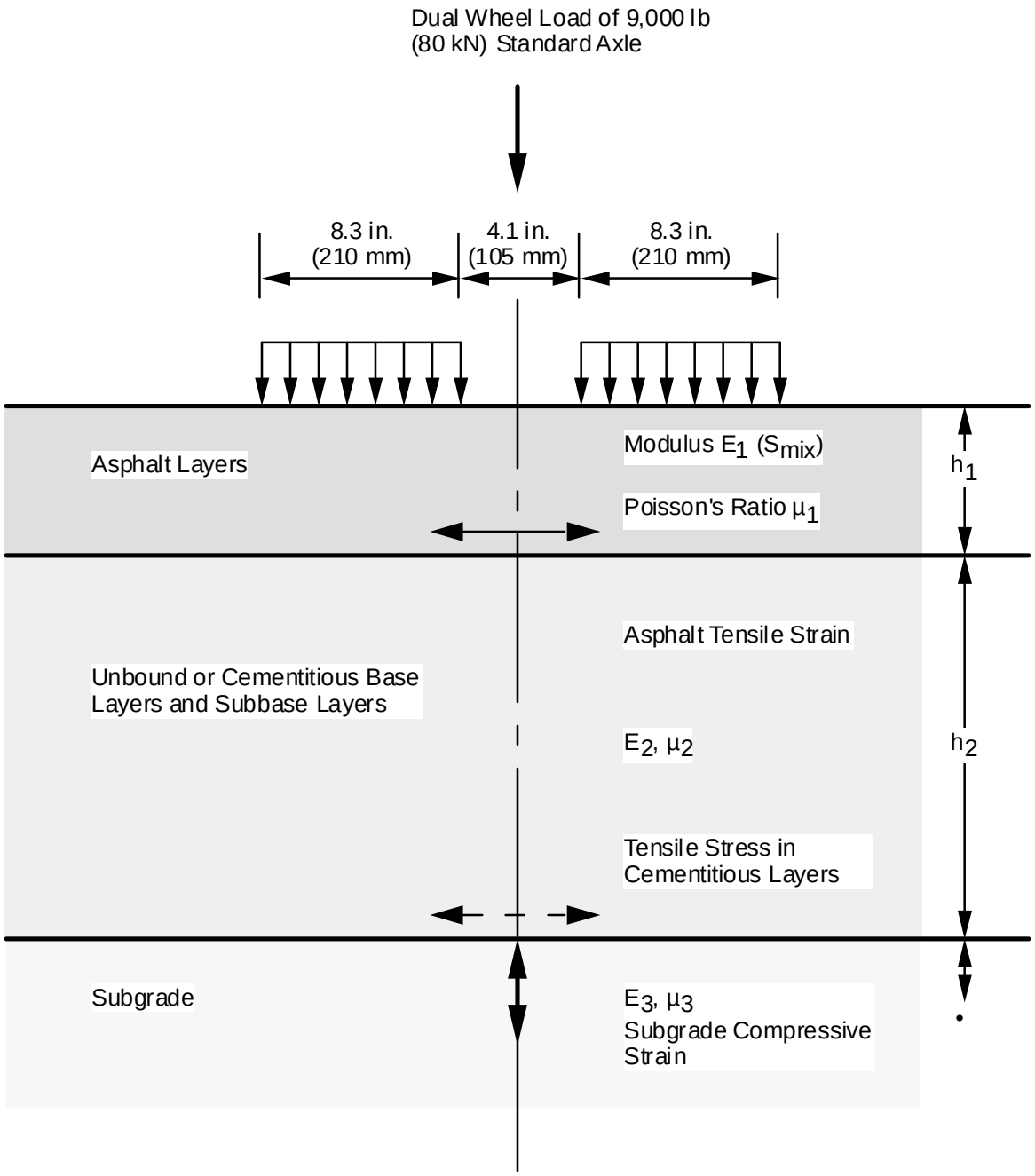


Figure 4.19. Design Load and Criteria—Shell Procedure [after Ref. 4.16]

$$\varepsilon_3 = 2.8 \times 10^{-2} (N)^{-0.25}$$

where ε_3 = permissible compressive strain in subgrade,

N = number of strain repetitions,

or rewriting $$N = \left[\frac{2.8 \times 10^{-2}}{\varepsilon_3}\right]^4$$

4.3.2 ASPHALT CONCRETE TENSILE STRAIN [4.16]

The permissible tensile strain in the asphalt layer is a function of mixture stiffness and the number of strain repetitions.

4.4 WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

A mechanistic-empirical overlay design procedure (EVERPAVE) was developed by the Washington State Department of Transportation. The procedure requires the use of layer moduli (usually backcalculated using nondestructive testing) and the calculation of strains to preclude fatigue and rutting failures. In this approach, layer moduli can be calculated for each deflection test point. The asphalt concrete modulus is corrected for temperature according to data for typical Washington mixtures (Figure 4.20). Next, an iterative process is used to determine an appropriate overlay thickness for each deflection test point as shown in Figure 4.21.

Both the unstabilized base course (subbase) and subgrade moduli can be non-linear with stress state, i.e., the base, subbase, and subgrade layer moduli can take the following form:

$$E = K_1 (\theta)^{K_2}$$

or

$$E = K_3 (\sigma_d)^{K_4}$$

with the exponents being either positive or negative.

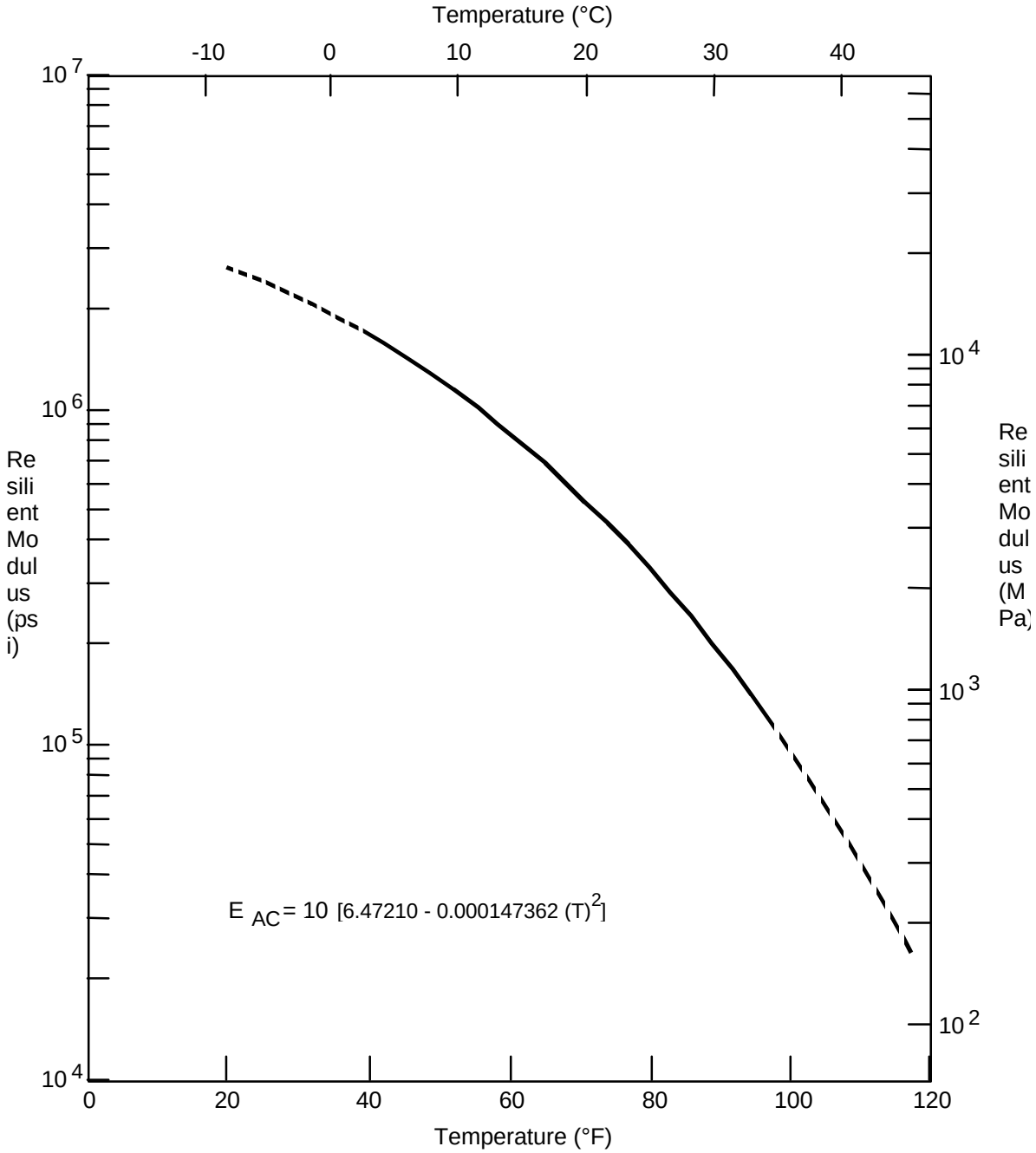


Figure 4.20. General Stiffness-Temperature Relationship for Class B (Dense Graded) Asphalt Concrete in Washington State [from Reference 2.1]

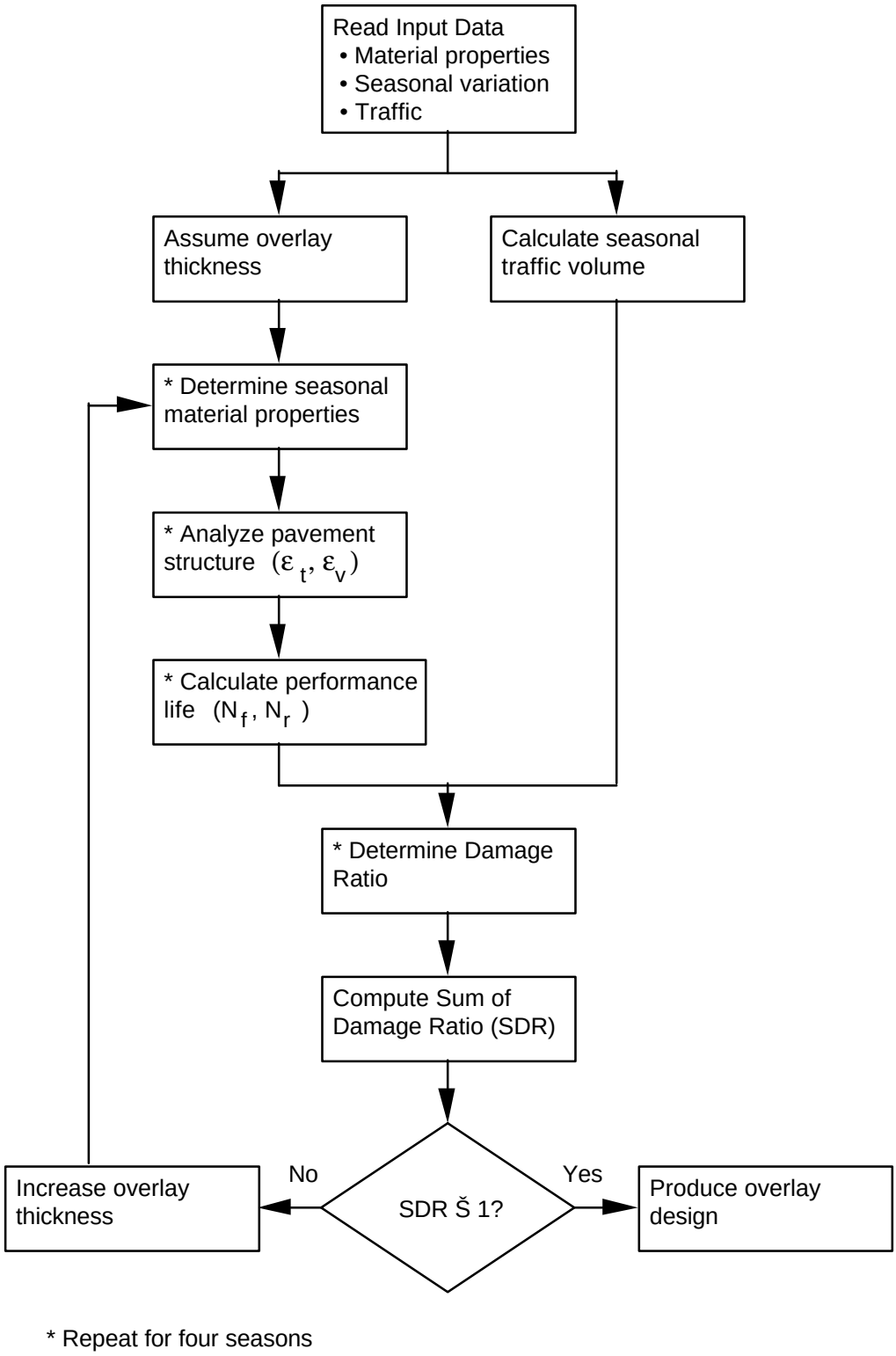


Figure 4.21. WSDOT Overlay Design Flow Chart

The failure criteria used in EVERPAVE are based on two criteria:

Rutting: $N_f = 1.077 \times 10^{18} \left(\frac{1}{\varepsilon_v}\right)^{4.4843}$

as previously described in this section, paragraph 3.1, and

Fatigue cracking: $N_{field} = (N_{lab})(SF)$

where N_{field} = loads to failure for field conditions,

N_{lab} = relationship from laboratory data [2.13, 2.14]

$$= 10^{\left[14.82 - 3.291 \log\left(\frac{e_t}{10^{-6}}\right) - 0.854 \log\left(\frac{E_{ac}}{10^{3}}\right)\right]}$$

SF = can range from about 4 to 10, depends on AC thickness, ESAL level, climate. The SF is in the lower portion of the range for original AC thicknesses greater than 7 in.(180 mm) (due to max tensile strains being in the upper portion of the layer).

5. USE OF ELASTIC ANALYSIS SOFTWARE

Use of WSDOT developed layered elastic software will be illustrated in Volume 3 of this Guide.

An understanding of calculated pavement responses (pavement deflection, stress, and strain) is important. WSDOT uses several computer programs which are based on these principles in pavement design and analysis.

6. STRESSES IN RIGID PAVEMENTS

6.1 INTRODUCTION

There are a variety of stresses which occur in rigid pavements which affect its performance (both initial and longterm). These stresses include

(a) Warping,
(b) Shrinkage/Expansion,
(c) Wheel load related, and
(d) Other.

Each of the three major stress types will be briefly presented and discussed.

6.2 WARPING STRESSES

6.2.1 INTRODUCTION

Differences in temperature between the top and bottom surfaces of a concrete slab will cause the slab to warp (illustrated in Figure 4.22). The weight of the slab and its contact with the subbase restrict its movement — thus, stresses are created.

Measurements reported by Teller and Southerland [4.19] of the Bureau of Public Roads in 1935 (almost 60 years ago) showed that the maximum temperature differential (hence, maximum warping) is much larger during the day than during the night. Further, during the day, the upper surface of the slab is at a higher temperature than the bottom resulting in tensile stresses at the bottom of the slab (again, refer to Figure 4.22).

To evaluate the tensile warping stresses which develop in the slab, the temperatures at the top and bottom of the slab must be estimated. The first formulas used to estimate warping stresses were developed by Bradbury [4.20]:

$$\sigma_t = \frac{(C)(E)(e)(\Delta T)}{2} \qquad \text{Eq. 4.3}$$

where σ_t = slab edge warping stress (psi),

E = modulus of elasticity of PCC (psi),

e = thermal coefficient of PCC

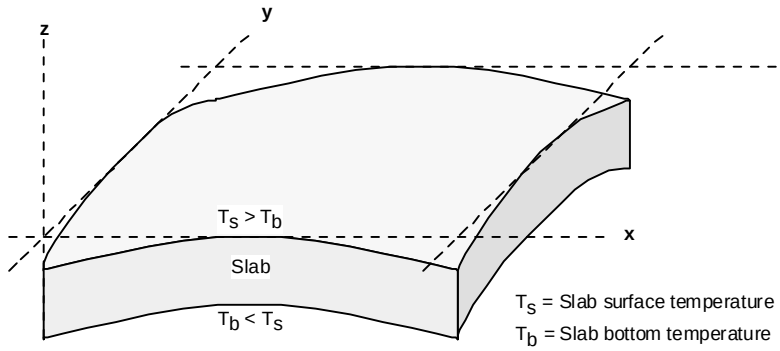
$\cong$ 0.000005/°F,

ΔT = temperature differential between the top and bottom of the slab (°F) and

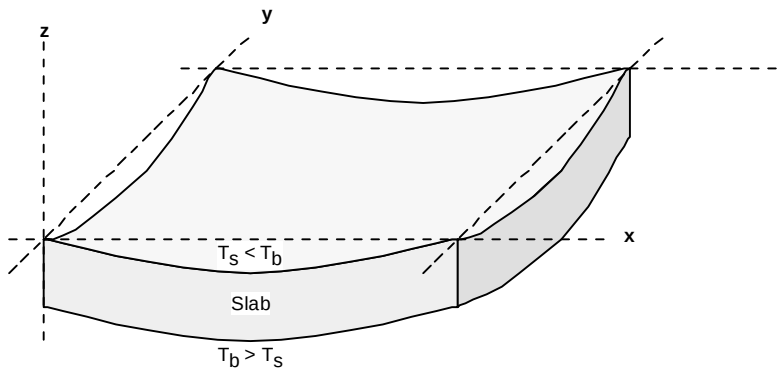
C = coefficient which is a function of slab length and the radius of relative stiffness (shown in Figure 4.23).

For interior stresses, Bradbury's formula is

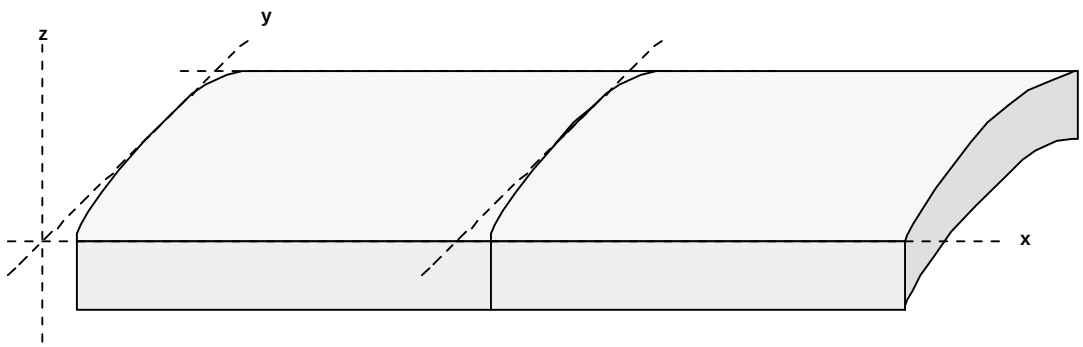
$$\sigma_t = \frac{(E)(e)(\Delta T)}{2}\left[\frac{C_1 + \mu C_2}{1 - \mu^2}\right] \qquad \text{Eq. 4.4}$$



(a) Day (slab surface temp > bottom temp)



(b) Night (slab bottom temp > surface temp)



(c) Constrained Transverse Joints (High slab mean temp., slab surface temp > bottom temp.)

Figure 4.22. Illustration of Warping Stress (Day and Night)

where σ_t = slab interior warping stress (psi),

E = modulus of elasticity of PCC (psi),

e = thermal coefficient of PCC

$\cong$ 0.000005/°F

μ = Poisson's ratio for PCC,

C_1 = coefficient in direction of calculated stress, and

C_2 = coefficient in direction perpendicular to C_1.

Bradbury also developed an approximate formula for slab corner warping stresses

$$\sigma_t = \frac{(E)(e)(\Delta T)}{3(1 - \mu)} \left[\sqrt{\frac{a}{\ell}} \right] \qquad \text{Eq. 4.5}$$

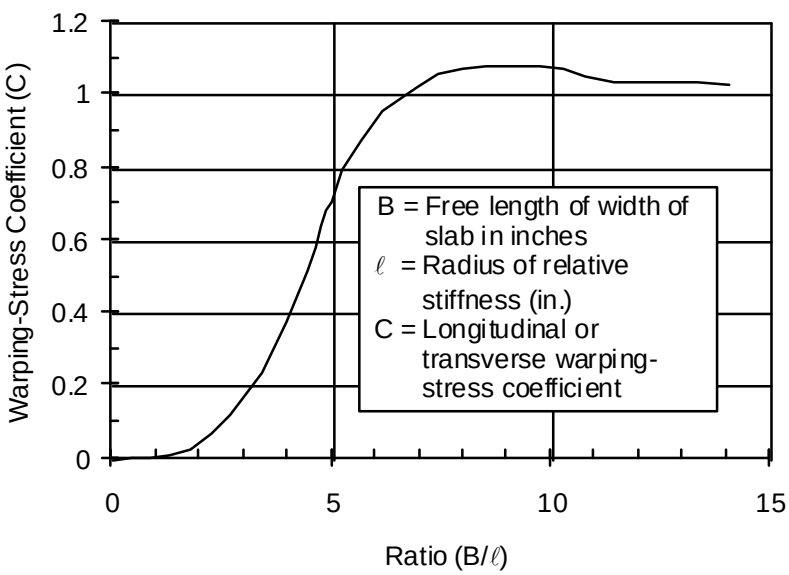


Figure 4.23. Curve Showing Variation in the Differential Temperature Stress Coefficient C for Different Values of the Ratio B/ℓ (redrawn from Bradbury [4.20])

where σ_t = approximate maximum corner warping stress (psi),

E = modulus of elasticity of PCC (psi),

e = thermal coefficient of PCC,

ΔT = temperature differential between the top and bottom of the slab (°F),

μ = Poisson's ratio for PCC,

a = radius of wheel load distribution for corner loading, and

ℓ = radius of relative stiffness.

The radius of relative stiffness (the relative stiffness of the slab and subgrade) is required for the above formulae. This equation is (from Westergaard [4.21]):

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)k}} \qquad \text{Eq. 4.6}$$

where ℓ = radius of relative stiffness (in.),
E = modulus of elasticity of PCC (psi),
h = slab thickness (in.),
k = modulus of subgrade reaction (pci), and
μ = Poisson's ratio for PCC.

6.2.2 EXAMPLE

(a) Calculate various warping stresses for

k = 200 pci
ΔT = 3° F/in. (maximum normally expected in Washington state)
e = 0.000005/°F
μ = 0.15
E = 5,000,000 psi

slab length = 9 ft
10 ft
13 ft
14 ft

slab thickness h = 9 in.

(b) First, calculate radius of relative stiffness.

$$\ell = \left[\frac{(5{,}000{,}000)(9)^3}{12(1 - 0.15^2)(200)}\right]^{0.25}$$

$$= 35.3 \text{ in.}$$

(c) Calculate B/ℓ values for determining C values (Figure 4.23).

slab = 9 ft (108 in.); B/ℓ = 108/35.3 = 3.1
slab = 10 ft (120 in.); B/ℓ = 120/35.3 = 3.4
slab = 13 ft (156 in.); B/ℓ = 156/35.3 = 4.4
slab = 14 ft (168 in.); B/ℓ = 168/35.3 = 4.8

(d) Calculate maximum edge warping stress. Use Equation 4.3.

(i) slab = 9 ft, C = 0.2

$$\sigma_t = \frac{(0.2)(5{,}000{,}000)(0.000005)(27)}{2}$$

$$= 68 \text{ psi}$$

(ii) slab = 10 ft, C = 0.25

$$\sigma_t = \frac{(0.25)(5{,}000{,}000)(0.000005)(27)}{2}$$

$$= 84 \text{ psi}$$

(iii) slab = 13 ft, C = 0.55

$$\sigma_t = \frac{(0.55)(5{,}000{,}000)(0.000005)(27)}{2}$$

$$= 186 \text{ psi}$$

(iv) slab = 14 ft, C = 0.68

$$\sigma_t = \frac{(0.68)(5{,}000{,}000)(0.000005)(27)}{2}$$

$$= 230 \text{ psi}$$

(e) Calculate interior warping stresses. Use Equation 4.4.

Longitudinally for 14 ft long slab

$B/\ell = 4.8$, $C_1 = 0.68$

Transversally for 12 ft wide slab

$B/\ell = 12(12)/35.3 = 4.1$, $C_2 = 0.45$

Thus,

$$\sigma_t = \frac{(5{,}000{,}000)(0.000005)(27)}{2}\left[\frac{0.68 + (0.15)(0.45)}{1 - (0.15)^2}\right]$$

$$= 258 \text{ psi}$$

(f) Calculate slab corner warping stress (nighttime condition) for maximum wheel load condition. Use 1° F/in. and load radius = 5.9 in. (same as FWD).

$$\sigma_t = \frac{(5{,}000{,}000)(0.000005)(9)}{3(1 - 0.15)}\left[\sqrt{\frac{5.9}{35.3}}\right]$$

$$= 36 \text{ psi}$$

6.2.3 SUMMARY — WARPING STRESSES

The preceding illustrates the relative magnitude of temperature induced warping stresses. Subsequent work by Darter [4.22] and illustrated for Washington state conditions by Hallin et al. [4.23] show that Bradbury's equations are a bit conservative (calculated stresses are higher than actual); however, the relative impact of warping stresses are well-illustrated using the preceding equations.

6.3 LOAD STRESSES

6.3.1 INTRODUCTION

The original equations developed by Westergaard [4.21] for the three primary, critical load locations will be presented. The critical load locations are (after Bradbury [4.20] and Westergaard [4.21]):

- Interior loading — Occurs when a load is applied on the interior of a slab surface which is "remote" from all edges
- Edge loading — Occurs when a load is applied on a slab edge "remote" from a slab corner.
- Corner loading — Occurs when the center of a load is located on the bisector of the corner angle.

6.3.2 WESTERGAARD'S ORIGINAL LOAD STRESS EQUATIONS

Westergaard's original equations are (assuming $\mu = 0.15$):

- Interior loading (tensile stress <u>bottom</u> of slab)

$$\sigma_i = \frac{0.3162(W)}{h^2}\left[4\log_{10}\left(\frac{\ell}{b}\right) + 1.069\right] \qquad \text{Eq. 4.7}$$

- Edge loading (tensile stress <u>bottom</u> of slab)

$$\sigma_e = \frac{0.572(W)}{h^2}\left[4\log_{10}\left(\frac{\ell}{b}\right) + 0.359\right] \qquad \text{Eq. 4.8}$$

- Corner loading (tensile stress <u>top</u> of slab)

$$\sigma_c = \frac{3(W)}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{\ell}\right)^{0.6}\right] \qquad \text{Eq. 4.9}$$

where $\sigma_i, \sigma_e, \sigma_c$ = maximum stress (psi) for in interior, edge and corner loadings, respectively,

W = wheel load (lb),
h = slab thickness (in.),
a = radius of wheel contact area (in.),
ℓ = radius of relative stiffness (in.) — refer to Equation 4.6, and
b = radius of resisting section (in.) — refer to Equation 4.10.

The radius of the resisting section can be estimated from

$$b = \sqrt{1.6(a^2) + h^2} - 0.675(h) \qquad \text{Eq. 4.10}$$

where b = equivalent radius of resisting section (in.),
a = radius of wheel contact area (in.), and
h = slab thickness (in.).

Specific note is made of the "h^2" term in all three equations for stress (Eq. 4.7, 4.8, 4.9). This suggests that slab thickness is very critical in reducing load stresses to "acceptable" levels.

6.3.3 EXAMPLE 1

Calculate the three critical stresses for 9 in. thick slabs. Use a 9,000 lb FWD loading (a = 5.9 in.). Use k = 200 pci for calculation of the radius of relative stiffness (l).

First, calculate b:

$$b = \sqrt{1.6(5.9^2) + (9)^2} - 0.675(9)$$

$$= 5.6 \text{ in.}$$

Second, calculate ℓ:

$$\ell = \sqrt[4]{\frac{(5{,}000{,}000)(9^3)}{12\,(1 - 0.15^2)\,(200)}} = 35.3 \text{ in.}$$

Third, calculate stresses:

- Interior

$$\sigma_i = \frac{0.3162\,(9000)}{9^2}\left[\,4\log_{10}\left(\frac{35.3}{5.6}\right) + 1.069\,\right]$$

$$= 150 \text{ psi}$$

- Edge

$$\sigma_e = \frac{0.572\,(9000)}{9^2}\left[\,4\log_{10}\left(\frac{35.3}{5.6}\right) + 0.359\,\right]$$

$$= 226 \text{ psi}$$

- Corner

$$\sigma_c = \frac{3\,(9000)}{9^2}\left[\,1 - \left(\frac{5.9\sqrt{2}}{35.3}\right)^{0.6}\,\right]$$

$$= 193 \text{ psi}$$

This illustrates why the middle of the slab at the longitudinal outer edge (next to shoulder) is considered critical. Further, it illustrates why tied PCC shoulders can reduce critical edge and corner stresses.

6.3.4 EXAMPLE 2

How would the stresses in Example 1 (Paragraph 6.3.3, above) be reduced if the slab was 12 in. thick?

First, calculate ℓ (from Eq. 4.6):

$$\ell = \sqrt[4]{\frac{(5{,}000{,}000)(12^3)}{12\,(1 - 0.15^2)\,200}}$$

$$= 43.8 \text{ in.}$$

Second, calculate b (from Eq. 4.10):

$$b = \sqrt{1.6(5.9^2) + 12^2} - 0.675(12)$$

$$= 6.0 \text{ in.}$$

Third, calculate critical stresses (from Equations 4.7, 4.8, and 4.9):

$$\sigma_i = \frac{0.3162\,(9000)}{12^2}\left[\,4 \log_{10}\left(\frac{43.8}{6.0}\right) + 1.069\,\right]$$

$$= 89 \text{ psi}$$

$$\sigma_e = \frac{0.572\,(9000)}{12^2}\left[\,4 \log_{10}\left(\frac{43.8}{6.0}\right) + 0.359\,\right]$$

$$= 136 \text{ psi}$$

$$\sigma_c = \frac{3\,(9000)}{12^2}\left[\,1 - \left(\frac{5.9\sqrt{2}}{43.8}\right)^{0.6}\right]$$

$$= 118 \text{ psi}$$

6.3.5 EXAMPLE 3

Using the three critical slab locations, what is the total stress (load plus warping) for the 9 in. slab?

- Interior (daytime temperature gradient)

  Load = 150 psi
  Warping = 258 psi (14 ft long slab)
  Total = 408 psi

- Edge (daytime temperature gradient)

  Load = 226 psi
  Warping = 230 psi (14 ft long slab)
  Total = 456 psi

- Corner (nighttime temperature gradient)

  Load = 193 psi
  Warping = 36 psi (14 ft long slab)
  Total = 229 psi

6.4 SLAB EXPANSION AND CONTRACTION

6.4.1 JOINT MOVEMENT

The following formula can be used to estimate joint movement in PCC slabs (Yoder and Witczak [4.24]):

$$z = (L)\,(12)\,[\,(e)\,(\Delta t) + \delta] \qquad \text{Eq. 4.11}$$

where z = joint opening (total amount a joint will open) (in.),
e = coefficient of thermal volume change,
δ = coefficient of shrinkage,
Δt = total temperature drop (°F), and
L = slab length (ft)

An updated version of the formula is (after FHWA [4.28]):

$$z = (C)(L)\,[\,(e)\,(\Delta t) + \delta] \qquad \text{Eq. 4.12}$$

where z = joint opening (or change in slab length) (in.),

C = base/slab frictional restraint factor,
= 0.65 for stabilized bases,
= 0.80 for granular bases,

L = slab length (in.),

e = PCC coefficient of thermal expansion (listed by coarse aggregate type)

= 6.6×10^{-6}/°F (quartz)
= 6.5×10^{-6}/°F (sandstone)
= 6.0×10^{-6}/°F (gravel)
= 5.3×10^{-6}/°F (granite)
= 4.8×10^{-6}/°F (basalt)
= 3.8×10^{-6}/°F (limestone)

Δt = the maximum temperature range (for some cases the temperature of the concrete at the time of placement minus the average daily minimum temperature in January) (°F),

δ = shrinkage coefficient of concrete
$\simeq$ 0.0008 in./in. for indirect tensile strength of 300 psi or less
$\simeq$ 0.00045 in./in. for indirect tensile strength of 500 psi
$\simeq$ 0.0002 in./in. for indirect tensile strength of 700 psi or greater

(Note: δ should be omitted for rehabilitation projects as shrinkage (assuming no new slab PCC) is not a factor.)

6.4.2 EXAMPLE

Calculate the total joint movement for existing PCC slabs 9, 10, 13, 14 ft long. Use $\Delta t = 67°$ F (eastern Washington) and $\Delta t = 45°$ F (western Washington), a granular base course and $e \simeq 5.0 \times 10^{-6}$/°F. ($\Delta t$'s were calculated from Table 4.6 by use of maximum and minimum seasonal slab temperatures (average of top and bottom temperatures) for July and January.) Using Eq. 4.12:

9 ft slab: $z = (0.80)(9)(12)[(0.000005)(67)]$
$= 0.029$ in.
$z = (0.80)(9)(12)[(0.000005)(45)]$
$= 0.019$ in.

10 ft slab: $z = (0.80)(10)(12)[(0.000335)]$
$= 0.032$ in.
$z = (0.80)(10)(12)[(0.000225)]$
$= 0.022$ in.

13 ft slab: $z = (0.80)(13)(12)[(0.000335)]$
$= 0.042$ in.
$z = (0.80)(13)(12)[(0.000225)]$
$= 0.028$ in.

14 ft slab: $z = (0.80)(14)(12)[(0.000335)]$
$= 0.045$ in.
$z = (0.80)(14)(12)[(0.000225)]$
$= 0.030$ in.

6.5 DISCUSSION

Joint performance without dowel bars has generally been judged to be adequate if the joint opening is 0.04 in. or less [4.24]. More recent information compiled by the FHWA [4.29] noted that aggregate interlock is "ineffective at crack widths greater than 0.035 inch." Further, the FHWA added "a smaller crack width, generally 0.025 inch, is considered necessary for satisfactory long-term performance of undoweled pavements." The random joint spacing used by WSDOT of 9, 10, 13, and 14 ft is therefore marginal for eastern Washington and substantially better for western Washington (assuming that the Δt's are correct — based on NOAA data). However, it must be noted that WSDOT has the shortest transverse joint spacings (11.5 ft) of all SHAs. Further, most of the faulting WSDOT PCC pavement has a 15 ft transverse joint spacing and, for the most part, these pavements have exceeded their original design lives. (The calculated z for a 15 ft slab is 0.048 in. for eastern Washington and 0.032 in. for western Washington.)

Up until the 1950s, it was common practice in the U.S. to use plain, jointed slabs with both contraction and expansion joints (Sutherland [4.25]). To examine the issue of joint performance, the states of Kentucky, Michigan, Minnesota, Oregon and Missouri built various PCCP test pavements in 1940-1941. The contraction joint spacing varied from 10 ft to 30 ft (3.0 m to 9.1 m) and expansion joint spacing from 120 ft (37 m) up to 1 mi (1.6 km). The basic findings relative to performance after 10 years of service follow.

- The expansion joints progressively closed with most of the change occurring within the first year after construction. For expansion joints spaced 120 ft (37 m) apart in Michigan with an original as-constructed width of 1.0 in. (25 mm), the joint had "closed" 0.8 in. (20 mm) after 10 years of service (i.e., only 20 percent of the original joint width remained.

- The contraction joints progressively "opened" as the expansion joints "closed." In general, the shorter the expansion joint spacing, the more the contraction joints opened.

A problem with progressive or even large seasonal contraction joint openings is the loss of joint load transfer — particularly so for joints without dowel bars. In general, the data summarized by Sutherland [4.25] showed that contraction joints of the same spacing with and without dowels exhibited slightly different amounts of faulting after 10 years of traffic (doweled joints had a bit less measured faulting). On the other hand, doweled and undoweled expansion joints had significantly different amounts of faulting.

Studies such as reported in Reference 4.25 (published in 1956) helped to eliminate the regular use of expansion joints in PCCP.

As a comparison to the example joint openings calculated in Paragraph 6.4.2, the data reported by Sutherland [4.25] was used as a comparison. For example, for the largest expansion joint spacings reported:

State	Contraction Joint Spacing, ft (m)	Expansion Joint Spacing, ft (m)	Measured Seasonal Joint Opening, in. (mm)
• Oregon	15 (4.6)	5280 (1609)	0.034 (0.86)
• Michigan	10 (3.0)	2700 (823)	0.024 (0.61)
• California	15 (4.6)	5280 (1609)	0.025 (0.64)
• Minnesota	15 (4.6)	5260 (1603)	0.043 (1.09)

Thus the measured joint openings were a bit less than those calculated for WSDOT conditions (the measured values from the four states were obtained by winter and summer joint measurements—which might have missed the "peak" conditions).

Study conducted by the PCA and reported by Colley and Humphrey [4.26] examined the issue of aggregate interlock in PCCP joints. The study was conducted in a controlled laboratory environment on full-scale slabs. The results included the following:

- Joint effectiveness was defined as

$$E = \frac{2D_0'}{D_0 + D_0'}(100)$$

where E = joint effectiveness (%),
D_0 = deflection of the loaded slab, and
D_0' = deflection of the unloaded slab.

If the deflection of the loaded and unloaded slab were equal, then E = 100 % and E = 0 % if there is no load transfer.

- The joint effectiveness increased for

- higher k values (i.e., improved subbase/subgrade),
- thicker slabs,
- smaller joint openings,
- smaller numbers of load repetitions,
- lower load levels, and
- "improved" PCC aggregate type (crushed gravel best in the PCA experiment).
- For a 9 in. (230 mm) thick PCC slab

Condition	E (%)
1: 0.035 in. (0.89 mm) opening at 1,000,000 loads cycles on cement treated base	78
2: Same as 1 but joint opening = 0.065 in. (1.65 mm)	58
3: Same as 1 but joint opening = 0.085 in. (2.16 mm)	48
4: Same as 1 but slab on clay subgrade	37
5: Same as 1 but slab on gravel subgrade	50
6: Joint opening = 0.065 in. (1.65 mm) at 1,000,000 load cycles on gravel subbase	20

Work reported by Darter in 1977 [4.31] examined the performance of PCC pavements located in several states (with special emphasis on the in-service PCC sections in Illinois on I-80 which were built for the AASHO Road Test). Using such data, a somewhat crude comparison of faulted transverse joints can be made for doweled and undoweled PCC pavements. Most of the joints were spaced 15 ft apart but this ranged up to 25 ft.

Condition	Slab Thickness (in.)	Average Faulting (in.)	Average ESALs
Undoweled	8.0	0.12	18,200,000
	9.0	0.04	13,600,000
	10.0	0.21	27,300,000

Doweled	8.0	0.25	11,200,000
	9.5	0.07	14,600,000
	11.0	0.04	17,100,000
	12.5	0.03	19,500,000

Such data suggest that slab thickness is important for reduced faulting levels only if the slabs contain dowel bars; or, stated differently, faulting is difficult to overcome with increased slab thickness if dowels are not used.

More recent data (1990) obtained for the FHWA [4.32] further reinforces the view that dowel bars enhance pavement performance. This work will be further summarized in SECTION 6.0.

The FHWA in their Technical Advisory 5040.30 [4.23] recommends that the L/ℓ ratio not exceed 5.0 when determining the maximum slab length. L is the slab length and ℓ is the radius of relative stiffness. For a range of "normal" slab thicknesses and moduli of subgrade reaction, the following slab lengths result using $L/\ell = 5.0$:

	k = 100 pci		k = 200 pci		k = 800 pci	
Slab Thickness (in.)	ℓ (in.)	L (ft)	ℓ (in.)	L (ft)	ℓ (in.)	L (ft)
9	42.0	17.5	35.3	14.7	25.0	10.4
13	55.3	23.0	46.5	19.4	32.9	13.7

Thus, "thinner" PCC slabs (say 9 in.) should have contraction joint spacings of no more than about 12 ft and "thicker" PCC slabs (say 13 in.) no more than about 15 ft given the tendency toward stiffer (high k) subbases.

A survey done by the AASHTO Subcommittee on Construction and summarized by the FHWA in 1987 [4.27] provides an updated view on joint practice in the U.S.

This 1987 survey noted that a few states still use (or have) expansion joints such as:

- Tennessee: 2640 ft (805 m) spacing
- New Jersey: 78 ft (24 m) spacing
- Michigan: during winter season paving
- Kansas: pressure relief (33 ft (10 m) spacing)

Other statistics compiled from the survey summary include

- 21 states provide for the use of plain jointed PCC (no dowels) with an average joint spacing of 15.0 ft (4.6 m) and a standard deviation of 2.0 ft (0.6 m).
- 23 states provide for the use of plain jointed PCC (with dowels) with an average joint spacing of 17.9 ft (5.5 m) and a standard deviation of 3.7 ft (1.1 m).
- States using a random joint spacing:
  - 12 states (plain jointed without dowels), and
  - 8 states (plain jointed with dowels).
- 22 states skew their contraction joints 2 ft in 12 ft (0.6 m in 3.7 m).

The majority of states which use dowel bars do the following:

- Dowel diameter: 1 1/4 in. (32 mm)
- Dowel length: 18 in. (460 mm)
- Dowel spacing: 12 in. (305 mm)
- Method of installation: baskets
- Dowel coating: epoxy; generally about 10 mils thick
- Alignment tolerances:
  - Vertical (in./ft): 1/8
  - Horizontal (in./ft): 1/8 - 1/4
  - Depth (in.): 1/8 - 1/4

The method of forming the contraction joint was predominantly by sawing a depth of either d/3 or d/4 (more used d/4 where d = slab depth).

The WSDOT Standard Specifications requires dowel bars used in PCCP to conform to AASHTO M183. Tie bars shall conform to AASHTO M31 (Standard Specification for Deformed Billet-Steel Bars for Concrete Reinforcement), Grade 60. Typically, WSDOT has used 1 1/4 in. (32 mm) diameter dowel bars 18 in. (457 mm) long and spaced 12 in. (305 mm) apart. The 1 1/4 in. (32 mm) diameter conforms to the general FHWA and AASHTO recommendations [4.28, 4.30] that the minimum dowel bar diameter be equal to D/8 (D = thickness of slab). Thus, the 1 1/4 in. (32 mm) diameter is appropriate for up to a 10 in. (0.83 ft or 250 mm) thick slab (i.e., 10/8 = 1.25).

Finally, from a Chilean study about plain, jointed, undoweled PCC pavements (joint spacings ranged from 11.0 ft (3.4 m) to 17.6 ft (5.4 m) and slab thicknesses from 8.2 to 10.1 in. (210 to 260 mm)) reported by Poblette et al. [4.33], the three conditions of slab warping were measured (Figure 4.22). The upward concave condition (Figure 4.22a) was due to a negative temperature gradient in the slab (PCC slab surface temperature less than

the bottom). This occurred typically from the late afternoon to the late morning hours. The slabs were concave downward (Figure 4.22b) when the opposite occurred. Further, a third condition occurred when the average slab temperature was high (summer) whereby the slab transverse joints were constrained during the afternoon hours (Figure 4.22c). The positive temperature gradient along with the high average temperature prevented the rotation of slab transverse joints due to "locked joints" (analogous to "prestressing" the slabs). A previous WSDOT study [4.34] postulated that this condition (Figure 4.22c) has occurred on WSDOT PCC pavements and may have contributed to longitudinal cracking.

Interestingly, the Chilean study [4.35, 4.36] found that:

- Due to thermal gradients, the top of the slab is cooler than the bottom (hence condition Figure 4.22a) about two-thirds of the time (14 hours a day during the summer (about 60 percent) and all day (24 hours) during the winter.

- As a result of this warping, corner breaks and transverse cracks have occurred with excessive frequency (slabs are on CTB). The transverse cracks start at the longitudinal edge and at the top of the PCC slab then progress downward. Similar distress was observed on I-5 at the Tracy, California, test site.

- Maximum seasonal joint openings ranged from about 0.024 to 0.055 in. (0.6 to 1.4 mm) and were a function of latitude (temperature actually). The Chilean test sites ranged from latitude 32°S (lowest mean winter temperature and largest joint openings) to 40°S (highest mean winter temperature and smallest joint openings). (Note: latitude-wise, on the west coast of the U.S. in the northern hemisphere, the Chilean latitudes are analogous to locations such as San Diego, California, at 33°N and Redding, California at 40°N. Washington state ranges from latitude 49°N to about 46°N.)

6.6 CLIMATE DATA

Most of the required climate data used in SECTION 4.0, Paragraph 6, was developed from NOAA data and is contained in the research report by Sharma et al. [4.23]. For convenience, it is repeated here.

Table 4.5 contains monthly 30-year average temperatures required for calculating temperature gradients in PCC slabs. The eastern Washington data were obtained from the Spokane International Airport and the western Washington data from the Olympia airport (solar radiation from Sea-Tac International Airport).

Table 4.6 contains the calculated slab temperatures (top and bottom) for four slab thicknesses (7, 9, 10, 12 in.) and eastern and western Washington. The reader should refer to Sharma et al. [4.23] for the details of how these temperatures were calculated.

Table 4.7 contains the maximum positive thermal gradients for the mean day in each month for both eastern and western Washington. These values are directly used in calculating warping stresses regardless of the specific method used (either Bradbury [4.20] or Darter [4.22]).

Table 4.5. Climatic Data Used to Calculate Temperature Gradients in PCC Pavements

Eastern Washington

Month	Average Air Temperature for Normal Days			Average Solar Radiation BTU/Sq. Ft.
	Daily Maximum °F	Daily Minimum °F	Monthly Average °F	
January	31.1	19.6	25.4	315.0
February	39.0	25.3	32.2	605.6
March	46.2	28.8	37.5	1040.6
April	57.0	35.2	46.1	1493.0
May	66.5	42.8	54.7	1917.9
June	73.6	49.4	61.5	2082.8
July	84.3	55.1	69.7	2357.5
August	81.9	54.0	68.0	1942.0
September	72.5	46.7	59.6	1435.3
October	58.1	37.5	47.8	840.9
November	41.8	29.2	35.5	397.7
December	33.9	24.0	29.0	255.2

Western Washington

Month	Average Air Temperature for Normal Days			Average Solar Radiation BTU/Sq. Ft.
	Daily Maximum °F	Daily Minimum °F	Monthly Average °F	
January	45.1	31.1	38.1	262.4
February	49.6	32.2	40.9	494.5
March	54.4	34.0	44.2	854.2
April	62.3	37.6	50.0	1295.3
May	68.6	41.6	55.1	1720.1
June	72.6	45.5	59.1	1797.4
July	79.7	48.0	63.9	1980.4
August	78.9	47.8	63.4	1607.3
September	72.6	44.4	58.5	1154.1
October	62.3	40.5	51.4	650.8
November	52.4	35.2	43.8	338.4
December	47.5	33.9	40.7	212.6

Table 4.6. Calculated Maximum Temperatures (°F) for the Mean Day of Each Month

	Eastern Washington					Western Washington				
		Bottom of Slab (Thickness)					Bottom of Slab (Thickness)			
Month	Surface	7"	9"	10"	12"	Surface	7"	9"	10"	12"
January	33	29	28	28	27	46	41	41	40	40
February	44	38	37	36	36	53	46	45	45	44
March	56	46	45	44	44	62	52	51	50	49
April	72	58	56	56	55	74	61	59	59	58
May	86	70	68	67	66	85	69	67	66	65
June	96	78	75	75	73	91	74	72	71	69
July	109	88	86	85	83	99	80	78	77	75
August	101	84	81	81	79	93	77	75	74	73
September	86	72	70	69	68	82	69	67	67	66
October	65	55	54	54	53	67	58	57	56	56
November	45	39	39	38	38	53	48	47	47	46
December	35	32	31	31	31	48	43	43	43	42

Table 4.7. Maximum Positive Thermal Gradients (°F/inch) for the Mean Day of Each Month in Washington State

	7" Pavement		9" Pavement		10" Pavement		12" Pavement	
Month	Eastside	Westside	Eastside	Westside	Eastside	Westside	Eastside	Westside
January	0.65	0.71	0.57	0.56	0.53	0.60	0.47	0.50
February	0.97	1.00	0.85	0.89	0.79	0.80	0.70	0.75
March	1.45	1.43	1.27	1.22	1.19	1.20	1.06	1.08
April	1.98	1.86	1.73	1.67	1.61	1.50	1.44	1.33
May	2.40	2.29	2.10	2.00	1.97	1.90	1.74	1.67
June	2.55	2.43	2.24	2.11	2.09	2.00	1.85	1.83
July	2.95	2.71	2.58	2.44	2.42	2.20	2.14	2.00
August	2.56	2.29	2.24	2.00	2.10	1.90	1.86	1.67
September	2.06	1.86	1.81	1.67	1.69	1.50	1.50	1.25
October	1.39	1.29	1.22	1.11	1.14	1.10	1.01	0.92
November	0.76	0.71	0.66	0.67	0.62	0.60	0.55	0.58
December	0.55	0.71	0.47	0.55	0.45	0.50	0.40	0.50

SECTION 4.0

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SECTION 5.0

THE AASHTO FLEXIBLE PAVEMENT DESIGN PROCEDURE (NEW OR RECONSTRUCTED)

This SECTION is used to describe the AASHTO flexible pavement design procedure (which has been adopted by WSDOT). Information is provided on how the procedure was developed, along with some insight into earlier versions (pre-1986). The required design input values are described and illustrated. The concept of reliability is introduced and is common to both flexible and rigid pavement design. Various design examples are shown. U.S. Customary Units are used throughout this SECTION since the AASHTO design procedure currently uses these units.

1. BACKGROUND

1.1 AASHO ROAD TEST

The information obtained from the AASHO Road Test in the early 1960s was crucial in advancing knowledge of pavement design, pavement performance, load equivalencies, climate effects, and much more. The basic performance information resulted in the performance equation which was used to generate the design nomograph shown in this SECTION. As such, some background information on the Road Test follows. It should be helpful in understanding both the strengths and weaknesses of that experiment. It also helps to put the LTPP experiments of the SHRP/FHWA program into perspective.

1.1.1 LOCATION — OTTAWA, ILLINOIS

The AASHO Road Test site (which eventually became part of I-80) at Ottawa, Illinois, was typical of northern climates. In comparison to the state of Washington:

		Washington	
	AASHO Site	Seattle	Spokane
• Average Mean Temperature (July)	76 °F	64 °F	70 °F
• Average Mean Temperature (January)	27 °F	38 °F	25 °F

		Washington	
	AASHO Site	Seattle	Spokane
• Annual Average Rainfall (in.)	34	36-39	15
• Average Depth of Frost (in.) (fine-grained soil)	28	6	27

1.1.2 TEST FACILITIES

(a) Six 2-lane test loops (Figure 5.1)
(b) No traffic — Loop 1 (environmental effects)
(c) Vehicles on Loops 2-6 are given in Figure 5.2

1.1.3 PERFORMANCE MEASUREMENTS

(a) Roughness and visual distress
(b) Deflections, strains, etc.
(c) Pavement Serviceability Index

1.2 FLEXIBLE PAVEMENT MATERIALS AND DESIGNS—AASHO ROAD TEST

1.2.1 MATERIALS [5.1, 5.12]

The primary materials used in the flexible pavement sections will be briefly described below. This kind of information is helpful in comparing WSDOT specified materials to those used at the Road Test.

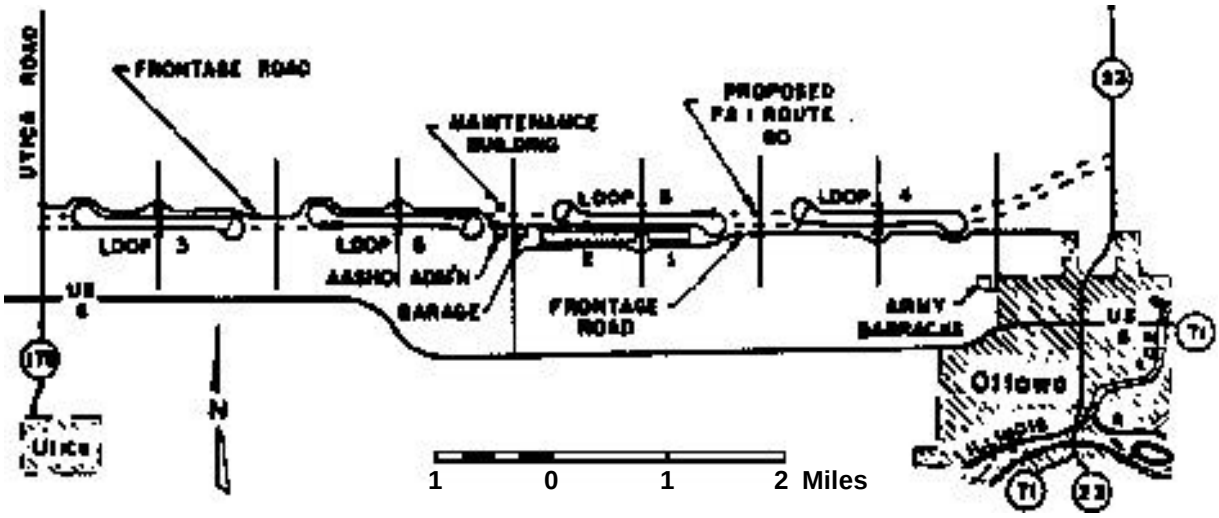
1.2.1.1 Asphalt Concrete

The asphalt concrete (AC) mixes used at the Road Test had:

- crushed limestone coarse aggregate,
- natural siliceous coarse sand,
- mineral filler which was limestone dust, and
- penetration grade asphalt cement (85-100 pen).

The gradation requirements for the surface and binder courses were

Sieve	Surface Course Gradation Limits	Binder Course Gradation Limits
1 in.		100
3/4 in.	100	88-100
1/2 in.	86-100	55-86
3/8 in.	70-90	45-72
No. 4	45-70	31-50
No. 10	30-52	19-35
No. 20	22-40	12-26
No. 40	16-30	7-20
No. 80	9-19	4-12
No. 200	3-7	0-6



Layout of AASHO Road Test

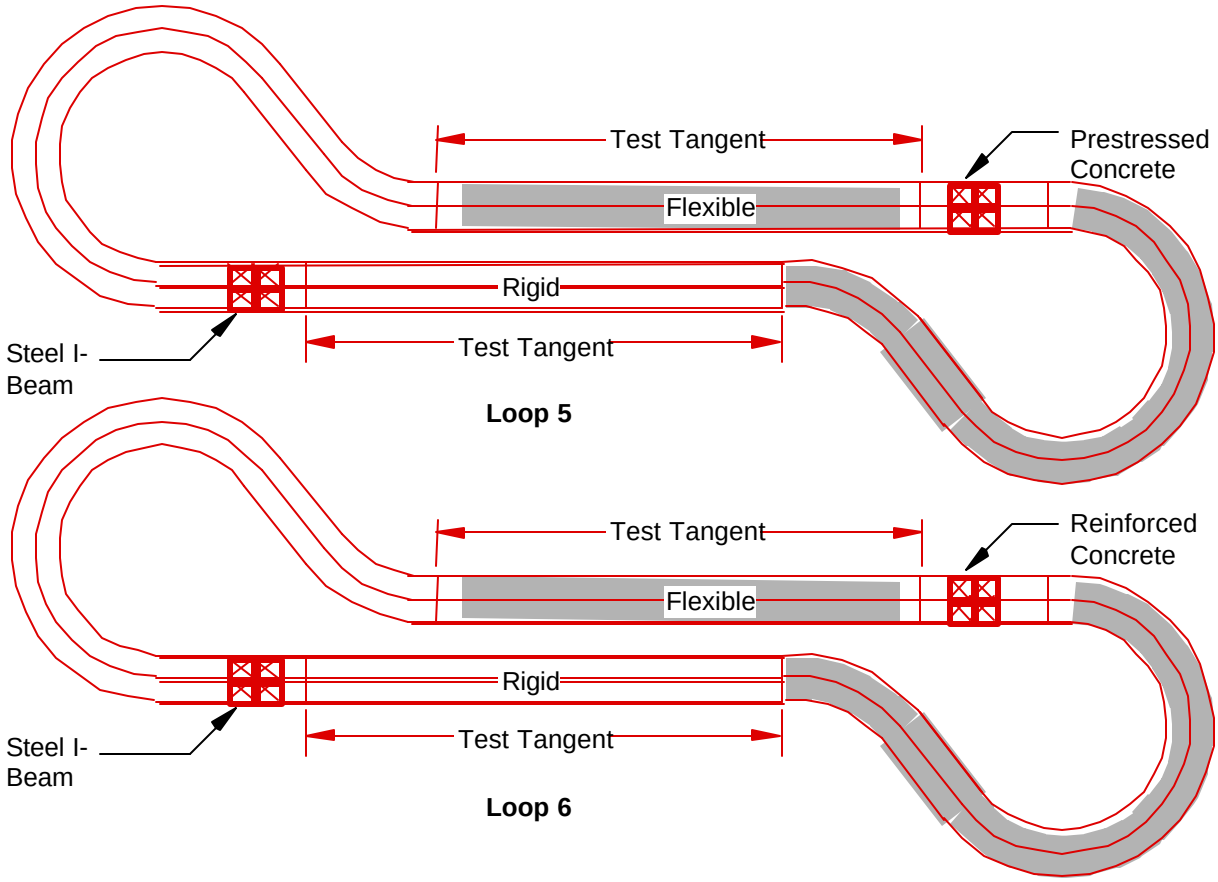


Figure 5.1. Layout of the AASHO Road Test [Redrawn from

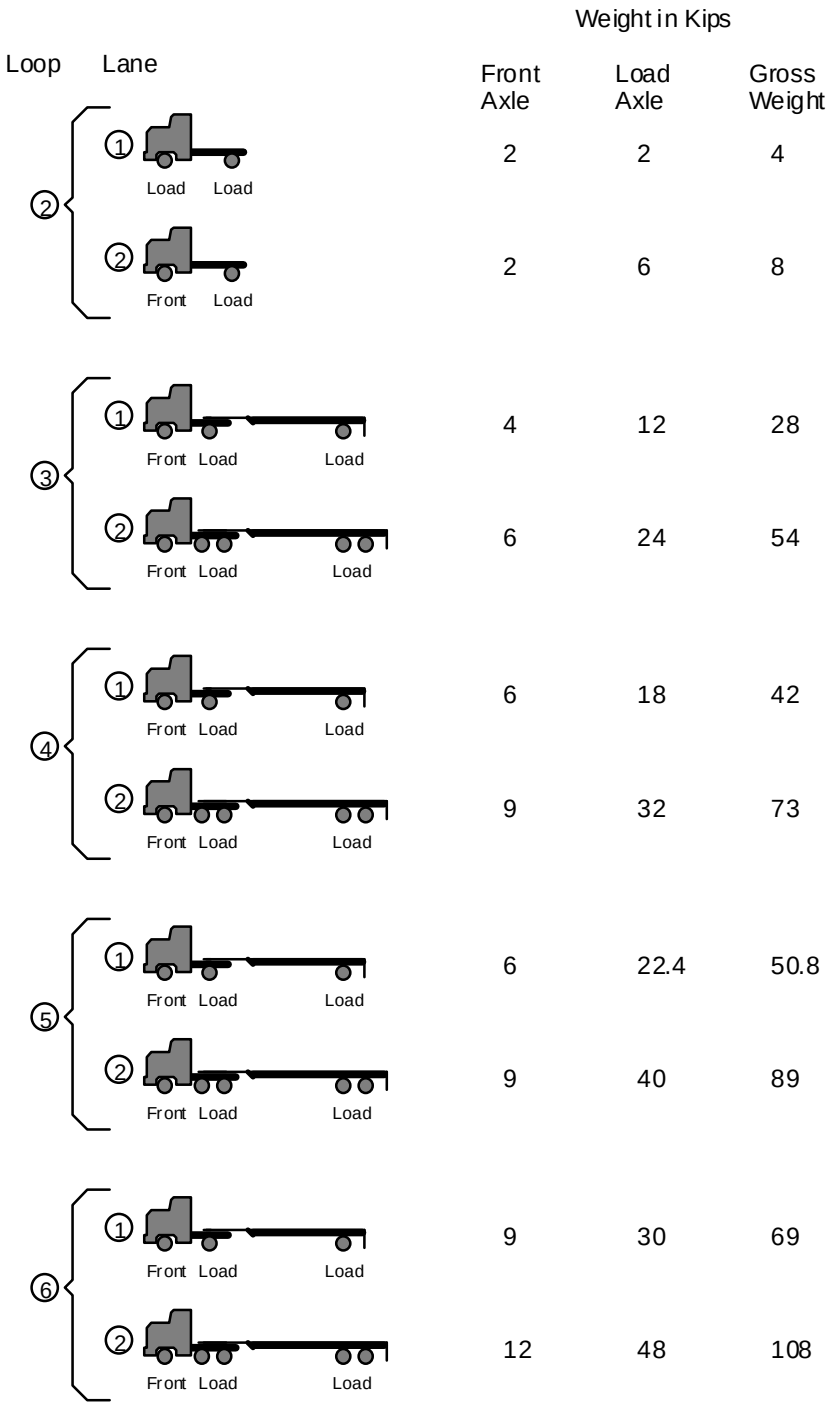


Figure 5.2. Axle Weights and Distributions Used on Various Loops of the AASHO Road Test [Redrawn from 5.1]

The AASHO binder course gradation is similar to WSDOT Class E (see SECTION 1.0, Table 1.3) and the surface course gradation is similar to WSDOT Classes A and B. For the AC surfaced test sections (in Loops 3-6), the surface course was 1.5 in. thick with the remainder of the AC thickness binder course.

The asphalt concrete mixes were designed by the Marshall method (50 blows per face). The typical field asphalt contents were about 5.4 and 4.4 percent by weight of total mix for the surface and binder courses, respectively. The field air voids averaged 7.7 percent for the six test loops.

1.2.1.2 Base Course

The base course material was a crushed dolomitic limestone which conformed to the following gradation range.

Sieve	Specification Range Percent Passing	Actual Mean Percent Passing
1-1/2 in.	100	100
1 in.	80-100	90
3/4 in.	70-90	81
1/2 in.	60-80	68
No. 4	40-60	48
No. 10	28-46	35
No. 40	16-33	20
No. 100	7-20	13.5
No. 200	3-12	10

This gradation is not similar to any currently used WSDOT base course material. The AASHO gradation is a bit like CSBS for 1/4 in. and above and bit like CSTC for particle sizes less than 1/4 in. It is significant to note that 10 percent passed the No. 200 sieve. Thus, the base was likely a bit frost susceptible.

The CBR of the base material averaged 107.7 percent based on laboratory tests; however, the test results ranged from a low of 52 percent to a high of 160 percent (total of 11 tests). The minimum specified CBR was 75 percent.

Typical inplace mean dry densities were about 140 to 142 lb/ft^3, with mean moisture contents ranging from 5.6 to 6.1 percent.

1.2.1.3 Subbase Course

The subbase material was a sand-gravel mixture which conformed to the following gradation range.

Sieve	Specification Range Percent Passing	Actual Mean Percent Passing
1-1/2 in.	100	100
1 in.	95-100	100
3/4 in.	90-100	96
1/2 in.	80-100	90
No. 4	55-100	71
No. 10	40-80	52
No. 40	10-30	25
No. 200	5-9	6.5

The CBR was specified not to exceed 60 percent. Typical laboratory CBR values ranged from 28 to 51 percent. The mean percent passing the No. 200 sieve was 6.5 percent (a lesser percentage than the base course).

Typical inplace mean dry densities ranged from 139 to 141 lb/ft^3, with moisture contents ranging from 6.1 to 6.8 percent.

1.2.1.4 Embankment Soil (Subgrade)

One Road Test requirement was that all test pavements be built on a uniform embankment with the top 3 ft. constructed of an A-6 soil. Tests on this soil revealed:

- LL = 31%
- PI = 16%
- P_{200} = 82%
- AASHTO T99
  γ_{dmax} = 119 pcf
  optimum w.c. = 13%

Field CBR and density tests on the completed embankment just prior to paving were [5.12]:

- Flexible Pavement Tangents (all Loops)
  - Average CBR = 2.9
  - CBR Range = 1.9 - 3.5
  - Average % Saturation = 85%
  - Average % Compaction = 98.5% (AASHO T99)
  - Average % w.c. over optimum = +0.8%

- Rigid Pavement Tangents (all Loops)
    - Average CBR = 1.7
    - CBR Range = 1.0 - 2.5
    - Average % Saturation = 87%
    - Average % Compaction = 97.8% (AASHTO T99)
    - Average % w.c. over optimum = +1.3%

1.2.2 AASHO FLEXIBLE PAVEMENT TEST SECTIONS

1.2.2.1 Experimental Designs

(a) Overall, there were 288 flexible pavement test sections in the main experiments (332 counting replications)

(b) Each flexible pavement section in the main experiment was 100 ft long with all sections being separated by a transition pavement of at least 15 ft.

(c) The structural thicknesses for each loop are summarized below.

Loop No.	AC Thickness (in.)	Base Thickness (in.)	Subbase Thickness (in.)
1	1.0 3.0 5.0	0.0 6.0	0.0 8.0 16.0
2	1.0 2.0 3.0	0.0 3.0 6.0	0.0 4.0
3	2.0 3.0 4.0	0.0 3.0 6.0	0.0 4.0 8.0
4	3.0 4.0 5.0	0.0 3.0 6.0	4.0 8.0 12.0
5	3.0 4.0 5.0	3.0 6.0 9.0	4.0 8.0 12.0
6	4.0 5.0 6.0	3.0 6.0 9.0	8.0 12.0 16.0

The AASHO Road Test flexible pavement experiment was designed as a full factorial experiment with the design factors being

- surfacing thickness,
- base thickness, and
- subbase thickness.

1.2.2.2 Overview of Flexible Pavement Performance

Of the various axle loads applied to each of the test loops, a total of 1,114,000 load applications were made (not ESALs). Of the 332 test sections, 264 sections (or 80 percent) had reached a PSI of 1.5 or less on or before the end of the load application period (from November 1958 to June 1960).

Tables 5.1 and 5.2 are used to overview the actual performance for Loops 4 and 6 (Loop 4 had an 18,000 lb single axle as one of its standard loads and Loop 6 had the heaviest axle loads used at the Road Test). These two loops are likely of the greatest contemporary interest to the reader. Table 5.1 shows that increasing AC thickness (from 3.0 to 5.0 in.) results in increased performance (loads to failure); however, not at as significant a rate as one might expect. Similar results are observed for Loop 6. Each section, if it reached a PSI of 1.5 prior to completion of all load applications (recall the total was 1,114,000 axle repetitions) was overlaid with new AC so that the trucks applying the loads could safely continue. For Loop 4, all of these overlays were applied during the months of March, April, and May. For Loop 6, 75 percent of the overlays were applied during March, April, and May. This suggests that freeze-thaw effects strongly influenced flexible pavement performance at the Road Test.

The depth of freeze varied a bit from year-to-year but averaged about 28 in. for the flexible sections and generally reached 30 in.

Such performance might provide additional insight into the required depth of pavement to resist freeze-thaw effects. The following summarizes information from Tables 5.1 and 5.2:

Loop 4: 18,000 lb Single Axle

AC Thickness (in.)	Averaged Total Depth of Pavement (in.) and Percent of Frost Depth[1, 2]		
	"Poor" Performance <500,000 reps	"Average" Performance >500,000 reps	"Better" Performance >1,114,000 reps
3.0	12.4 (41%)	19.5 (65%)	21.0 (70%)
4.0	12.7 (42%)	19.7 (65%)	20.0 (67%)
5.0	11.3 (38%)	18.3 (61%)	21.0 (70%)

Table 5.1. AASHO Road Test Loop 4 Results[2, 3]—Flexible Test Sections [after Ref. 5.1]

AC Thickness (in.)	Axle	Base and Subbase Thicknesses (in.)								
		0.0			3.0			6.0		
		4.0	8.0	12.0	4.0	8.0	12.0	4.0	8.0	12.0
3.0	18,000 lb single	2,000	72,000	98,000	74,000	82,000	583,000	80,000	92,000	(1.6)
	32,000 lb tandem	12,000	74,000	110,000	76,000	86,000	601,000	80,000	570,000	618,000
4.0	18,000 lb single	78,000	107,000	426,000	87,000	106,000	1,110,000	90,000	(1.9)	(1.9)
	32,000 lb tandem	83,000	102,000	576,000	93,000	144,000	796,000	120,000	(2.0)	(3.1)
5.0	18,000 lb single	88,000	119,000	676,000	125,000	589,000	592,000	626,000	(3.6)	(3.3)
	32,000 lb tandem	102,000	126,000	850,000	151,000	752,000	(2.2)	678,000	(2.7)	(2.7)

Notes:
1. Replicate sections averaged
2. Values in parentheses are PSI at end of load applications
3. Values not in parentheses are the numbers of load applications to reach a PSI = 1.5

Table 5.2. AASHO Road Test Loop 6 Results[2, 3]—Flexible Test Sections [after Ref. 5.1]

AC Thickness (in.)	Axle	Base and Subbase Thicknesses (in.)								
		3.0			6.0			9.0		
		8.0	12.0	16.0	8.0	12.0	16.0	8.0	12.0	16.0
4.0	30,000 lb single	72,000	373,000	112,000	82,000	83,000	552,000	82,000	353,000	(2.0)
	48,000 lb tandem	80,000	573,000	362,000	373,000	100,000	621,000	82,000	353,000	(2.0)
5.0	30,000 lb single	78,000	101,000	573,000	100,000	522,000	(1.8)	595,000	719,000	(3.3)
	48,000 lb tandem	103,000	419,000	652,000	105,000	606,000	809,000	624,000	722,000	(3.5)
6.0	30,000 lb single	141,000	113,000	627,000	106,000	(1.6)	(3.2)	624,000	(2.4)	(2.7)
	48,000 lb tandem	579,000	485,000	(2.4)	250,000	(3.0)	(3.9)	(2.2)	(2.6)	(3.6)

Loop 6: 30,000 lb Single Axle

AC Thickness (in.)	Averaged Total Depth of Pavement (in.) and Percent of Frost Depth[1, 2]		
	"Poor" Performance <500,000 reps	"Average" Performance >500,000 reps	"Better" Performance >1,114,000 reps
4.0	20.1 (67%)	27.5 (92%)	29.0 (97%)
5.0	18.3 (61%)	25.3 (84%)	28.5 (95%)
6.0	19.3 (64%)	26.3 (88%)	27.5 (92%)

Notes:

[1] Frost depth = 30 in.

[2] Load repetitions to a PSI of 1.5 (<500,000; >500,000) and load repetitions to greater than 1,114,000 (PSI > 1.5).

Based on the above three categories of load repetitions (less than 500,000; more than 500,000; more than 1,114,000), the total thickness of the base and subbase layers had a significant influence on the load repetitions. Given such information and the fact that most of the flexible sections which reached a PSI of 1.5 before completion of the Road Test traffic were overlaid immediately following the spring thaw (that is, the months of March, April, and May), then two preliminary observations are made:

- For low ESAL pavements (say 500,000 ESALs or less as illustrated by Loop 4), the total depth of the pavement should be about 65 percent of the expected depth of frost (WSDOT currently recommends a minimum of 50 percent of the maximum expected depth of frost; see SECTION 2.0, Paragraph 3.2.4).

- For pavements subjected to much higher axle loads ((say ESALs of about 5,000,000 or more) as illustrated by Loop 6 (recall the Load Equivalency Factor for a 30,000 lb single axle is about 8.0)), the total depth of the pavement should be about 90 to 100 percent of the expected depth of frost.

It is as yet uncertain as to whether such percentages as noted above apply to frost depths greater than say 30 in., since the evidence from the Road Test applies to a unique set of materials (base, subbase, and subgrade) and moisture conditions. These results are further supported by the COE pavement design process for frost areas as discussed in Paragraph 3.1.4.4, SECTION 2.0. For similar ESAL levels, the combined thickness of surface course and base course are comparable. Such information does suggest that the WSDOT criterion of a structural section exceeding 50 percent of the maximum expected depth of freeze is not overly conservative. (The approximate maximum depths of freeze were shown in Figures 2.34 and 2.35 for fine-grained and coarse-grained soils, respectively. Figure 2.36 showed measured depths of freeze during the severe winters of 1949-1950. These three figures are contained in SECTION 2.0 of this Guide.)

Frost depths similar to those observed at the Road Test can be expected over about 30 to 50 percent of the state of Washington every winter.

1.3 DEVELOPMENT OF AASHTO GUIDES

1.3.1 INTRODUCTION

The AASHTO Guides were based on the results of the AASHO Road Test, a $27 million investment (1960 dollars). Joint state funds amounted to about $12 million. Washington state provided $168,264 of that amount.

From the AASHO Road Test, equations were developed which related loss in serviceability, traffic, and pavement thickness. It is assumed that the equations developed:

(a) May be extended to other subgrade soils by an abstract soil support scale.

(b) Can be applied to mixed traffic by use of equivalent 18-kip single axle loads.

(c) May be applied to other regions through the use of a regional factor.

(d) May be applied to other surfaces, bases, and subbases by assigning appropriate layer coefficients.

(e) Accelerated testing (2-year period) can be extended to a 20-year design period.

1.3.2 AASHTO GUIDE VERSIONS

1959 — Guidelines

1962 — Interim Guide

1972 — Revision of Guide (Blue Manual) and NCHRP Report 128

1981 — Chapter III revisions (rigid pavement)

1986 — New Guide

1993 — New Part III, Chapter 5 (overlay design) and other minor revisions

1.3.3 DESIGN CRITERIA–DEFINITIONS

Present Serviceability Rating. A subjective rating of the present pavement condition by a panel of people on a scale of 0 to 5 (5 being an excellent pavement).

Present Serviceability Index. An index which gives present serviceability from physical measurements. Can be calculated as follows:

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

where: SV = slope variance (refer to SECTION 3.0 — CHLOE Profilometer)

$\overline{RD}$ = rut depth

C+P = area of class 2 & 3 cracking plus patching/1000 ft^2

The change in PSI is a function of maximum axle load, number of axles, number of load repetitions, seasonal effects, thickness of pavement components, and soil support.

1.3.4 LIMITATIONS OF AASHO ROAD TEST DATA

(a) Specific pavement materials and roadbed soil

(b) Single environment

(c) An accelerated two-year testing period extrapolated to 10-20 year (or more) designs

(d) Operating vehicles with identical axle loads and configurations, as opposed to mixed traffic

1.3.5 GENERAL MATERIAL CONSIDERATIONS

1.3.5.1 Roadbed Soils (Subgrade)

(a) Compaction of roadbed soils should include an appropriate density requirement.

(b) Expansive or resilient soils should receive special consideration.

(c) Pockets of frost-susceptible soils may necessitate removal and be replaced with nonsusceptible material.

(d) Highly organic soils may require special treatment such as preconsolidation.

(e) Unusually variable soils may require special treatment such as scarifying and recompacting, etc.

(f) Site conditions may require that special attention be given to design and construction of drainage systems. Drainage is always important.

(g) Certain roadbed soils cause construction problems such as wet clays or cohesionless soils which are displaced by construction equipment.

1.3.5.2 Base and Subbase Courses

(a) Untreated aggregate base and subbase courses should be compacted to a satisfactory density determined by standard tests methods, such as AASHTO T 99 or T-180.

(b) Cement-treated and lime-treated subbase and base courses should be compacted to a satisfactory density determined by the standard test method, AASHTO T-134.

(c) Asphalt treated subbase and base courses should be compacted to a satisfactory density based on the test method used to determine the stability of the mixture, i.e., Hveem Stabilometer, Marshall, etc.

(d) Control material quality by specification. Base course specifications which can be used, AASHTO M-147 and M-75. Many good specifications are available.

1.3.5.3 Surface Courses

Success of the surface course is primarily dependent upon obtaining a mixture with the optimum gradation of aggregate and percent of bituminous binder (asphalt). Laboratory mix design prior to construction is essential. Mixture design criteria set minimum (or maximum) values for items such as strength, deformation, air voids in the mix, etc.

1.3.6 PRE-1986 DESIGN PROCEDURE

The information which follows in this subparagraph provides a bit of background and insight into previous versions of the AASHTO Guide (pre-1986). If information on the current version of the AASHTO Guide (1993) is sought, skip to Paragraph 2.

1.3.6.1 Design Concept

$$SN = f(p_t, W_t, S, R) \qquad \text{Eq. 5.1}$$

where
- SN = structural number
- p_t = terminal serviceability index
- W_t = total 18,000 lb. ESAL's
- S = soil support (subgrade support)
- R = regional factor (climate adjustment)

then $SN = a_1D_1 + a_2D_2 + a_3D_3$

where $D_{1,2,3}$ = thickness (inches) of surface, base and subbase respectively, and

$a_{1,2,3}$ = layer coefficients

Equation 5.1 is solved by nomograph for desired level of p_t (refer to Figure 5.4)

Figure 5.3 illustrates a process which can be used to estimate individual layer thicknesses.

It may be helpful to note that the structural number (SN) was initially termed "thickness index." [5.1]. Further, if the layer coefficients (a_1, a_2, a_3) were all assigned a value of 1.0, then the SN would represent the total pavement structural thickness; however, in practice, this does not occur.

1.3.6.2 Traffic Analysis

(a) Select traffic analysis period (say 20 years)

(b) Determine, for each load group i, N_i = expected number of applications of axle loads in this group

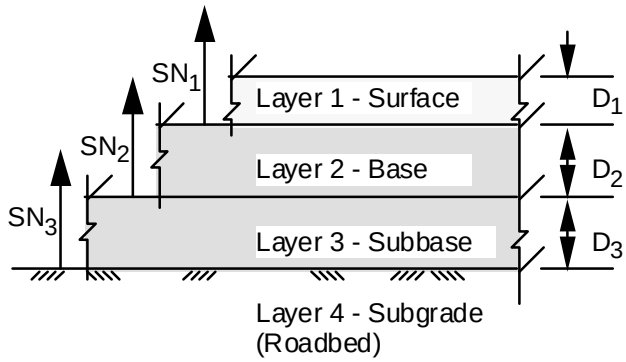
(c) Assume SN (usually 3.0 or 4.0) and determine load equivalency factors e_i (Table 5.3)

(d) Then

$$W_i = N_i e_i$$

$$W_t = \Sigma N_i e_i$$

(e) Recognize directional and/or lane distribution



$$D_1^* \geq \frac{SN_1}{a_1}$$

$$SN_1^* = a_1 D_1^* \geq SN_1$$

$$D_2^* \geq \frac{SN_2 - SN_1^*}{a_2}$$

$$SN_2^* = a_2 D_2^*$$

$$SN_1^* + SN_2^* \geq SN_2$$

$$D_3^* \geq \frac{SN_3 - (SN_1^* + SN_2^*)}{a_3}$$

* indicates value actually used which must be equal to or greater than the required value.

Figure 5.3. AASHTO Conceptual Flexible Pavement Layer Determination

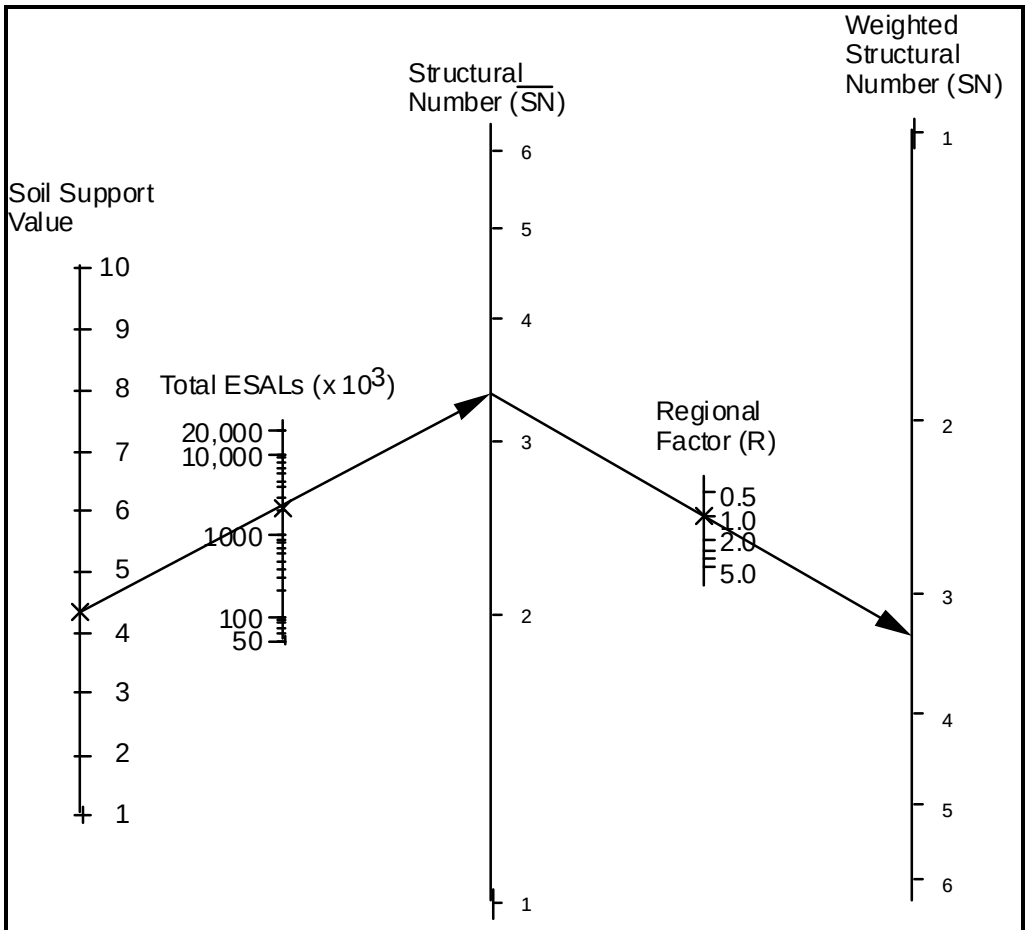


Figure 5.4. Sketch of Pre-1986 AASHTO Flexible Pavement Design Nomograph

Table 5.3. A Sample of AASHO Traffic Equivalence Factors for Flexible Pavement (from Ref. 5.3)

Axle Load		Structural Number					
kips	**kN**	**1**	**2**	**3**	**4**	**5**	**6**
Single Axles, p_t = 2.0							
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
10	44.5	0.08	0.08	0.09	0.08	0.08	0.08
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.59	1.56	1.55	1.57	1.60
30	133.4	10.38	10.03	9.24	8.65	8.73	9.17
40	177.9	39.57	38.02	34.34	30.92	30.04	31.25
Single Axles, p_t = 2.5							
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.57	1.49	1.47	1.51	1.55
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04
Tandem Axles, p_t = 2.0							
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
20	89.0	0.10	0.12	0.12	0.12	0.11	0.10
30	133.4	0.61	0.62	0.65	0.64	0.63	0.63
34	151.2	1.06	1.07	1.08	1.08	1.08	1.07
40	177.9	2.22	2.19	2.15	2.13	2.16	2.18
48	213.5	5.10	4.98	4.72	4.58	4.68	4.83
Tandem Axles, p_t = 2.5							
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

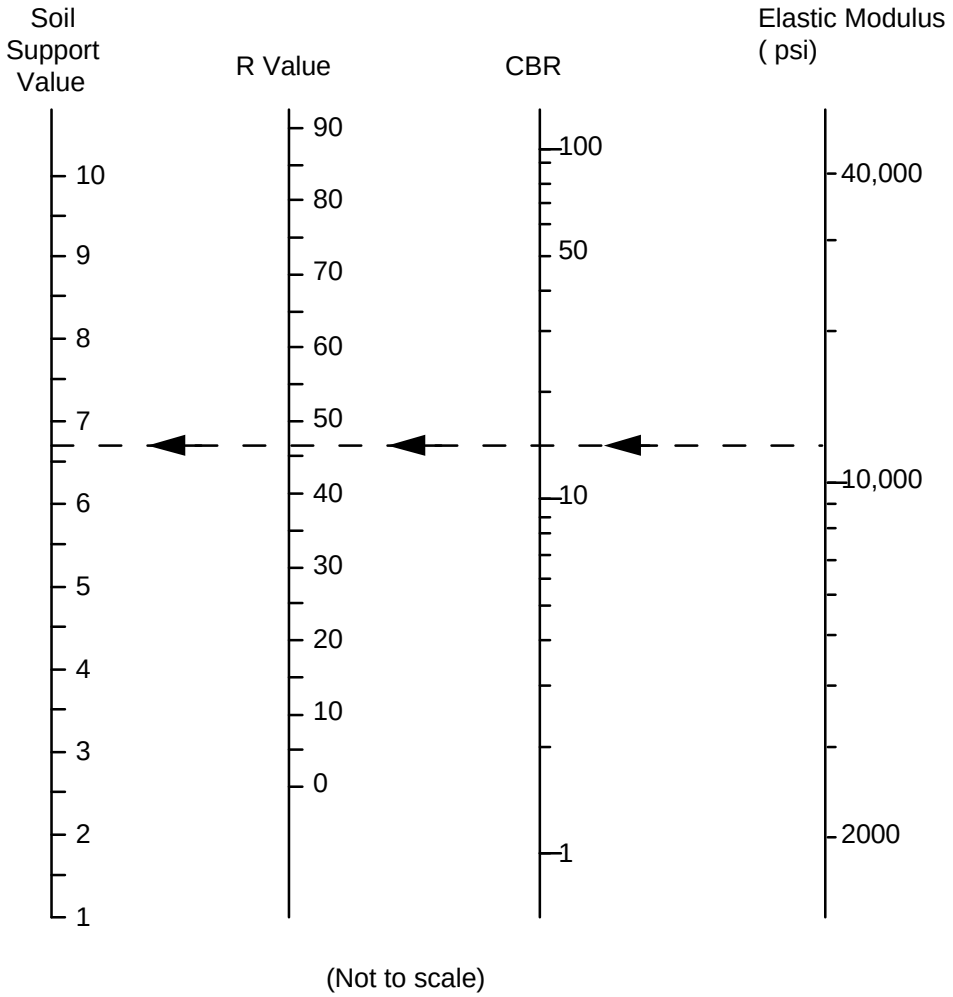


Figure 5.5. Illustration of How Soil Support is Determined from Other Test Data — Pre-1986 AASHTO Guide [after Ref. 5.2]

1.3.6.3 Subgrade Analysis: Determining S

(a) S = 3.0 for subgrade at road test site
S = 10.0 for rock base at road test site

(b) Linear S scale adopted, based on empirical observations and theoretical analysis

(c) Example of chart for determining soil support value given in Figure 5.5

1.3.6.4 Layer Coefficients

(a) AASHO Road Test (Table 5.4)

a_1 : asphalt concrete = 0.44
a_2 : crushed stone base = 0.14
a_3 : sandy gravel subbase = 0.11

The above layer coefficients imply that asphalt concrete is about three times as effective in accommodating ESALs as the crushed stone base and four times more effective than the sandy gravel subbase. The layer coefficients were originally derived from the AASHO Road Test data [5.1].

(b) Other values — Interim Guides (Figures 5.6 to 5.8)

Table 5.4. A Selection of Structural Layer Coefficients Proposed by AASHO Committee on Design, October 12, 1961 (after Ref. 5.3)

Pavement Layer	Layer Coefficient
Surface Course	
• Plant mix (high stability)	0.44*
• Sand Asphalt	0.40
Base Course	
• Crushed Stone	0.14*
• Sandy Gravel	0.07
• Bituminous-Treated	
• Coarse-Graded	0.34
• Sand Asphalt	0.30
Subbase Course	
• Sandy Gravel	0.11*
• Sand or Sandy Clay	0.07

*Established from AASHO Road Test data

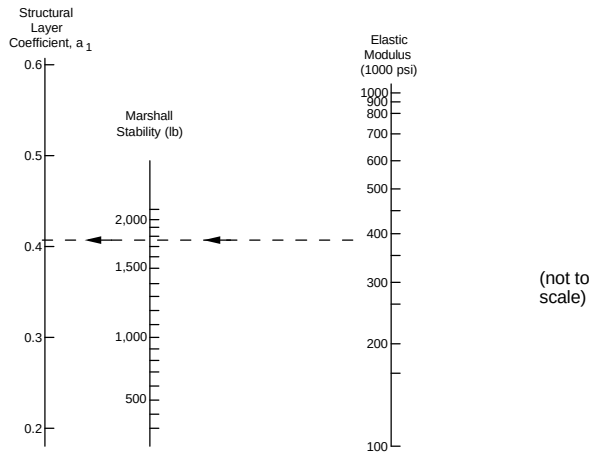


Figure 5.6. Illustration of "a $_1$" as Determined from Various Material Properties — Pre-1986 AASHTO Guide

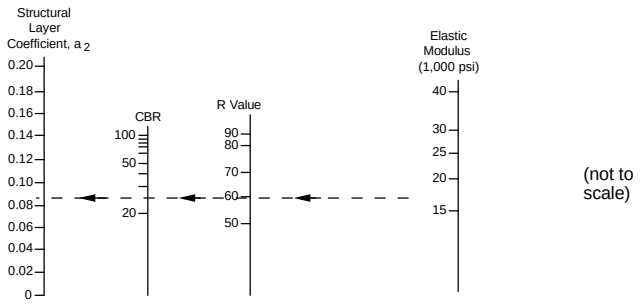


Figure 5.7. Illustration of "a $_2$" for Granular Base as Determined from Various Material Properties — Pre-1986 AASHTO Guide

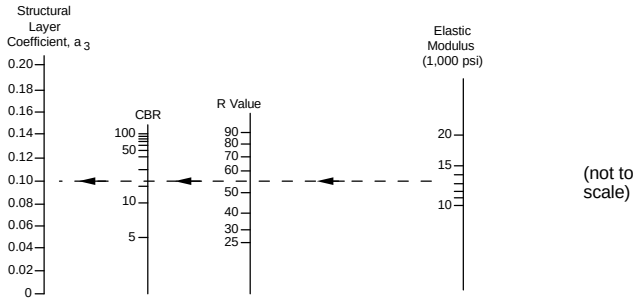


Figure 5.8. Illustration of "a $_3$" for Granular Subbase as Determined from Various Material Properties — Pre-1986 AASHTO Guide

1.3.6.5 Regional Factor, R

(a) AASHTO recommends 0.5 to 4.0 for U.S. (Figure 5.9); however, these contours are based on little information

(b) R = 1.0 at test site

(c) R = f(topo, rainfall, temperature, frost penetration, etc.)

(d) General guides

0.2 to 1.0	Roadbed frozen 5 in. (125 mm) or more
0.3 to 1.5	Roadbed dry
4.0 to 5.0	Roadbed wet, spring

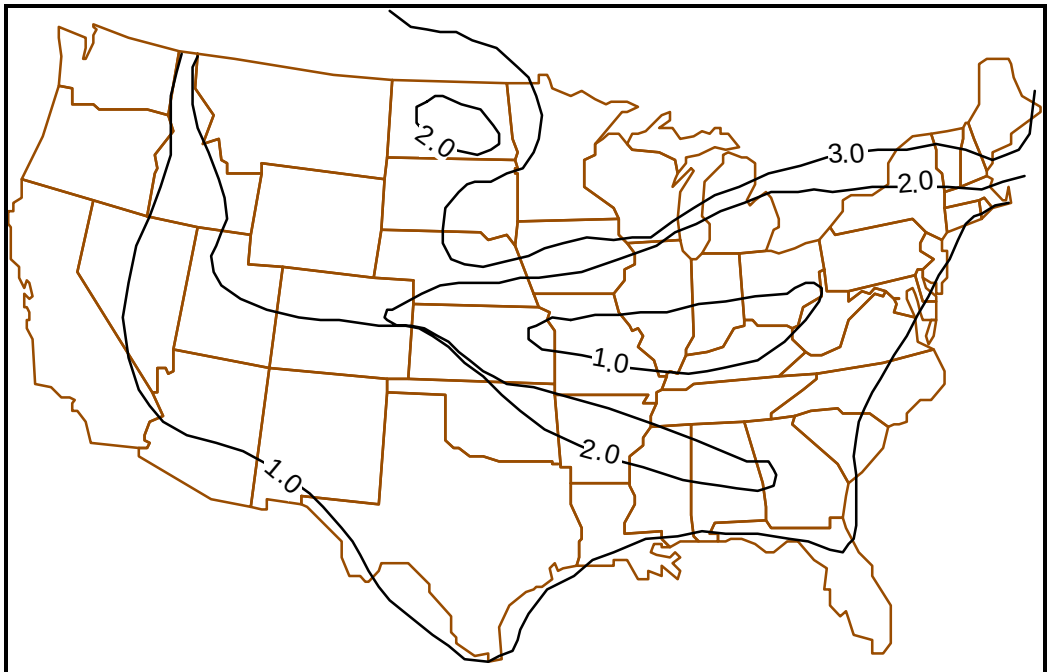


Figure 5.9. Contours of Equal Regional Factors — Pre-1986 AASHTO Guide [Redrawn from Ref. 5.2]

1.3.6.6 Thickness Design

(a) Get SN from nomograph

(b) Select layer thickness to satisfy constraints shown in Figure 5.3

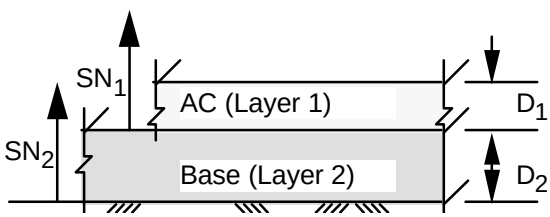
(c) Minimum thickness for each layer:

Surface	:	2 in. (50 mm)
Base	:	4 in. (100 mm)
Subbase	:	4 in. (100 mm)

1.3.6.7 Example

Design a flexible pavement structure for a highway in northwestern Washington (mountainous terrain). Use only asphalt concrete, granular base on subgrade (three layers).

SKETCH



$$\left.\begin{aligned} D^*_1 &\geq \frac{SN_1}{a_1} \\ SN_1^* &= a_1 D_1^* \geq SN_1 \\ D_2^* &\geq \frac{SN_2 - SN_1^*}{a_1} \end{aligned}\right\} \text{from Figure 5.3}$$

GIVEN

(a) Terminal Present Serviceability Index (p_t) = 2.0

(b) Regional Factor = 2.0 (estimated)

(c) Soil Support Values

(i) Subgrade modulus ≅ 10,000 psi
Soil Support Value ≅ 6.2

(ii) Base modulus ≅ 28,000 psi
Soil Support Value ≅ 9.0

} from actual nomograph used in Ref. 5.3 (Figure 5.5 illustrates process of obtaining SSV from elastic modulus)

(d) Layer coefficients (from Table 5.4)

(i) Asphalt concrete (a_1) = 0.44
(ii) Crushed aggregate base (a_2) = 0.14

TOTAL EQUIVALENT AXLES

1,000,000 18,000 lb. equivalent single axles (from SECTION 2.0)

SOLUTION

$SN_1 = 1.82$

$SN_2 = 2.76$

} from actual nomograph used in Ref. 5.3 (Figure 5.4 illustrates how SNs were obtained)

$$D^*_1 \geq \frac{SN_1}{a_1} = \frac{1.82}{0.44} = 4.1 \text{ inch} \cong 4.25 \text{ inch}$$

$$D^*_2 \geq \frac{SN_2 - a_1 D^*_1}{a_2} = \frac{2.76 - (0.44)(4.25)}{0.14} = 6.4 \cong 6.5 \text{ inch}$$

Other alternatives are possible.

2. 1986 (AND 1993) AASHTO DESIGN GUIDE

The information which follows provides insight on the use of the current version of the AASHTO Guide (1993). The AASHTO Guide was substantially revised in 1986 and further refined in 1993. With the exception of overlay design, the two versions are essentially identical. The information provided in this paragraph is rather basic and specific reference should be made to the AASHTO Guide whenever possible. In fact,

the user of this document must have a current version of the AASHTO Guide or the DARWIN computer program.

2.1 DEVELOPMENT

2.1.1 OBJECTIVES

(a) Incorporate new design tools
(b) Respond to industry concerns

2.1.2 GOALS/GUIDELINES FOR INCLUSION IN THE 1986 EDITION

(a) Characterization of strength of layers from tests
(b) Variability and reliability (emphasis on reliability)
(c) Life cycle costs
(d) Drainage
(e) National and international in scope
  (i) Cities and counties
  (ii) Other agencies
  (iii) Other countries

2.1.3 MAJOR IMPROVEMENTS TO GUIDES

(a) Reliability
(b) M_R for soil support
(c) M_R for layer coefficients
(d) Drainage
(e) Environment
(f) Load position
(g) Subbase erosion
(h) Life cycle costs
(i) Rehabilitation
(j) Pavement management
(k) Load equivalencies
(l) Traffic data
(m) Low volume roads
(n) Mechanistic/empirical design

2.1.4 PARTS OF 1986 (AND 1993) GUIDE

(a) Pavement management — Part I
(b) Design procedures for new construction — Part II
(c) Rehabilitation of existing pavements — Part III
(d) Mechanistic empirical design — Part IV

2.1.5 PARTIAL GLOSSARY (FROM REF. 5.4)

Analysis period	the period of the time for which the economic analysis is to be made; ordinarily will include at least one rehabilitation activity.
Design period	the period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal serviceability; this is also referred to as the performance period.
Drainage coefficients	factors used to modify layer coefficients in flexible pavements or stresses in rigid pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration.
Equivalent single axle loads (ESALs)	summation of equivalent 18,000 pound single axle loads used to combine mixed traffic to design traffic for the design period.
Layer coefficient (a_1, a_2, a_3)	the empirical relationship between structural number (SN) and layer thickness which expresses the relative ability of a material to function as a structural component of the pavement.
Low volume roads	a roadway generally subjected to low levels of traffic; in this guide, structural design is based on a range of 18,000 lb. ESALs from 50,000 to 1,000,000 for flexible and rigid pavements and from 10,000 to 1,000,000 for aggregate surfaced roads.
Modulus of subgrade reaction (k)	Westergaard's modulus of subgrade reaction for use in rigid pavement design (the load in pounds per square inch on a loaded area of the roadbed soil or subbase divided by the deflection in inches of the roadbed soil or subbase, psi/in.
Pavement performance	the trend of serviceability (PSI) with load applications.
Performance period	the period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal serviceability; this is also referred to as the design period.

Resilient modulus	a measure of the modulus of elasticity of roadbed soil or other pavement layer material.
Roadbed material	the material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.
Traffic equivalence factor	a numerical factor that expresses the relationship of a given axle load to another axle load in terms of their effect on the serviceability of a pavement structure. In this guide, all axle loads are equated in terms of the equivalent number of repetitions of an 18,000 lb. single axle.

2.2 PART I — PAVEMENT DESIGN AND MANAGEMENT

2.2.1 DESIGN CONSIDERATIONS

2.2.1.1 Performance Equations

(a) Refer to SECTION 2.0 and Figure 5.10. Recall that the serviceability of a pavement is expressed in terms of the present serviceability index (PSI).

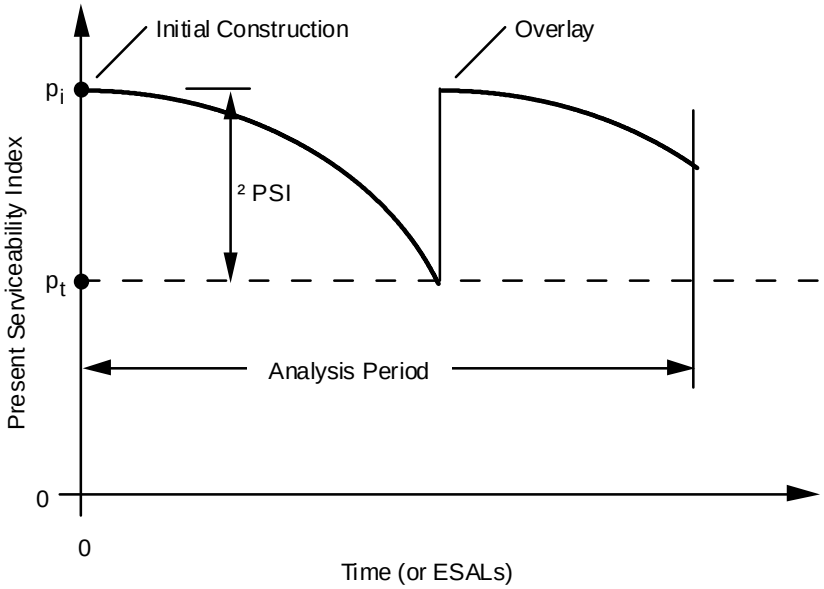


Figure 5.10. Illustration of Performance Concept

(b) Performance (Design) Equations

The pre- and post-1986 AASHTO performance equations are shown in Figure 5.11. By inspection, by comparing these two equations

- $(z_R)(s_o)$ is the added reliability term
- ΔPSI replaces $4.2\text{-}p_t$
- $(2.32)(\log M_R) - 8.07$ replaces the regional factor (climate) term $\left(\frac{1}{R}\right)$ and the soil support value term $(0.372(S\text{-}3.0))$.

Original AASHTO Performance (Design) Equation

$$\log W_{18} = (9.36)(\log (SN+1)) - 0.20$$

$$+ \frac{\log\left[\frac{4.2\text{-}p_t}{4.2\text{-}1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}}$$

$$+ \log\frac{1}{R} + 0.372(S\text{-}3.0)$$

Revised AASHTO Performance (Design) Equation

$$\log W_{18} = (z_R)(S_o) + (9.36)(\log(SN+1)) - 0.20$$

$$+ \frac{\log\left[\frac{\Delta PSI}{4.2\text{-}1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}}$$

$$+ (2.32)(\log M_R) - 8.07$$

Figure 5.11 "Old" and"New" AASHTO Flexible Pavement Performance Equations

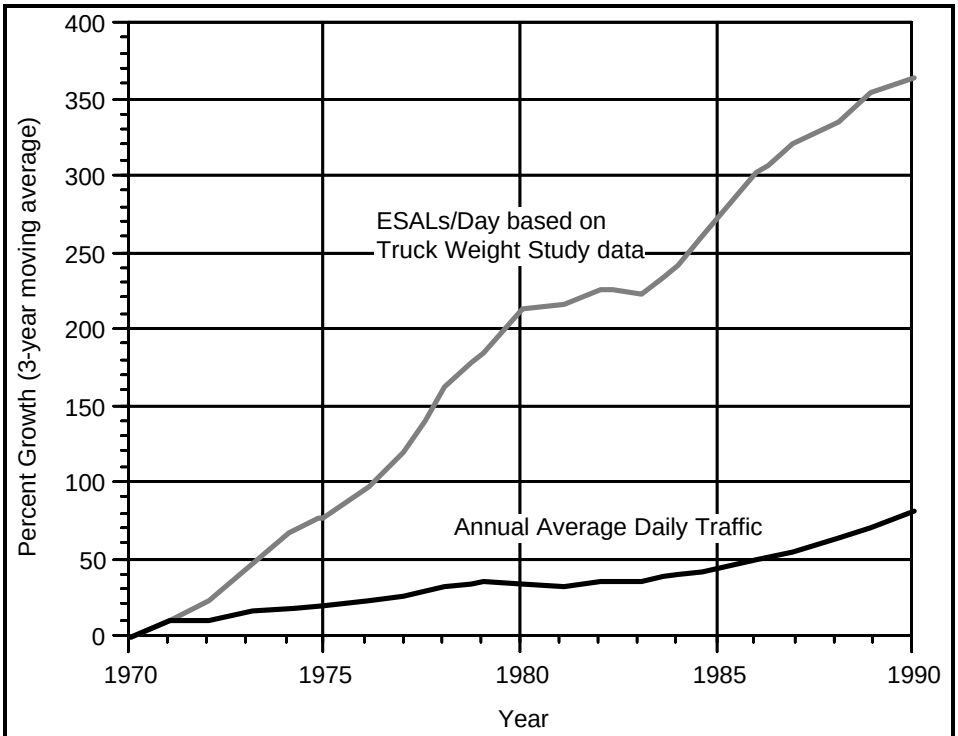


Figure 5.12. Growth of 18,000 lb. ESALs/Day for the Rural Interstate System [replotted from Ref. 5.6]

2.2.1.2 Traffic

(a) Truck Factors

As described in SECTION 2.0, Paragraph 1, the traffic input for design is in terms of the number of 18,000 lb. equivalent single axle loads over a specific period of time. Many state DOTs develop "truck factors" either for each major truck type or for an "average" truck to convert mixed truck traffic to 18,000 lb. ESALs. Table 5.5 contains a state summary of typical 18,000 lb. ESALs/truck (weighted average of two axle-six tire single unit trucks and above). This information was collected by the FHWA from state DOT weight station surveys [5.5].

(b) ESAL Trends

The designer must be alert to possible increases in the truck mix or axle load changes (or both) with time for the purpose of estimating total 18,000 lb. ESALs. To illustrate this cautionary note, Figure 5.12 is included. This figure is replotted data from "Highway Statistics 1990" [5.6] which is published by the FHWA. The

Table 5.5 ESALs per Truck For State DOTs [after Ref. 5.5]

Ordered by Increasing ESALs		Ordered by State	
State	18,000 lb ESALs/Truck*	State	18,000 lb ESALs/Truck*
Connecticut	0.345	Alabama	0.928
New Hampshire	0.367	Arizona	1.154
Pennsylvania	0.517	Arkansas	0.712
Vermont	0.559	California	1.156
Florida	0.666	Colorado	0.893
Arkansas	0.712	Connecticut	0.345
Maryland	0.719	Florida	0.666
Virginia	0.725	Georgia	0.925
Michigan	0.738	Idaho	1.027
Indiana	0.756	Illinois	0.830
North Carolina	0.779	Indiana	0.756
Kansas	0.792	Iowa	1.023
Wisconsin	0.794	Kansas	0.792
Louisiana	0.804	Kentucky	0.885
Oklahoma	0.820	Louisiana	0.804
Texas	0.826	Maine	1.167
Illinois	0.830	Maryland	0.719
Minnesota	0.841	Michigan	0.738
Mississippi	0.841	Minnesota	0.841
New Mexico	0.882	Mississippi	0.841
Kentucky	0.885	Missouri	0.902
Colorado	0.893	Montana	1.219
Missouri	0.902	Nebraska	1.215
New York	0.912	Nevada	1.220
Ohio	0.924	New Hampshire	0.367
Georgia	0.925	New Jersey	1.175
Alabama	0.928	New Mexico	0.882
Tennessee	0.930	New York	0.912
West Virginia	0.941	North Carolina	0.779
Iowa	1.023	North Dakota	1.082
Idaho	1.027	Ohio	0.924
Washington	1.028	Oklahoma	0.820
Utah	1.066	Oregon	1.136
North Dakota	1.082	Pennsylvania	0.517
South Dakota	1.088	South Dakota	1.088
Oregon	1.136	Tennessee	0.930
Arizona	1.154	Texas	0.826
California	1.156	Utah	1.066
Maine	1.167	Vermont	0.559
New Jersey	1.175	Virginia	0.725
Nebraska	1.215	Washington	1.028
Montana	1.219	West Virginia	0.941
Nevada	1.220	Wisconsin	0.794
Wyoming	1.511	Wyoming	1.511

*For two axle-six tire units and above

data shows that ESALs/day on a typical Rural Interstate has increased by about 360 percent during the past 20 years, while the ADT increased only 100 percent. To illustrate how "traffic mix" can change, the volume percentages for various vehicle mixes in 1970, 1980, and 1990 follow [after Ref. 5.6 and related references for 1970 and 1980 data]:

Vehicle Classification	Survey Year 1970	1980	1990
Passenger cars and buses, and light single unit trucks	86%	82%	78%
Heavy single unit trucks	3%	3%	4%
3 and 4 axle combinations	3%	1%	2%
5 axle or more combinations	8%	14%	16%
	100%	100%	100%

The above data show how truck volumes have increased since 1970. Most significantly, five-axle trucks (or larger) have doubled as a percentage of the total traffic.

To further reinforce the need to estimate future traffic growth, the total vehicle miles traveled in Washington state was estimated to be (10-year intervals):

Year	Vehicle Miles Traveled (billions)[1]
1965	15
1975	25
1985	35
1995 (projected)	52

[1]From WSDOT "Key Facts — A Summary of Useful Transportation Data," April 1994

Thus, VMT has increased about 250 percent over the last 30 years (1965-1995) and about 50 percent over the last 10 years (1985-1995). The period from 1965 to current is approximately the time span the Interstate Highway System has existed in Washington state.

(c) 1986 and 1993 AASHTO Design Guides

Appendix D in the AASHTO Design Guide [5.4] should be used in developing 18,000 lb. ESAL estimates for the design lane. Further, a sample of some of the currently used axle load equivalency factors are shown in Table 5.6. Basically, the AASHTO tables are for single, tandem and triple axles for terminal serviceability indices of 2.0, 2.5 and 3.0.

Table 5.6. A Sample of AASHTO Traffic Equivalence Factors for Flexible Pavement (from Ref. 5.4)

Single Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
2	8.9	0.0002	0.0004	0.0008	0.0002	0.0003	0.0006	0.0002	0.0002	0.0002
10	44.5	0.075	0.078	0.082	0.090	0.118	0.168	0.076	0.080	0.086
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.61	1.60	1.56	1.49	1.41	1.59	1.55	1.51
30	133.4	10.4	10.3	10.2	9.2	7.9	6.5	9.2	7.8	6.3
40	177.9	39.6	39.3	38.8	34.3	28.5	22.2	31.2	23.0	15.3

Tandem Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
10	44.5	0.007	0.008	0.011	0.008	0.011	0.020	0.006	0.006	0.007
20	89.0	0.103	0.107	0.113	0.124	0.162	0.232	0.105	0.110	0.119
30	133.4	0.607	0.611	0.616	0.646	0.703	0.788	0.617	0.633	0.656
34	151.2	1.06	1.06	1.07	1.08	1.11	1.15	1.07	1.08	1.09
40	177.9	2.22	2.21	2.21	2.15	2.06	1.94	2.18	2.14	2.08
50	222.5	6.15	6.12	6.08	5.64	5.03	4.31	5.77	5.28	4.70
60	267.0	14.3	14.2	14.1	12.7	10.9	8.9	12.6	10.7	8.6
70	311.5	29.2	29.0	28.7	25.6	21.5	17.0	24.3	19.2	13.9

Triple Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
20	89.0	0.024	0.027	0.031	0.029	0.042	0.069	0.023	0.024	0.026
30	133.4	0.125	0.129	0.136	0.149	0.195	0.279	0.126	0.133	0.143
40	177.9	0.434	0.439	0.447	0.481	0.554	0.671	0.443	0.459	0.483
50	222.5	1.17	1.17	1.18	1.20	1.24	1.30	1.18	1.20	1.22
60	267.0	2.67	2.67	2.66	2.59	2.48	2.34	2.63	2.58	2.50
70	311.5	5.40	5.38	5.34	5.03	4.57	4.02	5.15	4.84	4.44
80	356.0	9.98	9.92	9.84	9.05	7.95	6.67	9.18	8.21	7.06
90	400.5	17.2	17.1	16.9	15.3	13.2	10.7	15.2	12.9	10.4

If the ESALs have been developed for the total two-way traffic, the designer must assess whether the ESALs are equally split for each direction (referred to as the "directional distribution"). After the one-way ESALs have been calculated, then AASHTO recommends the following range of factors to recognize the distribution of ESALs on multi-lane highways (referred to as the "lane distribution factor"):

Number of Lanes in One Direction	Percent ESALs in Design Lane
1	100
2	80-100
3	60-80
4 or more	50-75

For example, if the total two-way ESALs for I-90 in District 5 is 20,000,000, the highway has a total of four lanes, and the directional distribution is 50 percent, then the lane distribution factor can range between 80 to 100 percent. Thus, the estimated design ESALs could be

(20,000,000) (0.50) (1.00) = 10,000,000 (maximum)

or

(20,000,000) (0.50) (0.80) = 8,000,000 (minimum)

Unless site-specific information indicates otherwise, use the high end of the lane distribution factors. Thus, for this example, use 10,000,000 ESALs.

2.2.1.3 Soils and Materials

(a) Introduction

The material property used to characterize the roadbed soil and other layers is resilient modulus (M_R). This material property was discussed in SECTION 2.0. Some of the reasons for adopting resilient modulus include:

(i) not identified with any specific agency,

(ii) fundamental engineering property,

(iii) techniques currently available for characterizing resilient modulus using nondestructive testing equipment,

(iv) resilient modulus now a standard test procedure (AASHTO T274),

(v) if initial equipment investment too high, possible to use correlation with other field or laboratory tests,

(vi) resilient modulus test is not too complex, familiarity and experience should reduce current problems with application, and

(vii) reservoir of information.

A schematic of the laboratory resilient modulus test is shown as Figure 5.13. The stress sensitivity of most unstabilized pavement materials is illustrated in Figure 5.14. Some of the correlations for determining resilient modulus from other material properties are shown in Paragraph 2.3 (specific correlations are shown in Part I of the *AASHTO Guide* as well).

(b) In situ resilient modulus determination

A method which has received recent, extensive development work is the "backcalculation" method. This is essentially a mechanistic evaluation (usually elastic analysis — refer to SECTION 4.0 of these notes) of pavement surface deflection basins generated by various pavement deflection devices. Backcalculation is one where measured and calculated surface deflection basins are matched (to within some tolerable error) and the associated layer moduli required to achieve that match are determined. The backcalculation process is usually iterative and normally done with software that can run on microcomputers. An illustration of the backcalculation process is shown in Figures 5.15 and 5.16.

(c) Special construction considerations for roadbed soils

(i) Need appropriate density requirement.

(ii) Excessively expansive or resilient soils should receive special treatment.

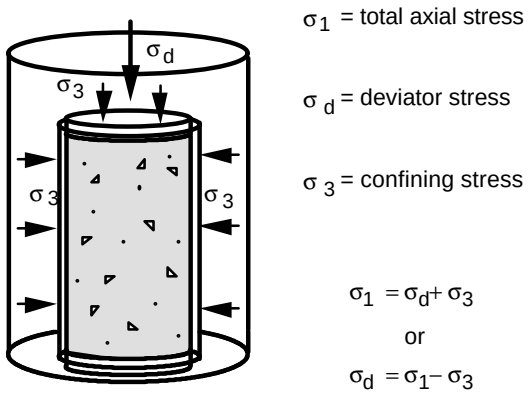
(iii) Frost susceptible soils may need special treatment such as

- removal and replacement,
- covered by other materials to a depth which minimizes detrimental effects,
- etc.

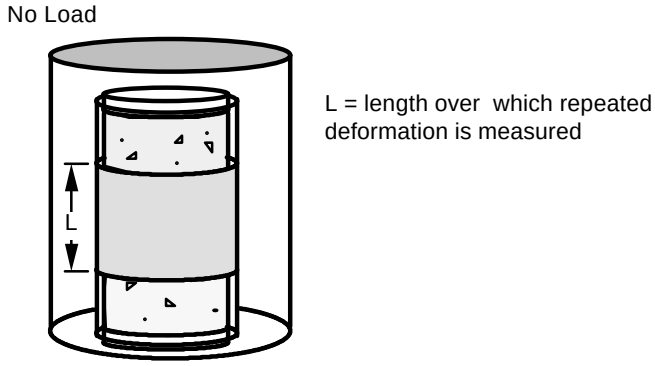
(iv) Organic soils may require special treatment.

(v) Subsurface drainage should be considered especially where major flows of groundwater can be expected.

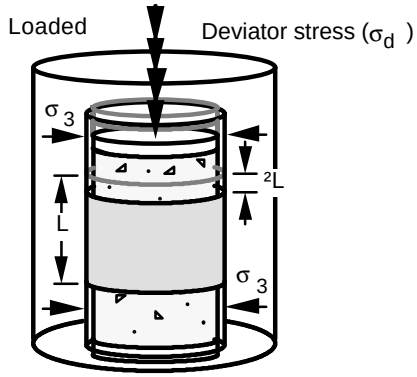
(vi) Need for "working platform" during construction.



a) Stresses Acting on Triaxial Specimen

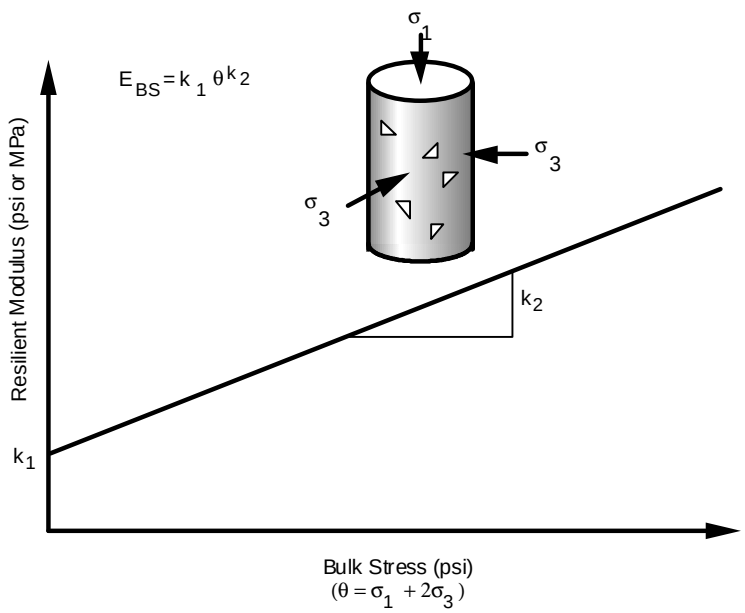


b) Gage Length for Measurement of Strain on Triaxial Specimen

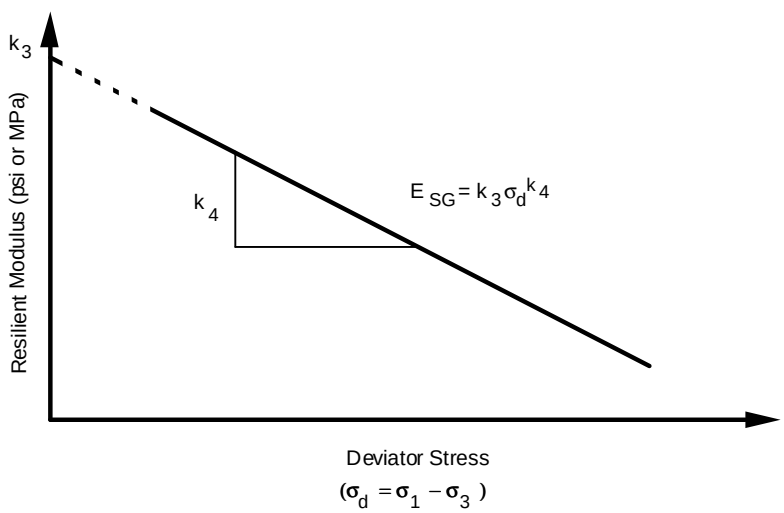


c) Deformation of Triaxial Specimen Under Load

Figure 5.13. Schematic Diagram of Laboratory Resilient Modulus Test



a) Resilient Modulus vs. Bulk Stress for Unstabilized Coarse Grained Materials



b) Resilient Modulus vs. Deviator Stress for Unstabilized Fine Grained Materials

Figure 5.14. Stress Sensitivity of Resiliency Moduli for Unstabilized Materials

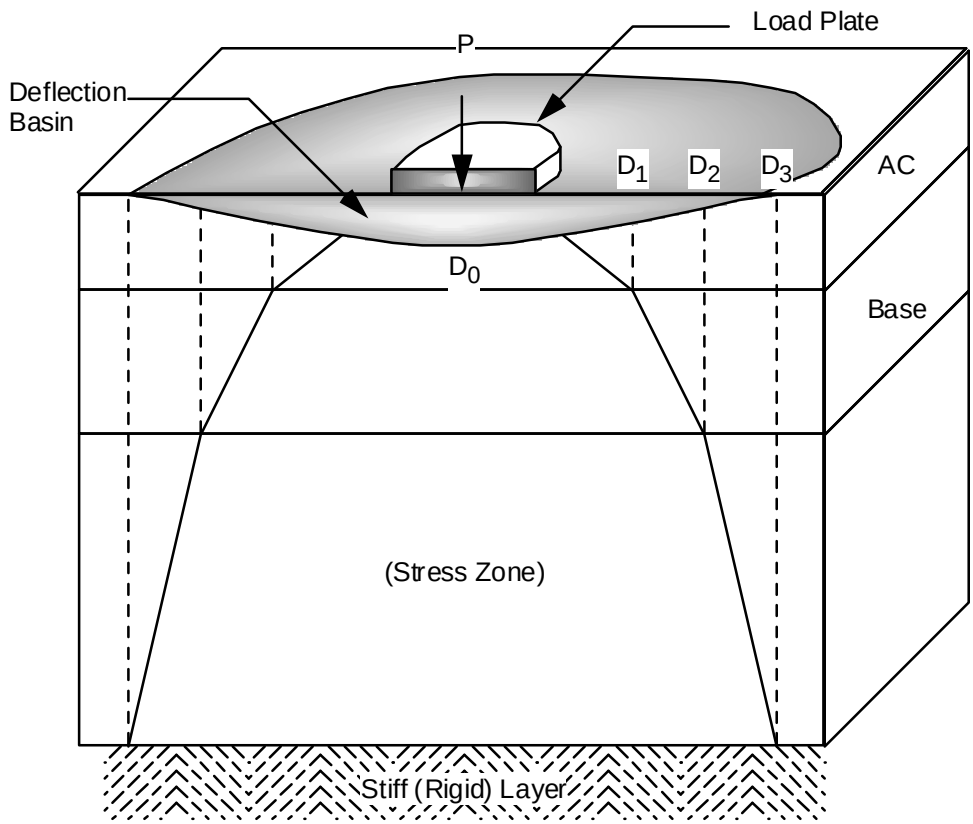


Figure 5.15. Illustration of Backcalculation to Estimate Layer Moduli

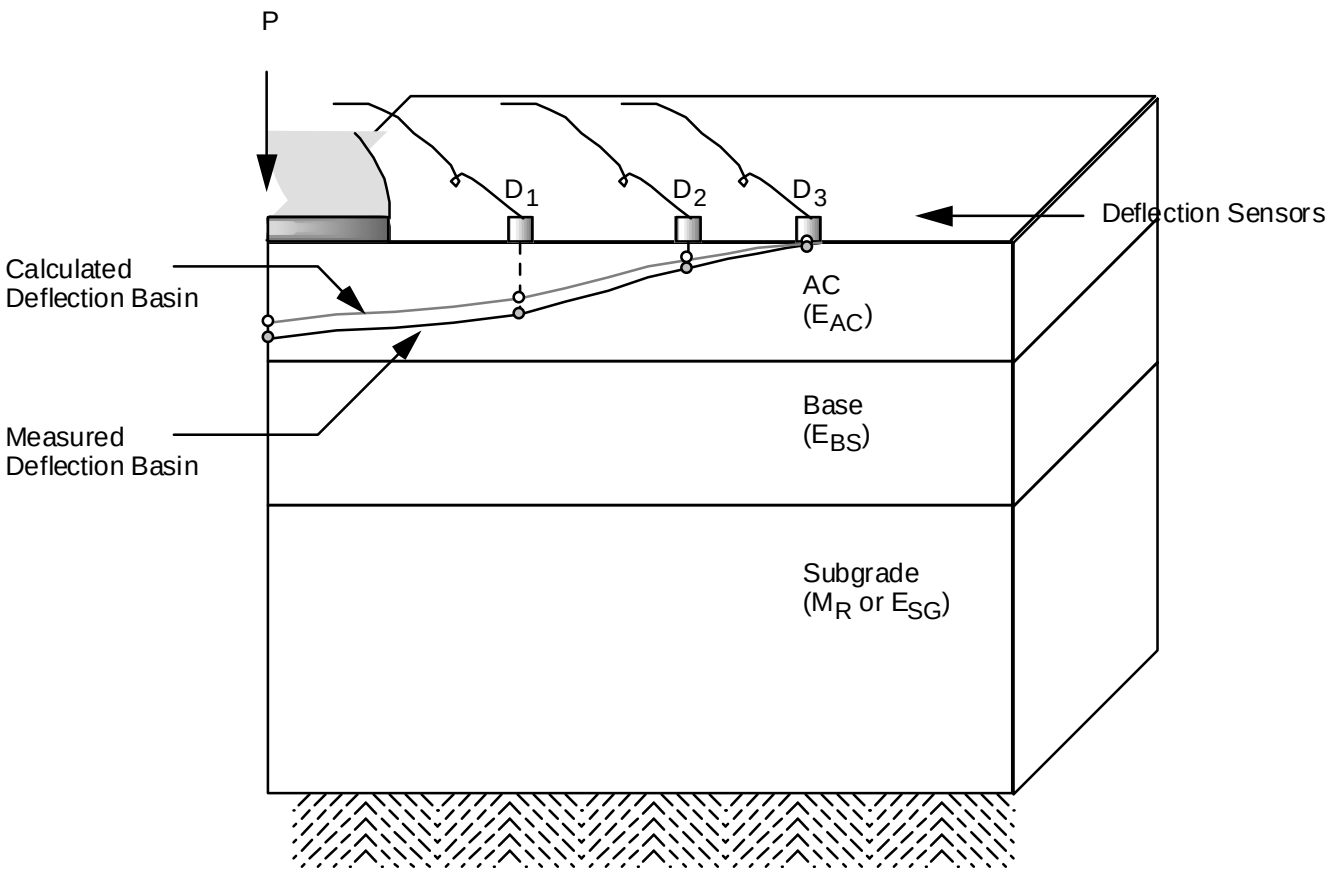


Figure 5.16. The Backcalculation Process - Matching Measured and Calculated Deflection Basins

(d) Seasonal variation of moduli

Figure 5.17 is used to illustrate that pavement layer moduli can change significantly throughout the year. The specific information contained in the figure is pavement surface deflections versus time. The greatest differences in pavement deflection's were for 1 inch asphalt concrete surfaces (as one would expect). This data thus suggests major changes can be expected for one or more of the pavement layers (hence layer moduli). Such moduli changes are conceptually illustrated by Figure 5.18.

Provision is made in the *AASHTO Guide* to accommodate roadbed moduli changes by determination of an "effective annual roadbed soil resilient modulus" (refer to Figure 5.19). The derivation of effective roadbed resilient modulus is contained in Appendix 5.2. In essence, it comes to the following relationship:

$$(M_R)_{eff} = (3005)\,(\bar{u}_f)^{-0.431}$$

where $(M_R)_{eff}$ = effective roadbed resilient modulus weighted by seasonal variation, and

$\bar{u}_f$ = average relative seasonal damage.

2.2.1.4 Environment

Refer to *AASHTO Guide*, Part I, SECTION 1.7

2.2.1.5 Drainage

(a) Need for subsurface drainage based on:
- (i) Frequency of rainfall
- (ii) Amount of rainfall
- (iii) Quantity of water to be drained
- (iv) Thickness of drainage layer
- (v) Permeability of drainage layer
- (vi) Hydrostatic head

(b) Refer to *AASHTO Guide*, Part I, SECTION 1.8

2.2.1.6 Shoulder Design

Refer to *AASHTO Guide*, Part I, SECTION 1.9

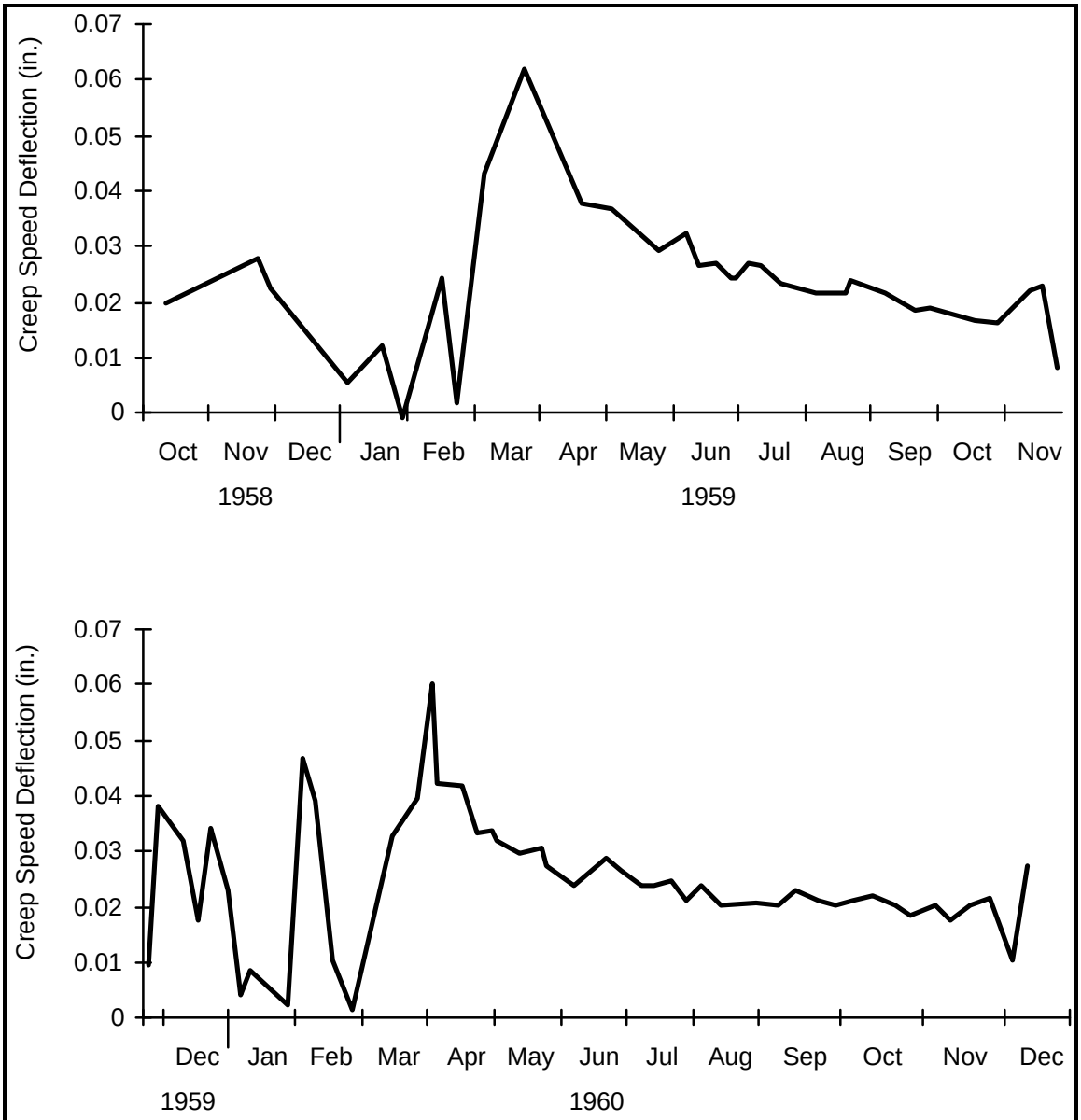


Figure 5.17. Illustration of Seasonal Pavement Surface Deflections for 1 Inch Thick Asphalt Concrete Surfaced Sections at the AASHO Road Test [Redrawn from Ref. 5.1]

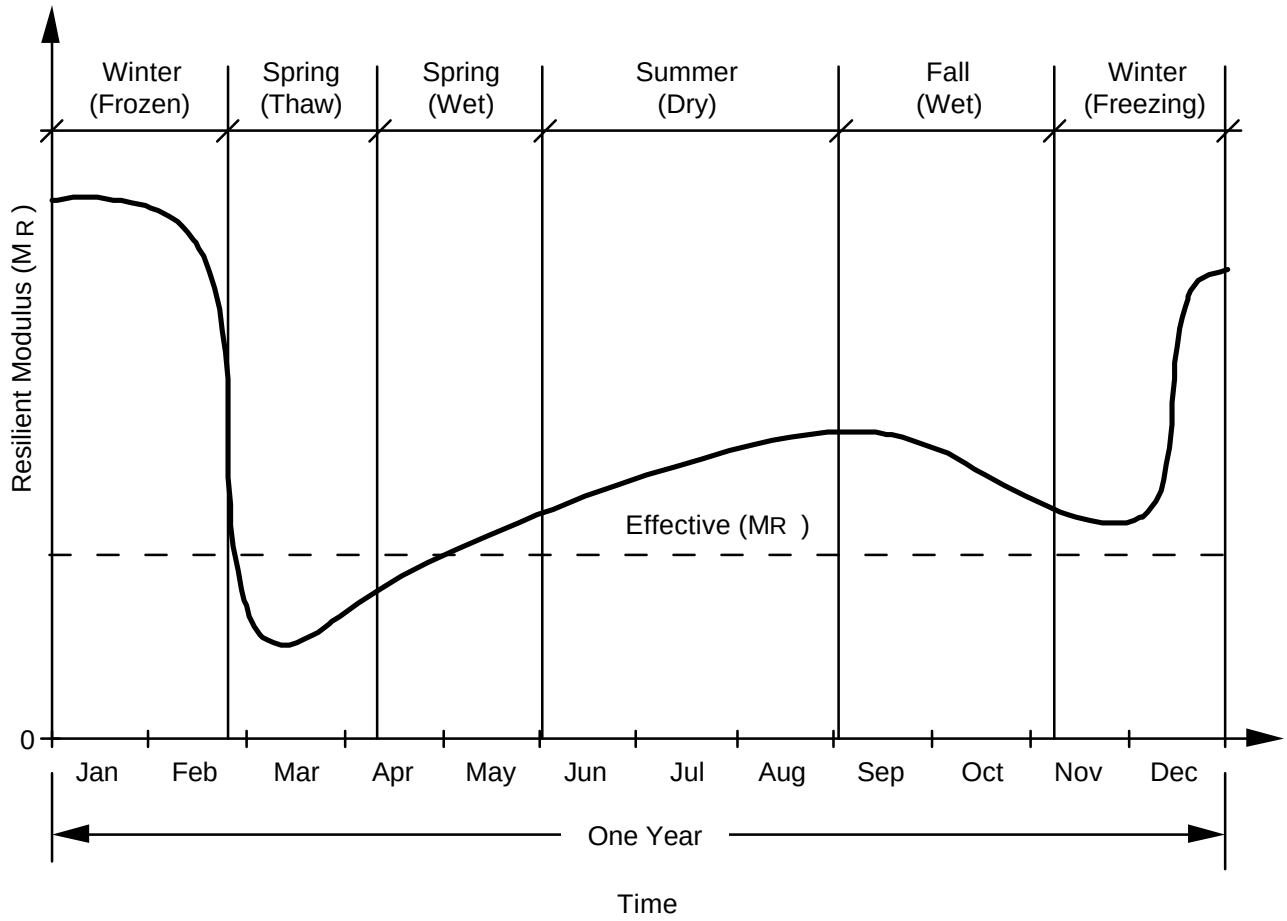


Figure 5.18. Concept of Seasonal Roadbed Soil Variation

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.	20,000	0.01
	20,000	0.01
Feb.	20,000	0.01
	5,000	0.31
Mar.	3,000	1.01
	4,000	0.52
April	4,000	0.52
	4,000	0.52
May	6,000	0.20
	7,000	0.14
June	7,000	0.14
	8,000	0.10
July	9,000	0.08
	10,000	0.06
Aug.	11,000	0.05
	11,000	0.05
Sept.	12,000	0.04
	12,000	0.04
Oct.	10,000	0.06
	9,000	0.08
Nov.	7,000	0.14
	6,000	0.20
Dec.	20,000	0.01
	20,000	0.01
Summation: $\sum u_f$ =		4.31

$$\text{Average: } \bar{u}_f = \frac{\sum u_f}{n} = \frac{4.31}{24} = 0.18$$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = 6,300 (corresponds to $\bar{u}_f$)

$u_f = (1.18 \times 10^8)(M_R)^{-2.32}$

$M_{R_{eff}} = (3005)(\bar{u}_f)^{-0.431}$

Figure 5.19. Chart for Estimating Roadbed Soil Resilient Modulus for Flexible Pavements

2.2.2 ECONOMIC EVALUATION (LIFE CYCLE COSTS)

2.2.2.1 Purpose

To determine the construction or rehabilitation alternatives that provide the required pavement performance with the least cost over the analysis period to both the "owner" and "user" of the pavement. This evaluation process will be treated with greater detail in SECTION 8.0 of these notes. The following should be considered as an overview.

2.2.2.2 Components of Life Cycle Cost Analysis

(a) Present worth method

This is a method that involves the conversion of all present and future expenses to a basis of today's costs. Basic present worth components include [5.7]:

(i) Present worth — single payment

$$PW = F\left[\frac{1}{(1+i)^n}\right]$$

where PW = present worth,
F = future sum of money at the end of "n" years from now that is equal to PW at discount rate "i,"
n = number of years,
i = discount rate (annual), and

SPW = single payment present worth factor

$$= \frac{1}{(1+i)^n}$$

(ii) Present worth — uniform series of payments

$$PW = A\left[\frac{(1+i)^n - 1}{i\,(1+i)^n}\right]$$

where A = end-of-year payments in a uniform series for "n" years that is equal to PW at discount rate "i",
n = number of years,
i = discount rate (annual), and

UPW = uniform present worth factor

$$= \frac{(1+i)^n - 1}{i\,(1+i)^n}$$

(b) Cost components

(i) Analysis period

This is the period of time used for comparing the pavement alternatives (either new construction or rehabilitation). An analysis period may contain one or more performance periods (rehabilitation activities such as overlays). The generally recommended analysis period for comparing new design alternatives is 40 years (for rehabilitation the period might be shorter).

(ii) Construction costs

These costs are those required for the initial construction (either new construction or reconstruction) of the project.

(iii) Rehabilitation costs

These costs are those associated with rehabilitation or restoration of the project following initial construction.

(iv) User costs

These costs are those associated with the user of a project which can be related to pavement type, pavement condition, maintenance activities, or rehabilitation work. Such costs can include:

- Delay costs: due to speed changes, speed reduction and delay (idling) time.
- Operating costs: due to fuel consumption, tire wear, vehicle maintenance, etc.
- Accident costs: due to fatal, non-fatal, and property damage

On high volume highways, user costs can easily exceed construction costs.

(v) Salvage value

This is the value of the pavement structure at the end of the analysis period and can be either positive or negative (depends on whether the material has some economic value or the cost of demolition and removal exceeds any salvage value).

(vi) Energy costs

These costs are associated with the relative energy consumption of the different alternatives (normally omitted in the cost analysis).

(vii) Discount rate

This rate is the difference between the market interest rate and the inflation rate using constant dollars. There is a general consensus [5.8, 5.9, 5.10] that a discount rate of four percent is reasonable for use in costing pavement structures.

(c) General equation

PW = Construction Costs + (Maintenance Costs) (UPW)
+ (Rehabilitation Costs)(SPW) + (User Costs) (UPW)
- (Salvage Value)(SPW)

2.2.3 RELIABILITY

2.2.3.1 Definitions of Reliability

(a) "The reliability of a system is the probability that, when operating under stated environmental conditions, the system will perform its intended function adequately for a specified interval of time." [from Kapar and Lamberson, Reference 5.11]

(b) "The reliability of a pavement design — performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period*." [from *AASHTO Guide*, Reference 5.4]

2.2.3.2 Reasons for Using Reliability

(a) Provide design engineers with capability for incorporating variability in the design process.

(b) Incorporate some degree of certainty into the design process.

(c) Provide a better basis for comparing flexible and rigid pavement designs.

* Design period is the same as performance period (time from initial construction or reconstruction to terminal serviceability).

2.2.3.3 Illustration of Variability of Pavement Performance

Figure 5.20a is used to illustrate how a number of pavements of the same design (i.e. same materials and thicknesses) might perform (serviceability versus traffic). At the terminal serviceability index (p_t) these pavements will reach that level after different amounts of traffic. Thus, when these "identical" pavements reach p_t a distribution is formed (in this case a "normal distribution" or "bell shaped curve" is formed for the log of ESALs (i.e. traffic)).

Reliability as used in the *AASHTO Guide* is applied by multiplying the predicted design period ESALs (i.e. the ESALs the designer uses in the design nomograph or equation) by a factor which is ≥ 1. These multipliers are:

Reliability Level (%)	Reliability Design Factor (F_R)	Percent Increase Design Period ESALs (w_T)
50	1.00	0%
70	1.72	72%
90	3.77	277%
95	5.50	450%
99	11.15	1015%
99.9	48.70	4770%

Essentially, it comes down to the following:

$$W_t = (w_T)(F_R)$$

where W_t = ESALs predicted by the performance equation to p_t,
w_T = ESALs predicted (or estimated) for the design period,
F_R = Reliability design factor.

Thus, the effect of F_R on the above equation is to increase the ESALs used to estimate the SN (hence layer thicknesses). If the designer uses a 90 percent reliability, one is increasing the estimated design period traffic by a factor of 3.77 (or 277%). Thus the selection of a reliability level for design is not trivial. Conceptually, the result of different reliability levels (50 and 90 percent) are shown in Figure 5.20b.

F_R is determined as follows:

$$F_R = 10^{-z_R(S_o)}$$

where F_R = Reliability design factor,

z_R = z-statistic (associated with normal data distributions)

S_o = Overall standard deviation of normal distribution of errors associated with traffic prediction and pavement performance.

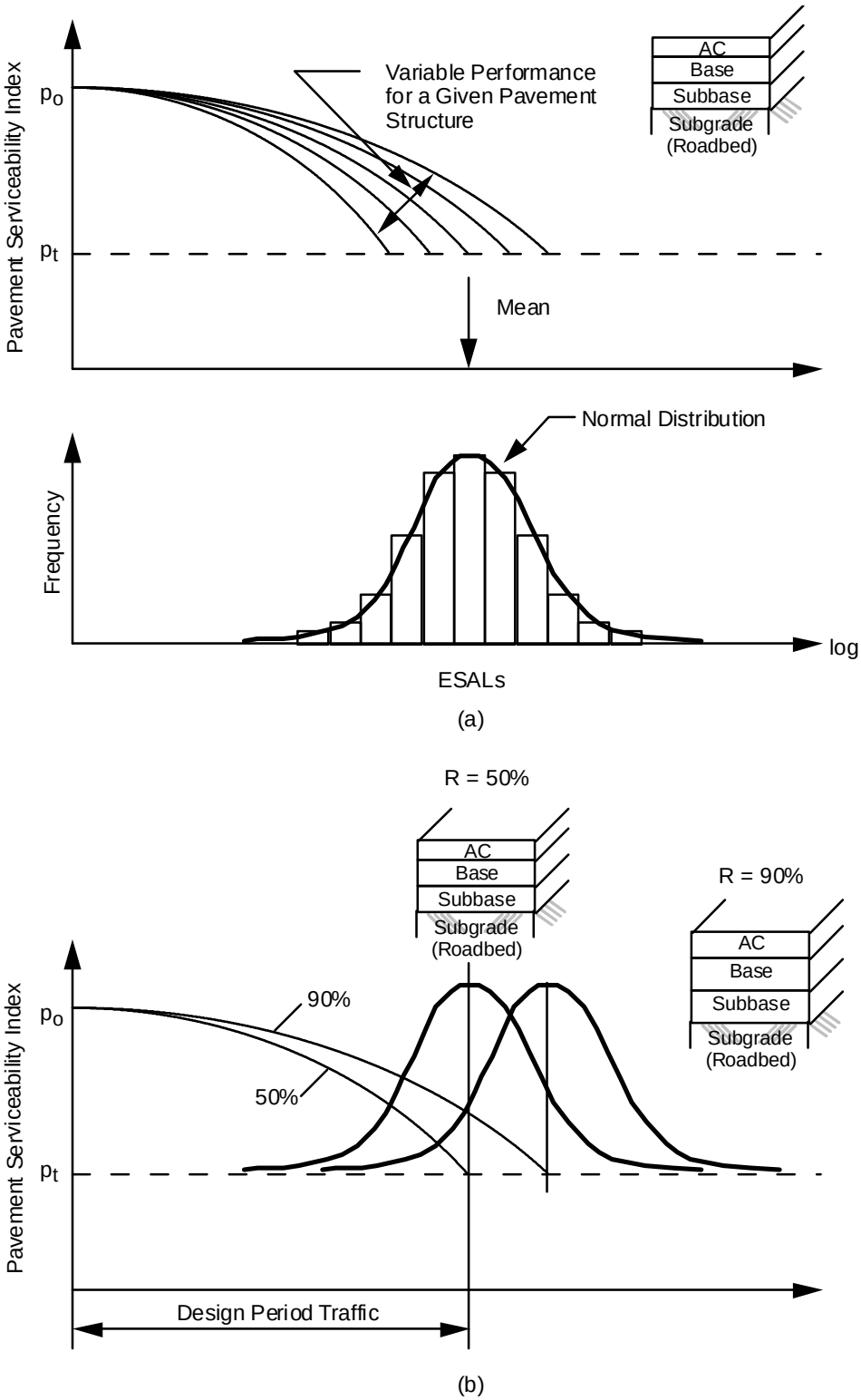


Figure 5.20. Illustration of Pavement Performance for a Given Design (a) and How Reliability Affects Performance (b)

The major sources of variation that affect pavement design and performance include:

(a) Construction: layer thicknesses, material strengths, etc.
(b) Environment: soil, climate, etc.,
(c) Traffic forecasts or projections,
(d) Prediction error: error in performance prediction.

Reliability (%) and S_o are directly used in the *AASHTO Guide* design nomograph (refer to Figure 5.21). You can see that "R" is a measure of "z_R" and along with S_o estimates "F_R" so the "W_t" is directly determined in the nomograph.

The recommended reliability levels in the *AASHTO Guide* are a function of highway functional class as follows:

Functional Class	Urban	Rural
• Interstates, Freeways	85 - 99.9	80 - 99.9
• Principal Arterials	80 - 99	75 - 95
• Collectors	80 - 95	75 - 95
• Local	50 - 80	50 - 80

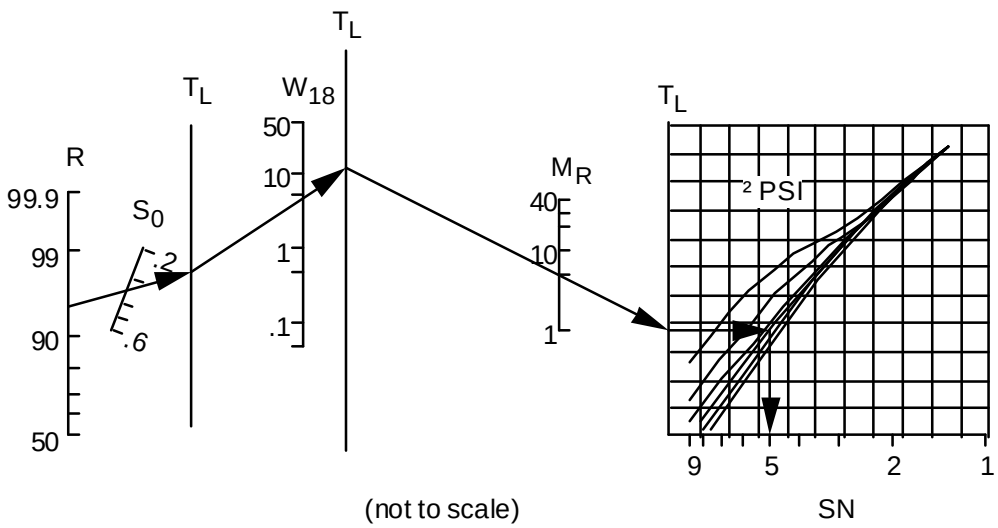


Figure 5.21. Illustration of How Reliability and S_0 Are Incorporated into Flexible Pavement Design Nomograph

2.3 FLEXIBLE PAVEMENT DESIGN PROCEDURE FOR NEW CONSTRUCTION OR RECONSTRUCTION (PART II OF *AASHTO GUIDE*)

2.3.1 OUTLINE AND BACKGROUND INFORMATION FOR USING PART II (NEW CONSTRUCTION OR RECONSTRUCTION)

2.3.1.1 Introduction

(a) Flexible pavement design includes both asphalt concrete and surface treatment surfaces.

(b) Design nomograph based on the original but slightly modified AASHO performance equation.

(c) If the designer only wants to consider serviceability loss due to traffic (and not loss due to expansive soils or frost heaving) ignore Section 3.1.3 and 3.2.4 in Part II of the *AASHTO Guide* [5.4].

2.3.1.2 Flexible Pavement Design

(a) A sketch of a nomograph solution shown in Figure 5.21.

(b) To determine Structural Number (SN)

(i) Estimate the future traffic for the performance period (ESALs or w_T). Refer to Section 2.1.2 in Part II of *AASHTO Guide*.

(ii) Select reliability (refer to Section 2.1.3 in Part II of *AASHTO Guide*).

As a starting point, consider using the following reliability levels:

Functional Class	Reliability
• Interstate	
• Urban (e.g., I-5)	95%
• Rural (e.g., I-90)	90%
• Principal Arterial	
• Urban (e.g., US99)	85%
• Rural (e.g., US2)	80%
• Collector	
• Urban (e.g., SR303)	80%
• Rural (e.g., SR172)	75%

(iii) Select or evaluate the overall standard deviation (S_o). Refer to Section 2.1.3 in Part II of *AASHTO Guide*.

S_o levels for flexible pavements can range from 0.40 (optimistic) to 0.50 (not so optimistic). Other values outside this range can be used but should be documented.

(iv) Determine the effective roadbed resilient modulus. Refer to Section 2.3.1 in Part II of *AASHTO Guide*.

(v) Select the design serviceability loss, $\Delta PSI = p_o - p_t$ (where $p_o \cong 4.5$ to 4.2 and $p_t = 3.0$, 2.5 or 2.0)

(c) Selection of layer coefficients (refer to Section 2.3 in Part II of *AASHTO Guide*)

(i) Asphalt concrete layer

Convert asphalt concrete to "a_1" (surface layer coefficient).
Refer to Figure 5.22, which is a sketch of the original nomograph in the *AASHTO Guide*.

(ii) Granular base layer

Figure 5.23 illustrates how to determine "a_2" or the following:

$$E_{BS} = k_1(\theta)^{k_2}$$

where E_{BS} = base layer modulus (psi),

θ = sum of principal stresses ($\sigma_1 + \sigma_2 + \sigma_3$) in psi, and

k_1, k_2 = regression coefficients which are a function of material type.

then $a_2 = 0.249 (\log E_{BS}) - 0.977$

(iii) Granular subbase layer

Figure 5.24 illustrations how to determine a_3 or the following

$$E_{SB} = k_1(\theta)^{k_2}$$

then $a_3 = 0.227 (\log E_{SB}) - 0.839$

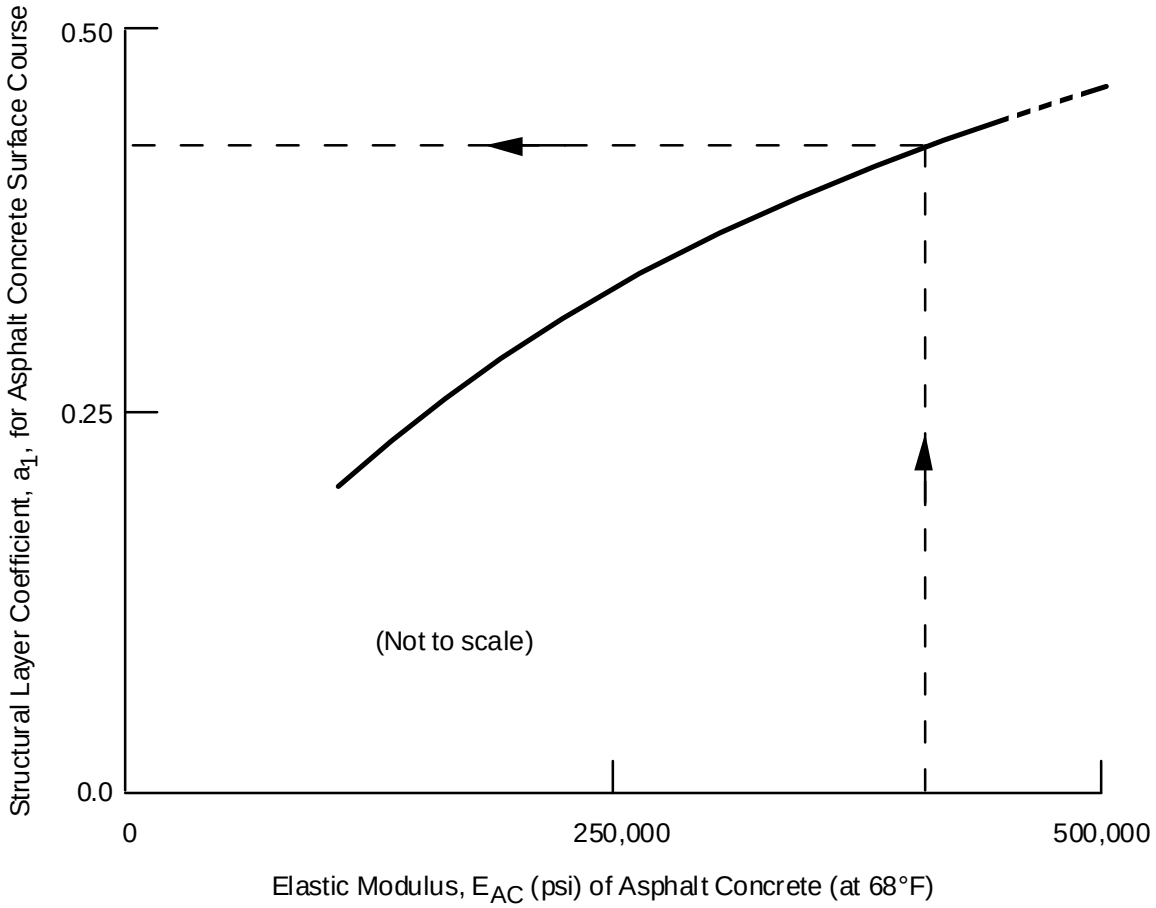


Figure 5.22. Sketch Illustrating the Determination of Structural Layer Coefficient of Asphalt Concrete [after Ref. 5.4]

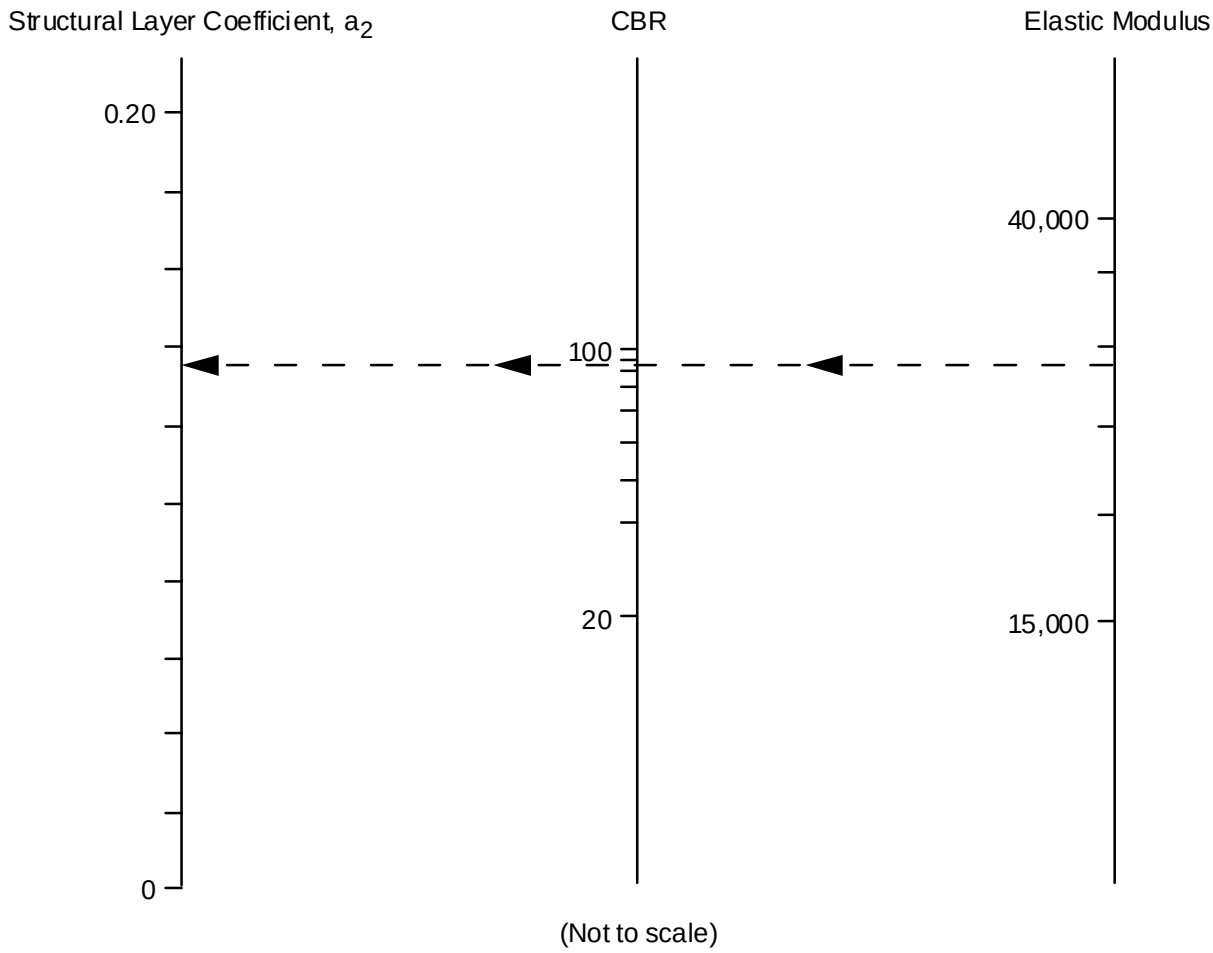


Figure 5.23. Sketch Illustrating the Determination of Structural Layer Coefficient for Base Materials [after Ref. 5.4]

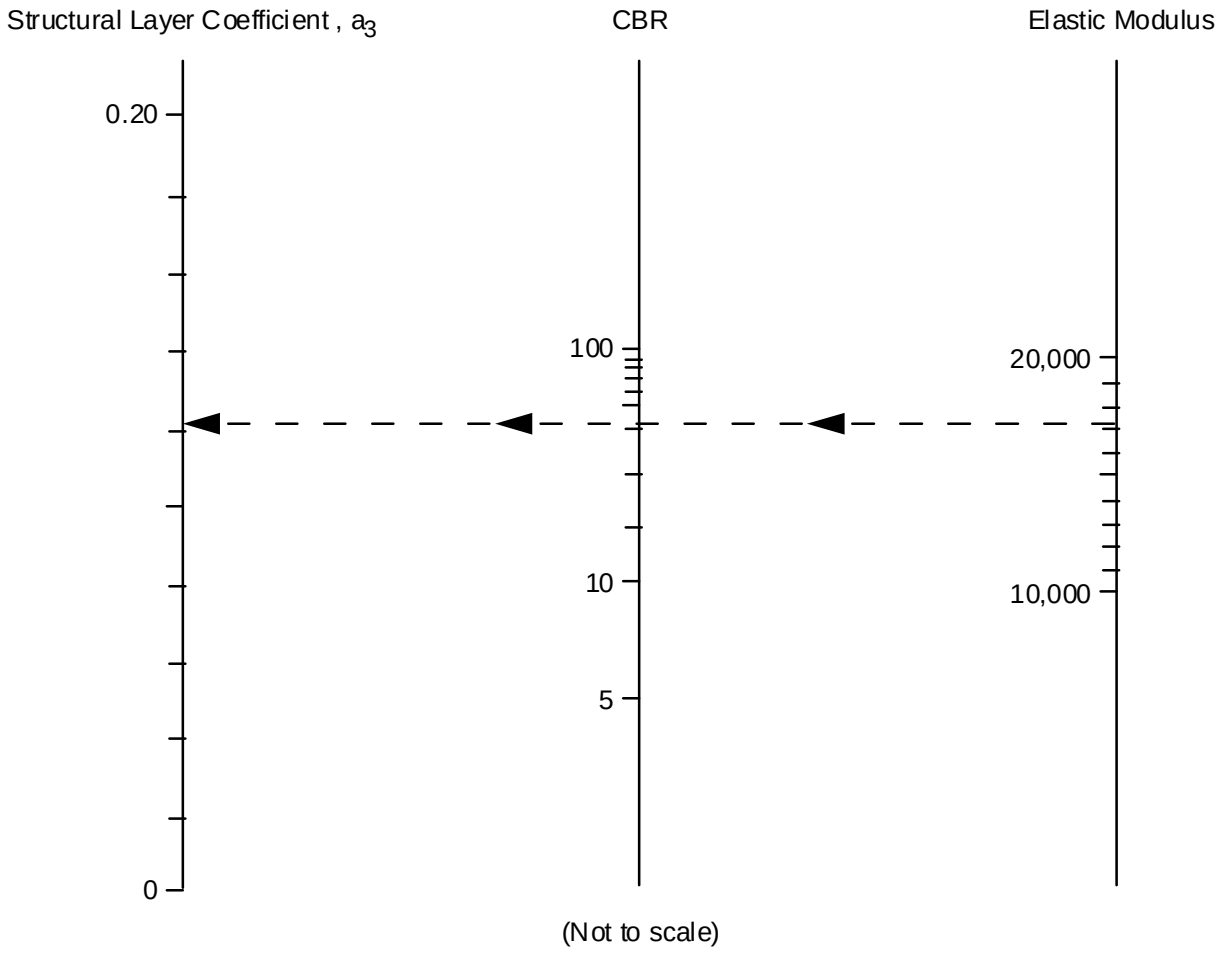


Figure 5.24. Sketch Illustrating the Determination of Structural Layer Coefficient for Subbase Materials [after Ref. 5.4]

(iv) Drainage coefficients

The drainage coefficient (m) for unstabilized base and subbase layers can be used to adjust for seasonal variation in those layers (seasonal variation for these materials generally implies changing moisture conditions). The m-values are listed in Part II, Chapter 2, Table 2.4 of the 1993 AASHTO Guide and range from 0.4 (worst) to 1.4 (best). In essence, they are used to adjust unstabilized base and subbase layer thicknesses. The m-value is selected on the basis of "quality of drainage" (a material's expected permeability) and "percent of time the pavement structure is exposed to moisture levels approaching saturation."

The m-value development was based on layered elastic analysis which was used to calculate pavement surface deflections for various combinations of structural thicknesses and layer moduli (refer to Volume II, Appendix DD, 1986 AASHTO Guide or Seed and Hicks [5.14]). The base course layer moduli used in the analysis ranged from 10,000 to 40,000 psi. These moduli were, in turn, related to the quality of drainage which was defined as follows:

Quality of Drainage	Water Removed Within
Very Poor	Does Not Drain
Poor	1 month
Fair	7 days
Good	1 day
Excellent	2 hours

The estimated permeability of the AASHO Road Test materials was about 0.1 ft/day, thus requiring about one week to drain. Further, the base course modulus used at the Road Test was estimated to be about 30,000 psi. Based on that analysis, low layer moduli were associated with low quality of drainage, as follows:

Base Course Moduli (psi)	Quality of Drainage	Recommended m-value
10,000	Very Poor	0.4
20,000	Poor	0.7
30,000	Fair	1.0
40,000	Good	1.2
50,000	Excellent	1.4

These m-values were further adjusted for duration of saturated conditions to arrive at the values shown in Table 2.4 of the 1993 AASHTO Guide.

Since the needed inputs for Table 2.4 (AASHTO Guide) may be difficult to determine or estimate, approximate levels of m-values were made a function of moduli ratios. These moduli ratios are the ratios of "seasonal" moduli to "summer" moduli. Typical moduli ratios for in-service WSDOT base courses are shown in Table 7.4 in SECTION 7.0. Two equations were developed from the original assumptions used by Seeds and Hicks [5.14] and m-values contained in Table 2.4 (AASHTO Guide) to relate layer modulus to m-value. These equations are for saturated layer conditions less than 25 percent of the time and greater than 25 percent. Further, the maximum base modulus used was 30,000 psi which is typical for as-constructed CSTC and CSBC materials. Layer moduli and m-values were regressed for the following data:

	m-value	
Base Modulus (psi)	Saturation < 25%	Saturation > 25%
30,000	1.0	0.8
20,000	0.7	0.6
10,000	0.4	0.4

The following equations result:

- For $E_{BS} \leq 30{,}000$ psi and saturated conditions less than 25 percent of the time

  $m = 0.1 + 0.00003\ (E_{BS})$

- For E_{BS} ε 30,000 psi and saturated conditions more than 25 percent of the time

  $m = 0.2 + 0.00002\ (E_{BS})$

These equations can be used to develop m-values as a function of moduli ratios (using the basic assumption that the "summer" modulus is 30,000 psi) as follows:

	Approximate m-value	
Moduli Ratio	Time Saturated < 25%	Time Saturated > 25%
1.00	1.00	0.80
0.95	0.96	0.77
0.90	0.91	0.74

Moduli Ratio	Approximate m-value: Time Saturated < 25%	Approximate m-value: Time Saturated > 25%
0.85	0.86	0.71
0.80	0.82	0.68
0.75	0.78	0.65
0.70	0.73	0.62
0.65	0.68	0.59

Thus, the above m-values associated with a specific moduli ratio can be used to adjust the base course thickness.

How can the m-values so obtained be used? The basic equation relating SN, layer coefficients (a's), layer thicknesses and drainage coefficient (m) for a three-layer system (AC, base, subgrade), is:

$$SN = a_1D_1 + a_2D_2m_2$$

For example, if the "needed" SN for calculating the base course thickness (D_2) is $SN_2 - SN_1^* = 0.92$ (see example in Paragraph 2.3.3.3 this SECTION) and $a_2 = 0.13$, then for various moduli ratios and time of saturation conditions:

- if the moduli ratio remains constant throughout the year, then $m \simeq 1.00$ and

$$D_2 = \frac{0.92}{(0.13)\,(1.00)} = 7.1 \text{ in.}$$

- if the lowest expected moduli ratio = 0.9 and is expected to occur less than 3 months each year, then $m \simeq 0.91$ and

$$D_2 = \frac{0.92}{(0.13)\,(0.91)} = 7.8 \text{ in.}$$

- if the lowest expected moduli ratio = 0.9 and is expected to occur more than 3 months each year, then $m \simeq 0.74$ and

$$D_2 = \frac{0.92}{(0.13)\,(0.74)} = 9.6 \text{ in.}$$

Use of m-values to adjust unstabilized base and subbase layer thicknesses may not be an adequate solution where severe subsurface moisture problems are not corrected in the initial pavement design and construction process.

Further, the approximate method described above for selecting an m-value should be used with caution and judgment. It is, at best, a very approximate method. Where saturated conditions exist for less than 5 percent of the time, Table 2.4 (AASHTO Guide) should be used.

Numerous sources have noted significant decreases in base layer moduli when the moisture content of dense graded materials exceed 85 percent of saturation. If such conditions are expected, the preceding suggests using thicker bases, modifying the base material to accommodate seasonal moisture changes or reducing or eliminating the sources of the water. The use of m-values fundamentally applies to the former

(d) Selection of layer thicknesses

(i) Basic equation:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where a_1, a_2, a_3 = layer coefficients of surface, base, subbase,

D_1, D_2, D_3 = layer thicknesses (inches) for surface, base, subbase, and

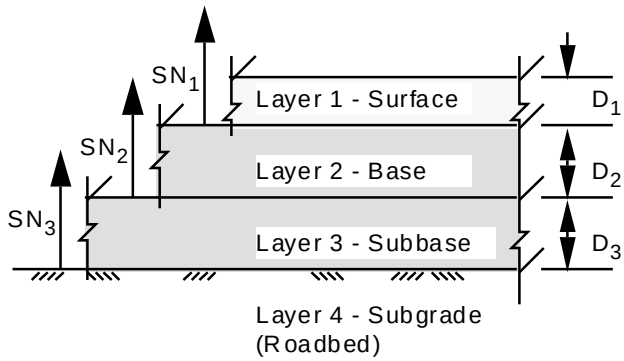
m_2, m_3 = drainage coefficients for base and subbase layers.

(ii) Minimum layer thicknesses as a function of ESALs (refer to Section 3.1.4 in Part II of *AASHTO Guide*)

	Minimum Layer Thicknesses (inches)	
Traffic (ESALs)	Asphalt Concrete	Aggregate Base
<50,000	1.0 (or surface treat.)	4.0
50,001 - 150,000	2.0	4.0
150,001 - 500,000	2.5	4.0
500,001 - 2,000,000	3.0	6.0
2,000,001 - 7,000,000	3.5	6.0
>7,000,000	4.0	6.0

(iii) Minimum layer thicknesses — another approach (originally shown in Figure 5.3 of these notes). Use layer analysis to determine individual layer thicknesses — refer to Figure 5.25. This process requires the determination of individual layer SNs required to protect layer below. To do this substitute resilient modulus for layer below in lieu of roadbed resilient modulus in design nomograph (refer back to Figure 5.21).

(iv) Check total thickness to insure it meets frost design requirements.



$$D_1^* \geq \frac{SN_1}{a_1}$$

$$SN_1^* = a_1 D_1^* \geq SN_1$$

$$D_2^* \geq \frac{SN_2 - SN_1^*}{a_2 m_2}$$

$$SN_1^* + SN_2^* \geq SN_2$$

$$D_3^* \geq \frac{SN_3 - (SN_1^* + SN_2^*)}{a_3 m_3}$$

*indicates value actually used which must be equal to or greater than the required value.

Figure 5.25. AASHTO Conceptual Flexible Pavement Layer Determination

2.3.2 WSDOT DESIGN INPUTS

2.3.2.1 Future ESALs

A number of difficulties exist in estimating ESALs. One difficulty is the annual rates to use for traffic growth (specifically truck growth rates). Refer to Paragraph 2.2.1, SECTION 6.0 for additional information. Additionally, a realistic assessment of truck axle loads is required. The information contained in SECTION 2.0, Paragraph 1.6 provides some insight into typical truck and bus axle loads, but project-specific conditions may differ. Project-specific information should be used to estimate ESALs.

2.3.2.2 Reliability

The following reliability levels are recommended for use (refer to Volume 1, SECTION 2.0, Paragraph 3.3.2):

- Highways of National Significance (HNS)
    - HNS Trunk (Interstate): 95%
    - HNS Branch (Principal Arterial): 85%
- Non-Federal Aid (Minor Arterial, Collector): 75%

2.3.2.3 Overall Standard Deviation

Unless available project-specific information suggests otherwise, use

$S_o = 0.50$

2.3.2.4 Design Serviceability Loss

Two decisions are required, selection of an initial PSI (p_o) and terminal PSI (p_t). The following are suggested:

	p_o	p_t	ΔPSI
• Rural Interstate	4.5	3.0	1.5
• Urban Interstate (minimal construction constraints)	4.5	3.0	1.5
• Urban Interstate (added HOV lanes, major construction constraints)	4.2	3.0	1.2
• Other ACP/PCC Surfaced Pavements	4.5	3.0	1.5
• BST Surfaced Flexible Pavements	4.0	3.0	1.0

A terminal PSI level of 3.0 is based, in part, on the original pavement serviceability-performance data reported by Carey and Irick [5.13]. This information was summarized in SECTION 3.0, Paragraph 3.1.1.

2.3.2.5 Effective Roadbed Resilient Modulus

The M_{Reff} is a function of seasonal roadbed (subgrade) resilient moduli. If site-specific seasonal moduli are not available, then the following moduli ratios (ratio of seasonal moduli to "summer" moduli) can be used:

	Moduli Ratio
• Western Washington	
• Winter (Dec, Jan, Feb)	0.85
• Spring (Mar, Apr, May)	0.90
• Summer (Jun, Jul, Aug, Sep)	1.00
• Fall (Oct, Nov)	0.90
• Eastern Washington	
• Winter (Jan)	1.00–1.10
• Winter/Spring (Feb, Mar, Apr, May)	0.85
• Summer (Jun, Jul, Aug, Sep)	1.00
• Fall (Oct, Nov, Dec)	0.90

The largest moduli variation observed by WSDOT in recent years is in the base course layer, with moduli ratios ranging from 0.75 to 1.00 in Western Washington, and 0.65 to 1.10 in Eastern Washington. Unfortunately, the AASHTO Guide does not have a direct way of dealing with variable base course moduli, other than adjusting the Drainage Coefficients (m's) for base and subbase layers. This will be covered in more detail shortly.

2.3.2.6 Layer Coefficients

(a) Asphalt Concrete

Class A and B mixes: $a_1 \leq 0.44$ (varies with E_{BS})

(b) Crushed Surfacing Base Course

$a_2 \leq 0.14$ (varies with E_{BS})

(b) Other Materials

Test results should be used along with information in Paragraph 2.3.5, Chapter 2, AASHTO Guide, to develop layer coefficients.

2.3.2.7 Drainage Coefficients

Drainage coefficients can be obtained for project-specific conditions directly from Table 2.4 (Part II, Chapter 2, 1993 AASHTO Guide). An alternative approach for base courses with initial, as constructed, moduli of about 30,000 psi (which is typical for CSTC and CSBC) is to use seasonal moduli ratios. Such ratios were noted in Paragraph 2.3.2.5 and Table 7.4 (SECTION 7.0 of this Guide). By combining the m-value equation (Paragraph 2.3.1.2 (c)(iv)) with various moduli ratios:

	Approximate m-value	
Lowest Expected Moduli Ratio [1,2,3]	Saturated Conditions < 25%	Saturated Conditions > 25%
1.00	1.00	0.80
0.95	0.96	0.77
0.90	0.91	0.74
0.85	0.86	0.71
0.80	0.82	0.68
0.75	0.78	0.65
0.70	0.73	0.62
0.65	0.68	0.59

Notes:

1. Summer moduli ratio taken as 1.0
2. Assumes base course modulus, as constructed, is similar to AASHO Road Test base modulus, which was about 30,000 psi.
3. Lowest expected moduli ratio should occur either less than or more than 25 percent of the time each year. For saturated conditions less than 5 percent of each year, refer to Table 2.4 (AASHTO Guide)

If the lowest moduli ratios are used which have been observed on some pavements for western and eastern Washington which occur 25 percent or more each year (refer to Table 7.4, SECTION 7.0), these values are 0.75 and 0.65, respectively. The associated m-values are 0.65 and 0.59. Thus, if these values are used, then the base course thickness would be increased about 53 percent in western Washington and 69 percent in eastern Washington. Clearly, this approach for adjusting the base course thickness as a function of a moduli ratio has serious limitations and should be used with considerable judgment.

If the modulus of an unstabilized layer is not expected to change significantly throughout the year and/or is a high permeability material, the m-value should be 1.0 or higher.

2.3.3 DESIGN EXAMPLE

Use example originally shown in Paragraph 1.8, SECTION 5.0 of these notes.

2.3.3.1 Given (or known or selected)

(a) Pavement location: Rural

(b) Pavement functional classification: Local

(c) Traffic: 1,300,000 18,000 lb. ESALs = w_{18} (from Paragraph 1.5.4, SECTION 2.0 of these notes)

(d) Performance period: 20 years

(e) Construction materials

(i) Asphalt concrete: E_{AC} = 400,000 psi
(ii) Base course: E_{BS} = 28,000 psi (unstabilized crushed surfacing)
(iii) Subbase: no subbase to be used
(iv) Subgrade: roadbed resilient modulus = 10,000 psi.

(f) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(g) Reliability: Use two levels 50% (similar to original example)
75% (top of recommended range)

(h) Overall standard deviation: $S_o = 0.50$

2.3.3.2 Solution for Reliability = 50%

(a) Determine SNs

SN_2 (protection of roadbed soil) = 2.64
SN_1 (protection of base) = 1.75

Refer to AASHTO nomograph to obtain the SNs above.

(b) Determine layer coefficients

(i) Asphalt concrete

$a_1 = 0.42$ (Use appropriate AASHTO figure. Concept illustrated in the sketch shown as Figure 5.22.)

(ii) Base course

$a_2 = 0.13$ (Use appropriate AASHTO figure. Concept illustrated in the sketch shown as Figure 5.23.)

(iii) Assume $m_2 = 1.0$ (drainage coefficient)

(c) Determine layer thicknesses

(i) Surface course thickness

$\min D_1 = SN_1/a_1 = 1.75/0.42 = 4.17 \text{ inches} \cong 4.5 \text{ inches}$
$SN_1^* = (4.5 \text{ inches})(0.42) = 1.89 > 1.75$, OK

(ii) Base course thickness

$$\min D_2 = \frac{(SN_2 - SN_1^*)}{a_2} = \frac{(2.64 - 1.89)}{0.13} = 5.77 \text{ in.} \simeq 6.0 \text{ in.}$$

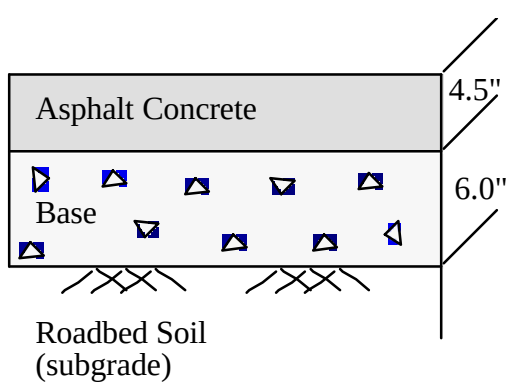
Minimum base = 6.0 inches (from Paragraph 2.3.1.2 (d)(ii), SECTION 5.0 of these notes)

Therefore, $SN_2^* = (6.0 \text{ inches})(0.13) = 0.78$

(iii) Overall SN

$SN_1^* + SN_2^* = 1.89 + 0.78 = 2.67 > 2.64$, OK

(iv) Cross section



(v) Other layer options are possible

(vi) Check total thickness to insure it exceeds one-half the maximum depth of freeze. Refer to Figures 2.34, 2.35, or 2.36 in SECTION 2.0 of this Guide. If structural thickness is not adequate, then add additional base and/or subbase.

2.3.3.3 Solution for Reliability = 75%

(a) Determine SNs

SN_2 (protection of roadbed soil) = 3.02
SN_1 (protection of base) = 2.00

(b) Determine layer coefficients

(same as Paragraph 2.3.2.2)

(c) Determine layer thicknesses

(i) Surface course thickness

$$\min D_1 = SN_1/a_1 = 2.00/0.42 = 4.76 \text{ inches} \simeq 5.0 \text{ in.}$$
$$SN_1^* = (5.0 \text{ inches})(0.42) = 2.1 \geq 2.00, \text{ OK}$$

(ii) Base course thickness

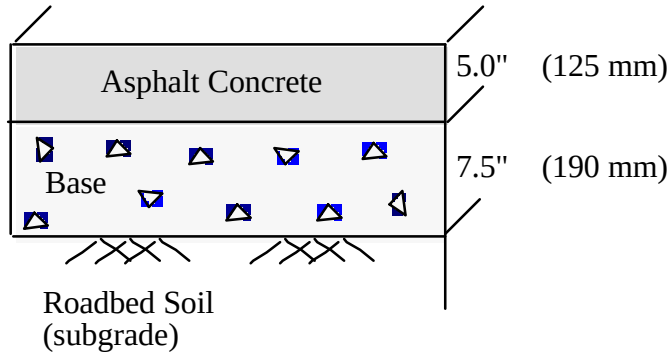
$$\min D_2 = \frac{(SN_2 - SN_1^*)}{a_2} = \frac{(3.02 - 2.1)}{0.13} = 7.08 \text{ inches} \simeq 7.5 \text{ in.}$$
$$D_2^* = 7.5 \text{ inches}$$

Therefore, $SN_2^* = (7.5 \text{ inches})(0.13) = 0.98$

(iii) Overall SN

$$SN_1^* + SN_2^* = 2.1 + 0.98 = 3.08 > 3.02, \text{ OK}$$

(iv) Cross section



(v) Other layer options are possible.

(vi) Check design for frost considerations.

2.4 FLEXIBLE DESIGN CASES

The design inputs used in the following four cases do not necessarily reflect actual design inputs used by WSDOT. The cases are intended to illustrate the AASHTO design process.

2.4.1 FLEXIBLE DESIGN CASE NO. 1

Design new flexible pavements (various conditions) for I-5 in the vicinity of N.E. 175th Street (Seattle — MP 176.35 to 177.35) for the following conditions (four lanes each direction):

(a) Pavement location: urban

(b) Pavement functional classification: Interstate

(c) Design period: 20, 30 and 40 years

(d) Construction Materials

(i) Asphalt Concrete Pavement (Class A): 500,000 psi @ 68 °F

(ii) Asphalt Concrete Base (Class E): 500,000 psi @ 68 °F

(iii) Base (CSTC): 28,000 psi

(iv) Subbase: none used

(v) Subgrade: 15,000 psi (dry months)
12,500 psi (wet months)

(e) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(f) Reliability: use three levels R = 90%
R = 95%
R = 99%

(g) Overall standard deviation (S_o): use two levels 0.35
0.50

(h) Traffic (refer to SECTION 2.0, Paragraph 1.6 for background information on definitions of terms and ESALs per vehicle)

Singles: 1872/day
Doubles: 1762/day
Trains: 247/day

(i) Calculations and Results

(i) ESALs per year
(Vehicles/day) (Lane Distribution Factor) (days/yr) (ESAL/Vehicle)

Singles:	(1872/day) (0.8) (365) (0.40)	=	218,650 ESALs/yr
Doubles:	(762/day) (0.8) (365) (1.00)	=	514,504 ESALs/yr
Trains:	(247/day) (0.8) (365) (1.75)	=	126,217 ESALs/yr
			859,371 ESALs/yr
		≈	860,000 ESALs/yr

(ii) ESALs for design periods
(use 2% annual growth rate)

20 years: $$860{,}000 \left[\frac{(1 + 0.02)^{20} - 1}{0.02}\right] \cong 21{,}000{,}000$$

30 years: $$860{,}000 \left[\frac{(1 + 0.02)^{30} - 1}{0.02}\right] \cong 35{,}000{,}000$$

40 years: $$860{,}000 \left[\frac{(1 + 0.02)^{40} - 1}{0.02}\right] \cong 52{,}000{,}000$$

(iii) Assume wet and dry months are equal (6 months each) and use Figure 5.19 and Paragraph 2.2.1.3 (d)

Month	M_R (psi)	u_f
Jan	12,500	0.037
Feb	12,500	0.037
Mar	12,500	0.037
Apr	12,500	0.037
May	15,000	0.024
Jun	15,000	0.024
Jul	15,000	0.024
Aug	15,000	0.024
Sep	15,000	0.024
Oct	15,000	0.024
Nov	12,500	0.037
Dec	12,500	0.037

$u_f = (1.18 \times 10^8)\,(12{,}500)^{-2.32} = 0.037$
$u_f = (1.18 \times 10^8)\,(15{,}000)^{-2.32} = 0.024$

$$\overline{u_f} = \frac{(6 \times 0.037) + (6 \times 0.024)}{12} = 0.0305$$

$M_{R_{eff}} = (3005)\,(0.0305)^{-0.431} \cong 13{,}500$ psi

(iv) Summary of Results[1, 2, 3, 4] (only for $S_0 = 0.50$)

Design Period ESALs	Pavement Layer	Reliability		
		90% ($S_0 = 0.50$)	95% ($S_0 = 0.50$)	99% ($S_0 = 0.50$)
21,000,000 (20 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	5.1" (130 mm)	5.8" (150 mm)	7.2" (180 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.79	5.10	5.70
35,000,000 (30 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	6.0" (150 mm)	6.7" (170 mm)	8.1" (205 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	5.17	5.49	6.11

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
52,000,000 (40 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	6.6" (170 mm)	7.4" (190 mm)	8.8" (225 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	5.47	5.80	6.44

1 Note: Layer thicknesses do not include periodic resurfacing (generally AC overlays every 10 to 15 years).

2 Standard or "fixed" layer thicknesses include:
- ACP = 4.2" (105 mm)
- CS = 5.4" (135 mm)

3 Layer Coefficients
$a_{ACP,ACB}$ = 0.44 (Figure 2.5, AASHTO Guide)
a_{CS} = 0.13 (Figure 2.6, AASHTO Guide)

4 Frost design requirements must be checked if subgrade soil frost susceptible.

2.4.2 FLEXIBLE DESIGN CASE NO. 2

(a) Use the same conditions as stated for Case No. 1 (I-5) <u>except</u> use an asphalt treated permeable base (ATPB) beneath the ACP/ACB layers. Assume that the underlying CS will serve as an adequate separator layer between the ATPB and subgrade soil. Further, use a "fixed" or standard thickness of ACP (4.2" or 105 mm), ATPB (4.0" or 100 mm) and CS (5.4" or 135 mm).

(b) Summary of Results[1, 2] (only for S_0 = 0.50)

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
21,000,000 (20 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	3.9" (100 mm)	4.6" (120 mm)	6.0" (150 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.79	5.10	5.70

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
35,000,000 (30 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	4.8" (120 mm)	5.5" (140 mm)	6.9" (175 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	5.17	5.49	6.11
52,000,000 (40 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	5.5" (140 mm)	6.2 " (160 mm)	7.7" (195 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	5.47	5.80	6.44

1 Note: Layer thicknesses do not include periodic resurfacing (generally AC overlays every 10 to 15 years).

2 Layer Coefficients
$a_{ACP,ACB}$ = 0.44
a_{ATPB} = 0.13 (refer to Section 8.0, Paragraph 3.2)
a_{CS} = 0.13

2.4.3 FLEXIBLE DESIGN CASE NO. 3

Design new flexible pavements (various conditions) for I-90 (Spokane — MP 278 to 279) for the following conditions (three lanes each direction):

(a) Pavement location: urban

(b) Pavement functional classification: Interstate

(c) Design period: 20, 30 and 40 years

(d) Construction Materials

(i) Asphalt Concrete Pavement (Class A): 600,000 psi @ 68 °F

(ii) Asphalt Concrete Base (Class E): 600,000 psi @ 68 °F

(iii) Base (CSTC): 28,000 psi

(iv) Subbase: none used

(v) Subgrade: 20,000 psi (dry months)
15,000 psi (wet months)

(e) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(f) Reliability: use two levels R = 90%
R = 95%
R = 99%

(g) Overall standard deviation (S_o): use two levels 0.40
0.50

(h) Traffic (refer to Section 2.0, Paragraph 1.6 for background information on definitions of terms and ESALs per vehicle)

Singles: 956/day
Doubles: 1302/day
Trains: 366/day

(i) Calculations and Results

(i) ESALs per year
(Vehicles/day) (Lane Distribution Factor) (days/yr) (ESAL/Vehicle)

Singles:	(956) (0.8) (365) (0.40)	=	111,661 ESALs/yr
Doubles:	(1302) (0.8) (365) (1.00)	=	380,184 ESALs/yr
Trains:	(366) (0.8) (365) (1.75)	=	187,026 ESALs/yr
			678,871 ESALs/yr
		≈	680,000 ESALs/yr

(ii) ESALs for design periods
(use 2.4% annual growth rate)

20 years: $$680{,}000 \left[\frac{(1 + 0.024)^{20} - 1}{0.024}\right] \simeq 17{,}000{,}000$$

30 years: $$680{,}000 \left[\frac{(1 + 0.024)^{30} - 1}{0.024}\right] \simeq 29{,}000{,}000$$

40 years: $$680{,}000 \left[\frac{(1 + 0.024)^{40} - 1}{0.024}\right] \simeq 45{,}000{,}000$$

(iii) Assume 5 months wet and 7 months dry. Use Figure 5.19 and Paragraph 2.2.1.3 (d):

Month	M_R (psi)	U_f
Jan	15,000	0.024
Feb	15,000	0.024
Mar	15,000	0.024
Apr	15,000	0.024
May	20,000	0.012
Jun	20,000	0.012
Jul	20,000	0.012
Aug	20,000	0.012
Sep	20,000	0.012
Oct	20,000	0.012
Nov	20,000	0.012
Dec	15,000	0.024

$u_f = (1.18 \times 10^8)\,(15{,}000)^{-2.32} = 0.024$
$u_f = (1.18 \times 10^8)\,(20{,}000)^{-2.32} = 0.012$

$$\overline{u_f} = \frac{(5 \times 0.024) + (7 \times 0.012)}{12} = 0.017$$

$M_{Reff} = (3005)\,(0.017)^{-0.431} \cong 17{,}400$ psi

(iv) Summary of Results[1, 2, 3, 4] (only for $S_0 = 0.50$)

Design Period ESALs	Pavement Layer	Reliability		
		90% ($S_0 = 0.50$)	95% ($S_0 = 0.50$)	99% ($S_0 = 0.50$)
17,000,000 (20 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	3.8" (95 mm)	4.5" (115 mm)	5.8" (145 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.23	4.52	5.09
29,000,000 (30 years)	ACP	4.2" (105 mm)	4.2" (105) mm)	4.2" (105 mm)
	ACB	4.7" (120 mm)	5.4" (135 mm)	6.7" (170 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.60	4.91	5.49

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
45,000,000 (40 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	5.4" (135 mm)	6.1" (155 mm)	7.5" (190 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.92	5.23	5.84

1 Note: Layer thicknesses do not include periodic resurfacing (generally AC overlays every 10 to 15 years).

2 Standard or "fixed" layer thicknesses include:
- ACP = 4.2" (105 mm)
- CS = 5.4" (135 mm)

3 Total pavement structure must exceed 1/2 of frost depth if subgrade soil is frost susceptible. Assume this applies for this example. Design frost depth $\simeq$ 35" (Figure 2.34) or use Figure 2.36. Pavement structure >35/2 = 17.5" if Figure 2.34 used. Since additional thickness is required for all cases, use additional base or subbase.

4 Layer Coefficients
$a_{ACP,ACB}$ = 0.44 (Figure 2.5, AASHTO Guide)
a_{CS} = 0.13 (Figure 2.6, AASHTO Guide)

2.4.4 FLEXIBLE DESIGN CASE NO. 4

(a) Use the same conditions as stated for Case No. 3 (I-90) <u>except</u> use an ATPB beneath the ACP/ACB layers. Assume that the underlying CS will serve as an adequate separator layer between the ATPB and subgrade soil.

(b) Summary of Results[1, 2, 3, 4] (only for S_0 = 0.50)

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
17,000,000 (20 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	2.6" (65 mm)	3.3" (85 mm)	4.6" (115 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.23	4.52	5.09

Design Period ESALs	Pavement Layer	Reliability		
		90% (S_0 = 0.50)	95% (S_0 = 0.50)	99% (S_0 = 0.50)
29,000,000 (30 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	3.5" (90 mm)	4.2" (105 mm)	5.5" (140 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.60	4.91	5.49
45,000,000 (40 years)	ACP	4.2" (105 mm)	4.2" (105 mm)	4.2" (105 mm)
	ACB	4.2" (105 mm)	4.9" (125 mm)	6.3" (160 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)	5.4" (135 mm)
	Structural Number	4.92	5.23	5.84

1 Note: Layer thicknesses do not include periodic resurfacing (generally AC overlays every 10 to 15 years).

2 Standard or "fixed" layer thicknesses include:
- ACP = 4.2" (105 mm)
- ATPB = 4.0" (100 mm)
- CS = 5.4" (135 mm)

3 Total pavement structure must exceed 1/2 of frost depth if subgrade soil is frost susceptible. Assume this applies for this example. Design frost depth $\simeq$ 35" (Figure 2.34) or use Figure 2.36. Pavement structure >35/2 = 17.5" if Figure 2.34 used. If additional thickness required, use additional base or subbase.

4 Layer Coefficients

$a_{ACP,ACB} = 0.44$

$a_{ATPB} = 0.13$

$a_{CS} = 0.13$

SECTION 5.0

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SECTION 5.0, APPENDIX 5.1

THE AASHTO RELIABILITY CONCEPT

1. INTRODUCTION

The AASHTO reliability concept is based on a normal distribution of errors (more on this later). Thus, the normal distribution will be first explained followed by the specific definitions and concepts related to the AASHTO reliability scheme.

2. NORMAL DISTRIBUTION

The normal distribution is a data distribution that can be used to describe many types of measurements in engineering. Basically, a normal distribution is a bell shaped curve. The role of the normal distribution in statistics has been stated to be analogous to the role of the straight line in geometry. Figure 1 illustrates a bell curve, superimposed over a histogram of PCC compressive strength data. Such a distribution is very convenient to use because it is characterized by the mean (μ or $\bar{x}$) and standard deviation (σ or s). As Figure 1 shows, most of the strength measurements cluster around the mean ($\bar{x}$ = 4,824 psi), while fewer measurements are near the lowest (3,875 psi) and highest (5,975 psi) strength values.

The theoretical NORMAL DISTRIBUTION extends out infinitely in both directions and never quite reaches the horizontal axis and has a total area under the curve of 1.00 (i.e. 100 percent of the data values are represented by the distribution). Since it extends from - to + (minus infinity to plus infinity), it encompasses <u>all</u> of the results that can occur. The area under the curve within these two limits must therefore be equal to unity (i.e. 1.000 or 100 percent). For practical purposes, however, most of the data values (actually 99.73 percent) occur between 3 σ limits <u>below</u> the mean and 3 σ limits <u>above</u> the mean.

A far more important result than those mentioned above is also related to the fact that the area under the curve is equal to 100 percent. Because of this, it can be stated that the probability of finding a data value between any two values of x is equal to the area under the normal distribution between those values.

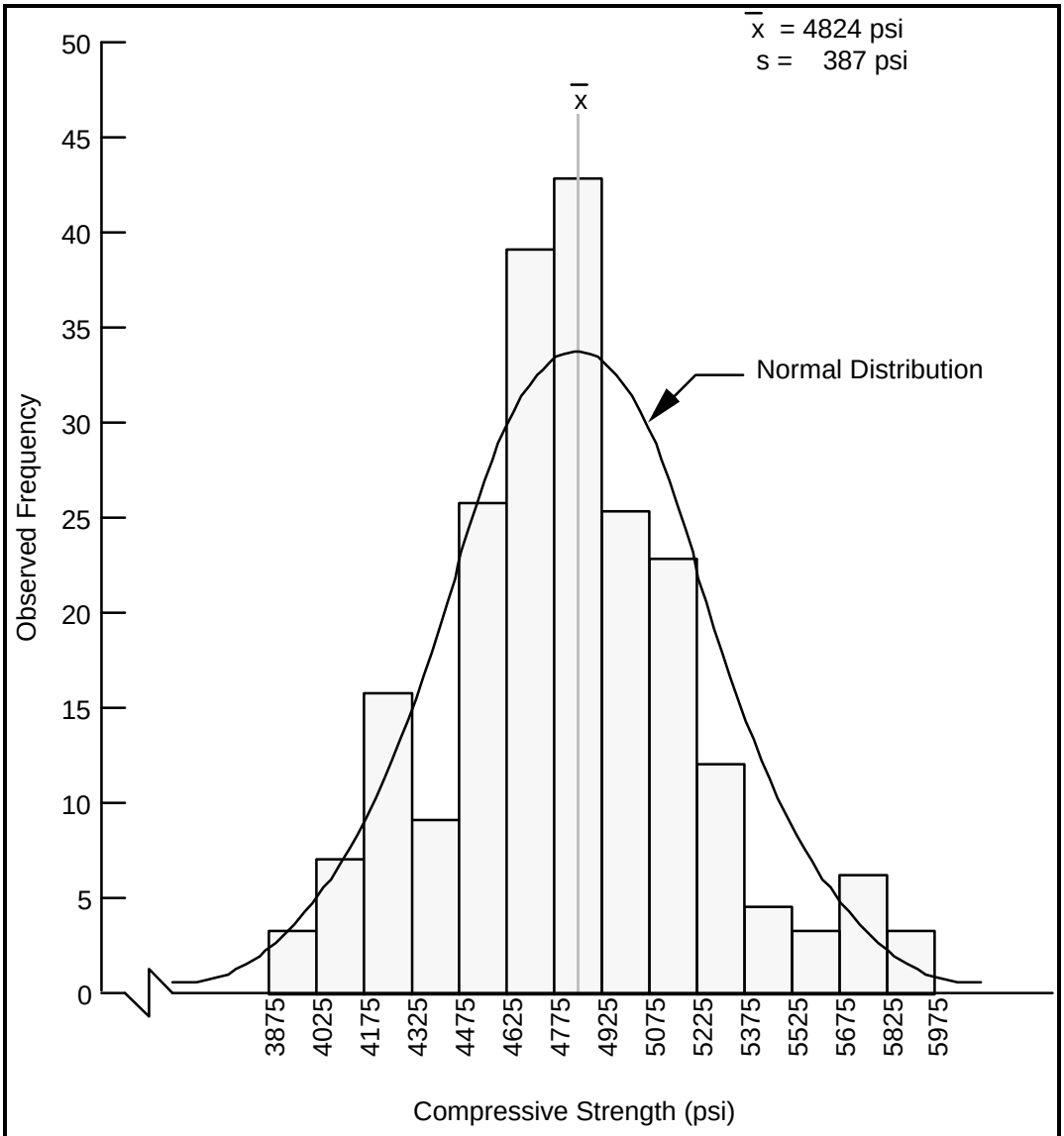


Figure 1. Histogram and the Normal Distribution for PCC Compressive Strength Data [after Ref. 2.1]

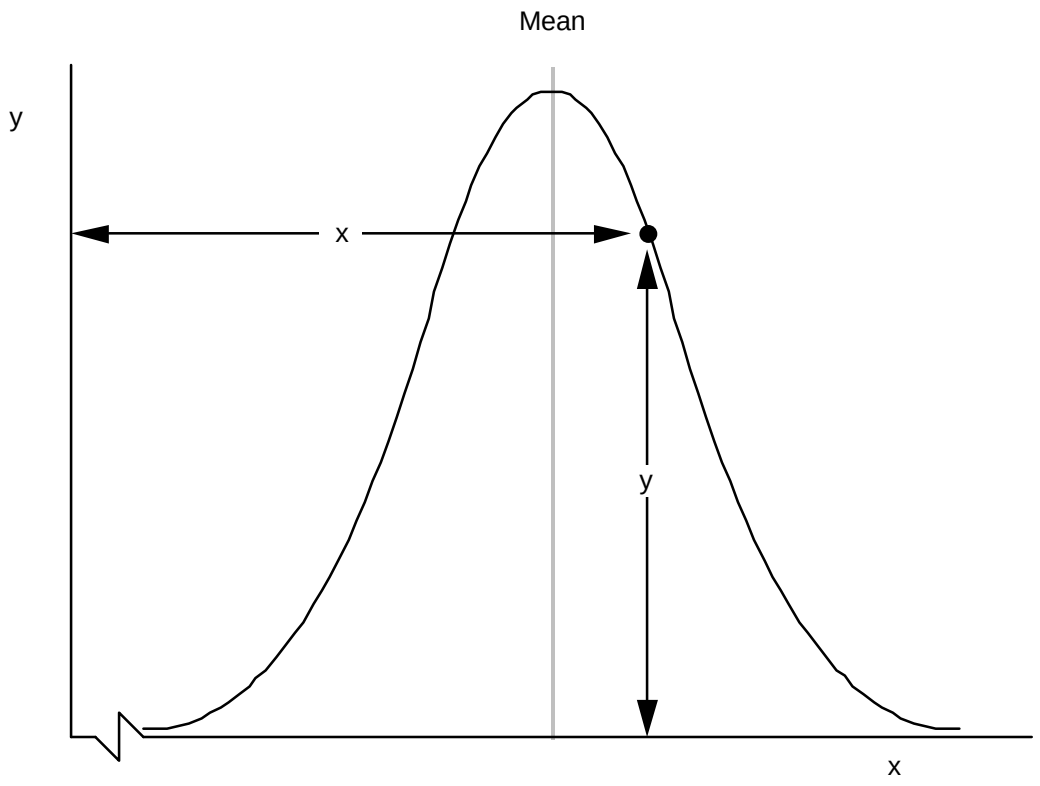


Figure 2. Relationship of "y" and "x" Values in the Normal Distribution

2.1 NORMAL DISTRIBUTION EQUATION

The height of a normal distribution (y) can be defined by its corresponding value of x (refer to Figure 2) by the following equation [after Ref. 1]:

$$y = \frac{1}{\sigma\sqrt{2\pi}} \quad e^{-(x - \mu)^2/2\sigma^2} \qquad \text{Eq. 1}$$

where y = vertical height of a point on the normal distribution,
x = distance along the horizontal axis,
σ = standard deviation of the data distribution,
μ = mean of the data distribution,
e = constant = 2.71828 ... ,
π = constant = 3.14159 ....

To illustrate how Equation 1 can be used to determine area under a normal distribution, we can use the Procter density data. Calculate the area under the normal curve between 105 and 115 pcf for a standard deviation of 5 pcf. These calculations are

$$y = \frac{1}{(5)\sqrt{2\pi}} \quad e^{-(105 - 105)^2 / 2(5)^2} = 0.079788$$

and

$$y = \frac{1}{(5)\sqrt{2\pi}} \quad e^{-(115 - 105)^2 / 2(5)^2} = 0.010798$$

The approximate area under the curve is about 0.45 (or 45 percent), which is close to the "theoretical" value of 48 percent (refer to sketch in Figure 3). The significance of this value is that the <u>probability</u> of a density measurement falling within the range of 105 to 115 pcf is about 0.48 (let's use the "theoretical" value).

To determine such probabilities in this manner is tedious and time consuming. There is an easier way to determine these probabilities than computing and tabulating y's for various μ's and σ's. To do this, you must convert the <u>normal distribution</u> to a <u>standard normal distribution</u> and define a variable "z," which is:

$$z = \frac{\text{deviation from mean}}{\text{standard deviation}} = \frac{x - \mu}{\sigma} \qquad \text{Eq. 2}$$

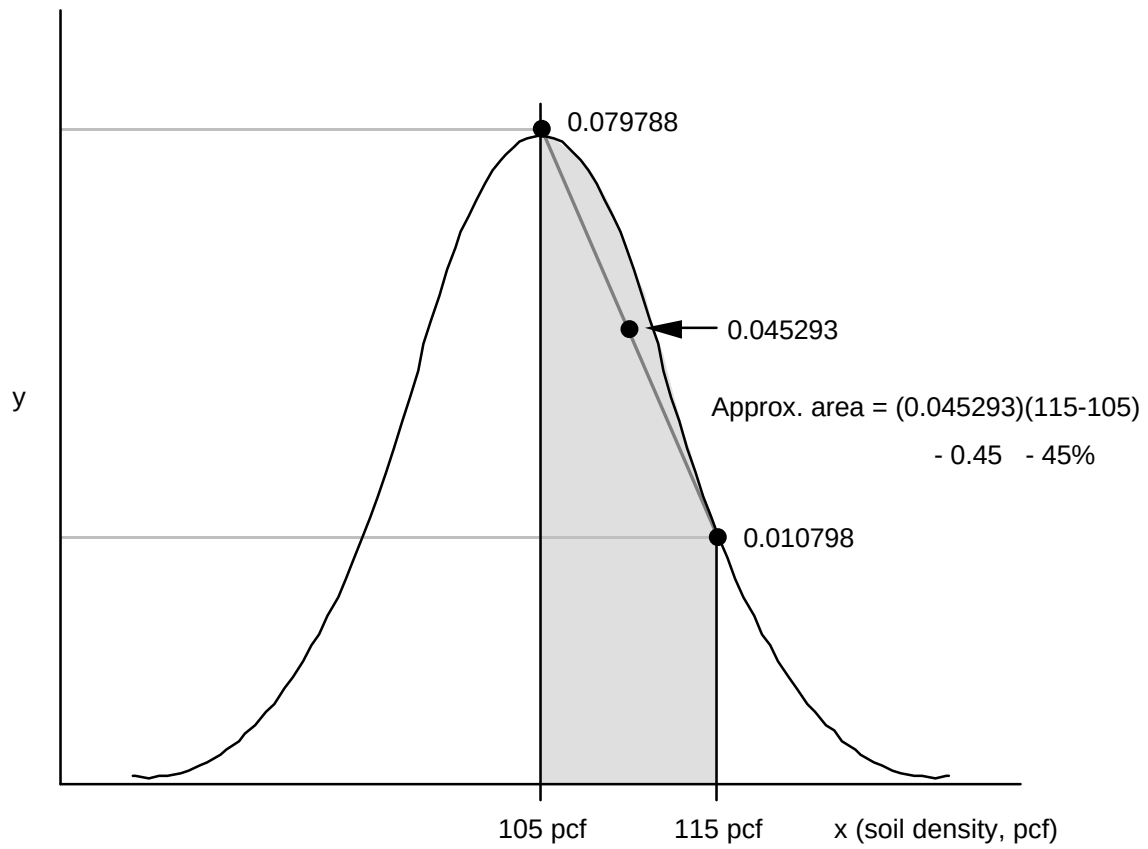


Figure 3. Determination of Approximate Area
Under the Normal Distribution

If you substitute z into Equation 1, the following relationship results:

$$y_z = \frac{1}{\sqrt{2\pi}} \; e^{-z^2/2} \qquad \text{Eq. 3}$$

where y_z = vertical height on the standard normal distribution, and
z = as previously defined.

Refer to Figure 4, which illustrates this important transformation. Thus, you can see that the probability of having a density test between 105 and 115 pcf is about 47.7 percent (or 34.1 + 13.6 percent). Fortunately, the "z-statistic" has been published in tables to allow for easy computation. Such a table is shown as Table 1. You can see that

mean ± 1 standard deviations ≅ 68.2% of area
mean ± 2 standard deviations ≅ 95.4% of area
mean ± 3 standard deviations ≅ 99.8% of area

Recall that all of the area under a normal distribution is 100% and the values 1, 2, 3 above correspond to the z-statistic.

3. RELIABILITY

3.1 AASHTO DEFINITION

> "The reliability of the pavement design-performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period." [from Ref. 3, p. I-53]

(Note: design period is defined as the time from initial construction or rehabilitation to its terminal serviceability index.)

3.2 FORMULA

$$R(\%) = (100)\ \text{Prob}\ (N_t \geq N_T)$$

where
N = actual ESALs
W = predicted ESALs
N_T = actual design period ESALs
N_t = actual ESALs to p_t
w_T = predicted design period ESALs
W_t = predicted ESALs to p_t

[refer to figure 5 [from Ref. 3 (Figure 4.2, p. I-57)]

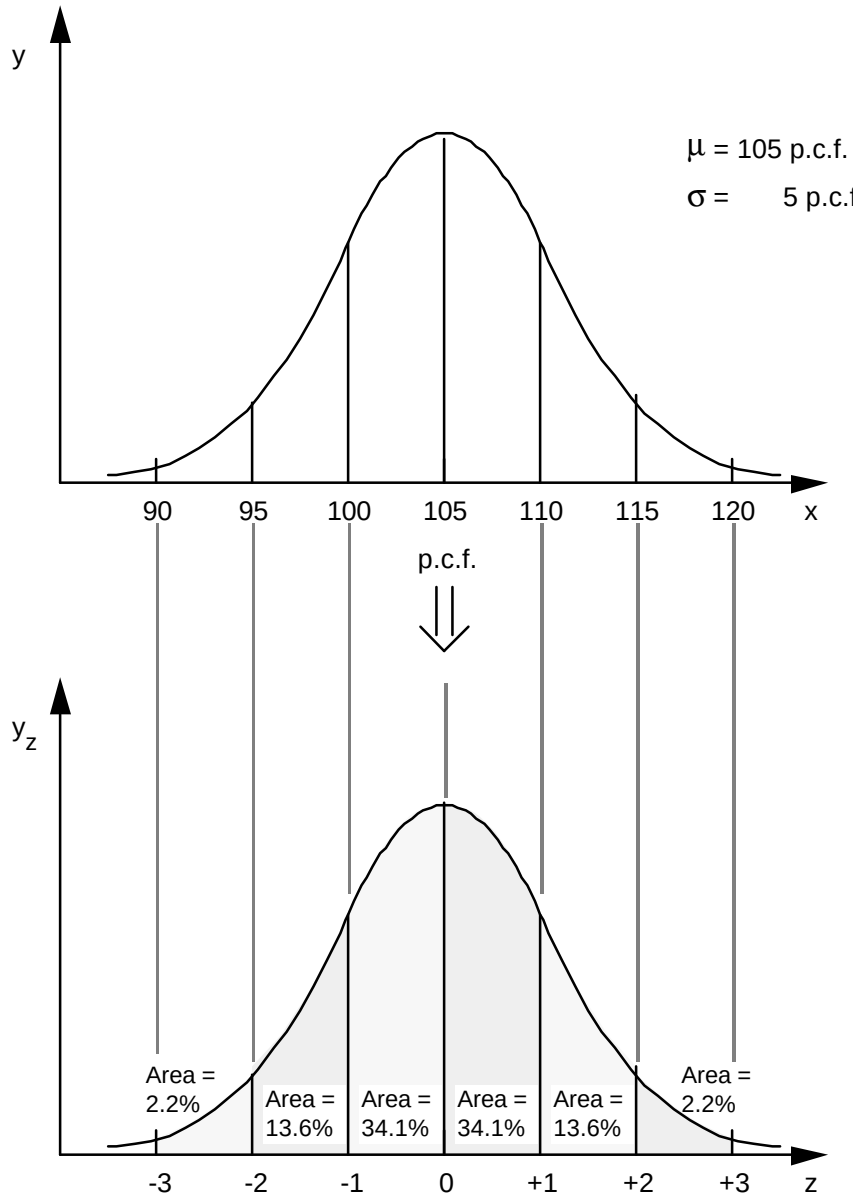
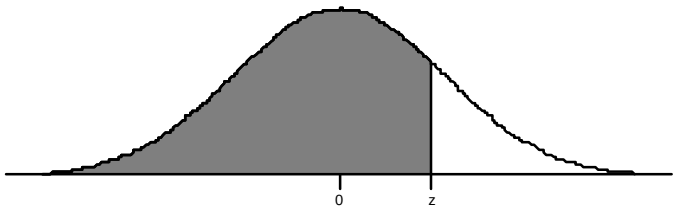


Figure 4. The Proctor Density Distributions
(Normal and Standard Normal)

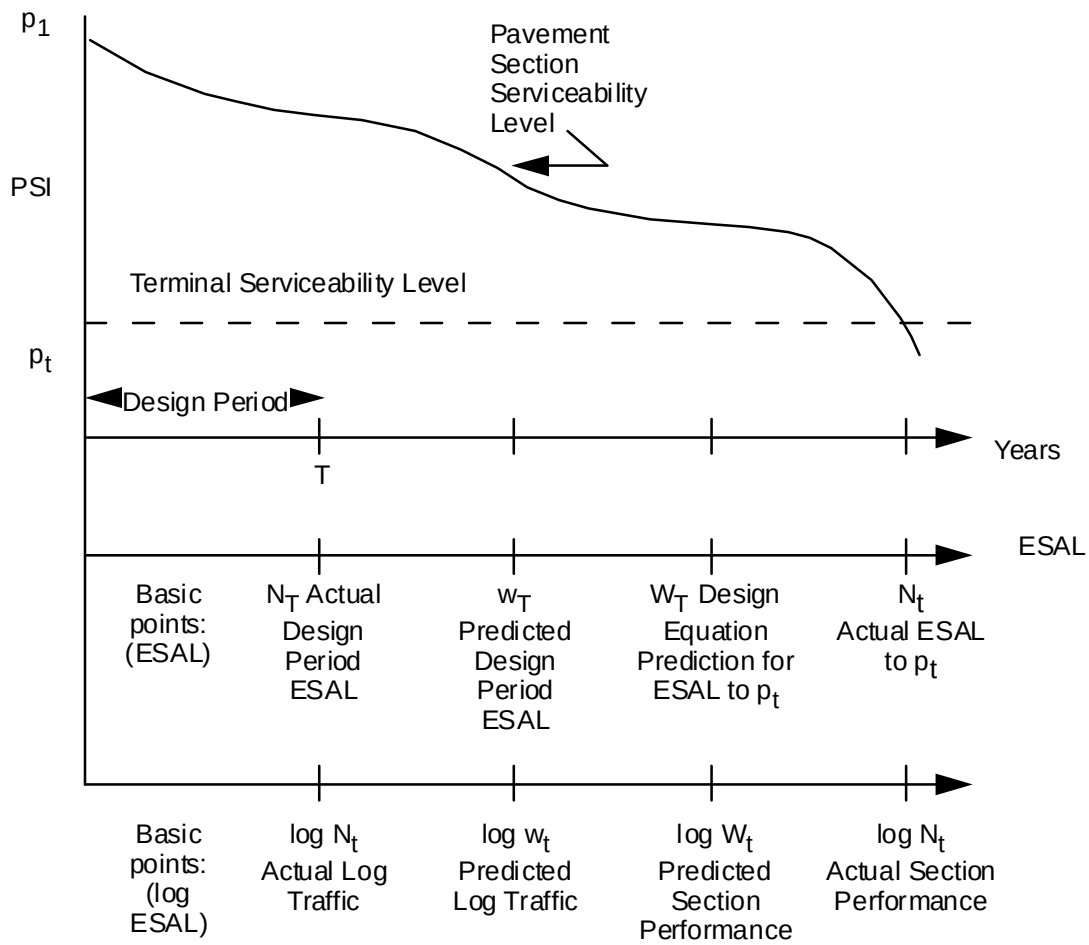
Table 1. Normal Distribution Table [from Ref. 2]

Normal Distribution



Normal Deviate z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
-4.0	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
-3.9	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
-3.8	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
-3.7	.0001	.0001	.0000	.0000	.0000	.0000	.0000	.0000	.0000	.0000
-3.6	.0002	.0002	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
-3.5	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
-3.4	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0002
-3.3	.0005	.0005	.0005	.0004	.0004	.0004	.0004	.0004	.0004	.0003
-3.2	.0007	.0007	.0006	.0006	.0006	.0006	.0006	.0005	.0005	.0005
-3.1	.0010	.0009	.0009	.0009	.0008	.0008	.0008	.0008	.0007	.0007
-3.0	.0013	.0013	.0013	.0012	.0012	.0011	.0011	.0011	.0010	.0010
-2.9	.0019	.0018	.0018	.0017	.0016	.0016	.0015	.0015	.0014	.0014
-2.8	.0026	.0025	.0024	.0023	.0023	.0022	.0021	.0021	.0020	.0019
-2.7	.0035	.0034	.0033	.0032	.0031	.0030	.0029	.0028	.0027	.0026
-2.6	.0047	.0045	.0044	.0043	.0041	.0040	.0039	.0038	.0037	.0036
-2.5	.0062	.0060	.0059	.0057	.0055	.0054	.0052	.0051	.0049	.0048
-2.4	.0082	.0080	.0078	.0075	.0073	.0071	.0069	.0068	.0066	.0064
-2.3	.0107	.0104	.0102	.0099	.0096	.0094	.0091	.0089	.0087	.0084
-2.2	.0139	.0136	.0132	.0129	.0125	.0122	.0119	.0116	.0113	.0110
-2.1	.0179	.0174	.0170	.0166	.0162	.0158	.0154	.0150	.0146	.0143
-2.0	.0228	.0222	.0217	.0212	.0207	.0202	.0197	.0192	.0188	.0183
-1.9	.0287	.0281	.0274	.0268	.0262	.0256	.0250	.0244	.0239	.0233
-1.8	.0359	.0351	.0344	.0336	.0329	.0322	.0314	.0307	.0301	.0294
-1.7	.0446	.0436	.0427	.0418	.0409	.0401	.0392	.0384	.0375	.0367
-1.6	.0548	.0537	.0526	.0516	.0505	.0495	.0485	.0475	.0465	.0455
-1.5	.0668	.0655	.0643	.0630	.0618	.0606	.0594	.0582	.0571	.0559
-1.4	.0808	.0793	.0778	.0764	.0749	.0735	.0721	.0708	.0694	.0681
-1.3	.0968	.0951	.0934	.0918	.0901	.0885	.0869	.0853	.0838	.0823
-1.2	.1151	.1131	.1112	.1093	.1075	.1056	.1038	.1020	.1003	.0985
-1.1	.1357	.1335	.1314	.1292	.1271	.1251	.1230	.1210	.1190	.1170
-1.0	.1587	.1562	.1539	.1515	.1492	.1469	.1446	.1423	.1401	.1379
-.9	.1841	.1814	.1788	.1762	.1736	.1711	.1685	.1660	.1635	.1611
-.8	.2119	.2090	.2061	.2033	.2005	.1977	.1949	.1922	.1894	.1867
-.7	.2420	.2389	.2358	.2327	.2296	.2266	.2236	.2206	.2177	.2148
-.6	.2743	.2709	.2676	.2643	.2611	.2578	.2546	.2514	.2483	.2451
-.5	.3085	.3050	.3015	.2981	.2946	.2912	.2877	.2843	.2810	.2776
-.4	.3446	.3409	.3372	.3336	.3300	.3264	.3228	.3192	.3156	.3121
-.3	.3821	.3783	.3745	.3707	.3669	.3632	.3594	.3557	.3520	.3483

Normal Deviate z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
-.2	.4207	.4168	.4129	.4090	.4052	.4013	.3974	.3936	.3897	.3859
-.1	.4602	.4562	.4522	.4483	.4443	.4404	.4364	.4325	.4286	.4247
-.0	.5000	.4960	.4920	.4880	.4840	.4801	.4761	.4721	.4681	.4641
0.0	.5000	.5040	.5080	.5120	.5160	.5199	.5239	.5279	.5319	.5359
0.1	.5398	.5438	.5478	.5517	.5557	.5596	.5636	.5675	.5714	.5753
0.2	.5793	.5832	.5871	.5910	.5948	.5987	.6026	.6064	.6103	.6141
0.3	.6179	.6217	.6255	.6293	.6331	.6368	.6406	.6443	.6480	.6517
0.4	.6554	.6591	.6628	.6664	.6700	.6736	.6772	.6808	.6844	.6879
0.5	.6915	.6950	.6985	.7019	.7054	.7088	.7123	.7157	.7190	.7224
0.6	.7257	.7291	.7324	.7357	.7389	.7422	.7454	.7486	.7517	.7549
0.7	.7580	.7611	.7642	.7673	.7703	.7734	.7764	.7793	.7823	.7852
0.8	.7881	.7910	.7939	.7967	.7995	.8023	.8051	.8078	.8106	.8133
0.9	.8159	.8186	.8212	.8238	.8264	.8289	.8315	.8340	.8365	.8389
1.0	.8413	.8438	.8461	.8485	.8508	.8531	.8554	.8577	.8599	.8621
1.1	.8643	.8665	.8686	.8708	.8729	.8749	.8770	.8790	.8810	.8830
1.2	.8849	.8869	.8888	.8907	.8925	.8944	.8962	.8980	.8997	.9015
1.3	.9032	.9049	.9066	.9082	.9099	.9115	.9131	.9147	.9162	.9177
1.4	.9192	.9207	.9222	.9236	.9251	.9265	.9279	.9292	.9306	.9319
1.5	.9332	.9345	.9357	.9370	.9382	.9394	.9406	.9418	.9429	.9441
1.6	.9452	.9463	.9474	.9484	.9495	.9505	.9515	.9525	.9535	.9545
1.7	.9554	.9564	.9573	.9582	.9591	.9599	.9608	.9616	.9625	.9633
1.8	.9641	.9649	.9656	.9664	.9671	.9678	.9686	.9693	.9699	.9706
1.9	.9713	.9719	.9726	.9732	.9738	.9744	.9750	.9756	.9761	.9767
2.0	.9772	.9778	.9783	.9788	.9793	.9798	.9803	.9808	.9812	.9817
2.1	.9821	.9826	.9830	.9834	.9838	.9842	.9846	.9850	.9854	.9857
2.2	.9861	.9864	.9868	.9871	.9875	.9878	.9881	.9884	.9887	.9890
2.3	.9893	.9896	.9898	.9901	.9904	.9906	.9909	.9911	.9913	.9916
2.4	.9918	.9920	.9922	.9925	.9927	.9929	.9931	.9932	.9934	.9936
2.5	.9938	.9940	.9941	.9943	.9945	.9946	.9948	.9949	.9951	.9952
2.6	.9953	.9955	.9956	.9957	.9959	.9960	.9961	.9962	.9963	.9964
2.7	.9965	.9966	.9967	.9968	.9969	.9970	.9971	.9972	.9973	.9974
2.8	.9974	.9975	.9976	.9977	.9977	.9978	.9979	.9979	.9980	.9981
2.9	.9981	.9982	.9982	.9983	.9984	.9984	.9985	.9985	.9986	.9986
3.0	.9987	.9987	.9987	.9988	.9988	.9989	.9989	.9989	.9990	.9990
3.1	.9990	.9991	.9991	.9991	.9992	.9992	.9992	.9992	.9993	.9993
3.2	.9993	.9993	.9994	.9994	.9994	.9994	.9994	.9995	.9995	.9995
3.3	.9995	.9995	.9995	.9996	.9996	.9996	.9996	.9996	.9996	.9997
3.4	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9998
3.5	.9998	.9998	.9998	.9998	.9998	.9998	.9998	.9998	.9998	.9998
3.6	.9998	.9998	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.7	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.8	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.9	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
4.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000



Basic deviations: (Level 1)	$\pm\delta(N_T w_T)$ Prediction error in design period traffic (log)	log F_R Reliability design factor (log)	$\pm\delta(W_t N_t)$ Prediction error in section performance
Basic deviation: (Level 0)	$\pm\delta_0 = \pm\delta(N_T w_T) + \text{Log } F_R \pm \delta(W_t N_t)$ Overall deviation of actual section performance from actual design period traffic $\delta_0 = (\log N_t - \log N_T)$ $\pm\delta_0$ means section survival of design period ESAL		

Figure 5. Basic Points and Deviations for Design-Performance Reliability [after Ref.3]

In general, reliability is the probability of success.

$$R(\text{traffic}) = 1 - \underbrace{F(\text{traffic})}_{\text{Distribution representing failure}}$$

= Prob (actual traffic to failure (N_t) actual design period traffic (N_T))

3.3 THE BASIS

Let δ represent error (refer to Figure 5). What kinds of errors do we have?

3.3.1 THE BASIC DEVIATIONS

(a) Prediction error in the design period traffic

$$(\log w_T - \log N_T) = \pm\, \delta\, (N_T, w_T)$$

Note: log transformations used to ensure normality in the error distributions.

(b) Reliability design factor

$$(\log W_t - \log w_T) = \log F_R$$

where $\log F_R$ is a "spacing factor" between $\log w_T$ (predicted design period ESALs) and $\log W_t$ (predicted ESALs to p_t).

(c) Prediction error in pavement section performance

$$(\log N_t - \log W_t) = \pm\, \delta\, (W_t, N_t)$$

(d) Overall (designated "o" hereafter) deviation of actual section performance from actual design period traffic.

$$(\log N_t - \log N_T) = \pm\, \delta_0$$

3.3.2 THE BASIC PROBABILITY DISTRIBUTIONS

The basic probability distributions are shown in Figure 6 [from Ref. 3 (Figure 4.3, p. I-59)]. The following discussion will use both Figure 6 and Figure 7 [from Ref. 3 (Figure 4.4, p. I-61)].

In the upper portion of Figure 6, the distribution of $\delta\ (N_T,\ w_T)$ is the distribution of <u>errors</u> for the <u>design period</u> traffic (differences between <u>actual</u> and <u>predicted</u> design period ESALs).

Also in the upper portion of this figure is the distribution of $\delta\ (W_t,\ N_t)$. This distribution is for the <u>errors</u> for predictions of pavement performance (differences between <u>actual</u> and <u>predicted</u> ESALs to $\underline{p_t}$).

In the lower portion of Figure 6, the distribution of all δ_0's represents all possible combinations of $\delta\ (N_T,\ w_T)$ and $\delta\ (W_t,\ N_t)$. Since

$$\pm\ \delta_0 = \underbrace{\pm\ \delta\ (N_T,\ w_T)}_{\text{chance deviation}} + \underbrace{\log F_R}_{\text{fixed deviation}} + \underbrace{\pm\ \delta\ (W_t,\ N_t)}_{\text{chance deviation}}$$

the laws of probability require that δ_0 has a <u>normal</u> <u>distribution</u> whose <u>mean</u> = sum of the three deviate means and whose <u>variance</u> = sum of the three deviate variances. Thus,

$$\bar{\delta}_0 = \underbrace{\bar{\delta}\ (N_T,\ w_T)}_{0} + \underbrace{\log F_R}_{\log F_R} + \underbrace{\bar{\delta}\ (W_t,\ N_t)}_{0}$$

$$= 0 + \log F_R + 0 = \log F_r$$

and $$S_0^2 = S_w^2 + \underbrace{0}_{\text{zero since } \log F_R \text{ is } \underline{\text{fixed}} \text{ (i.e., no variation)}} + S_N^2$$

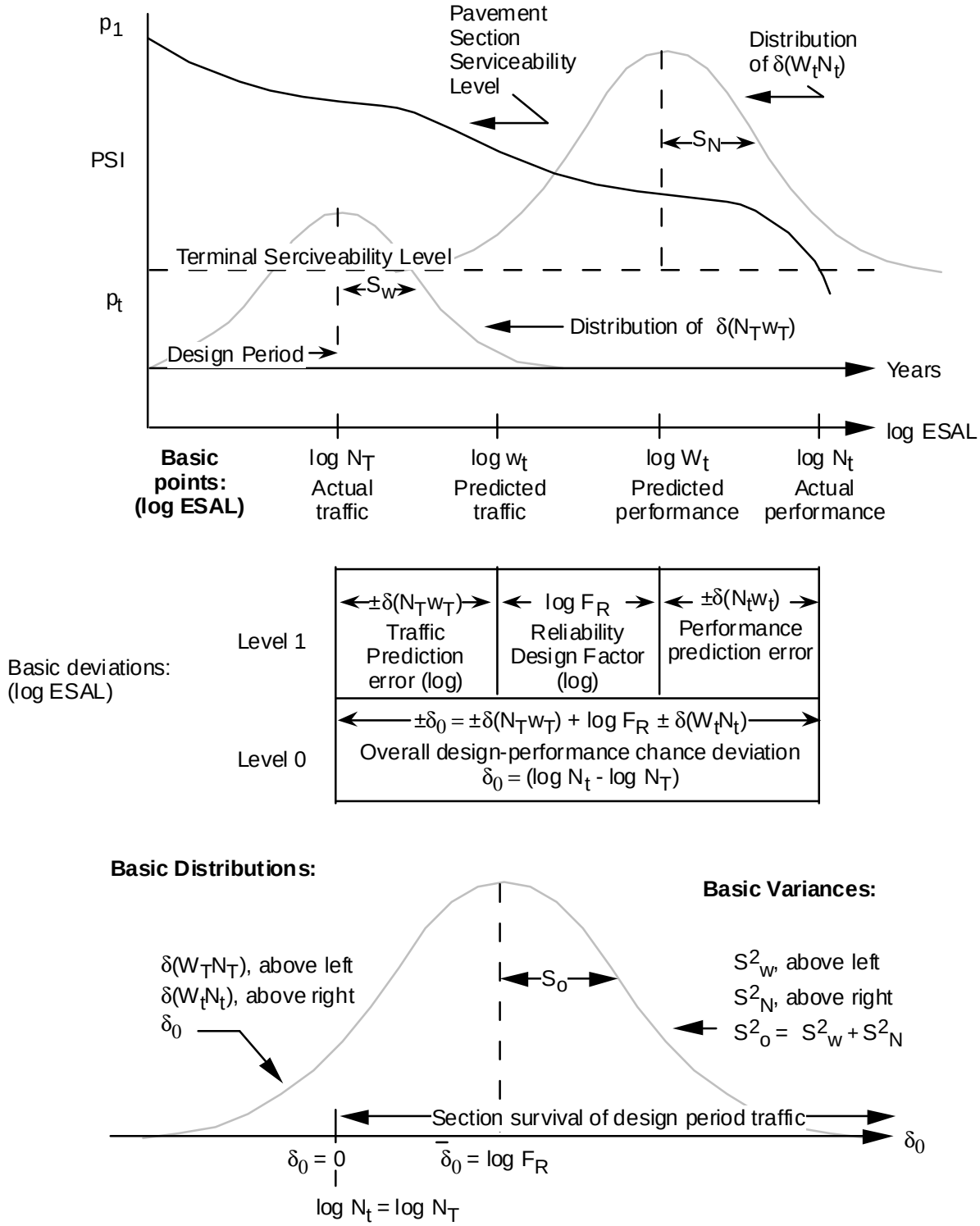
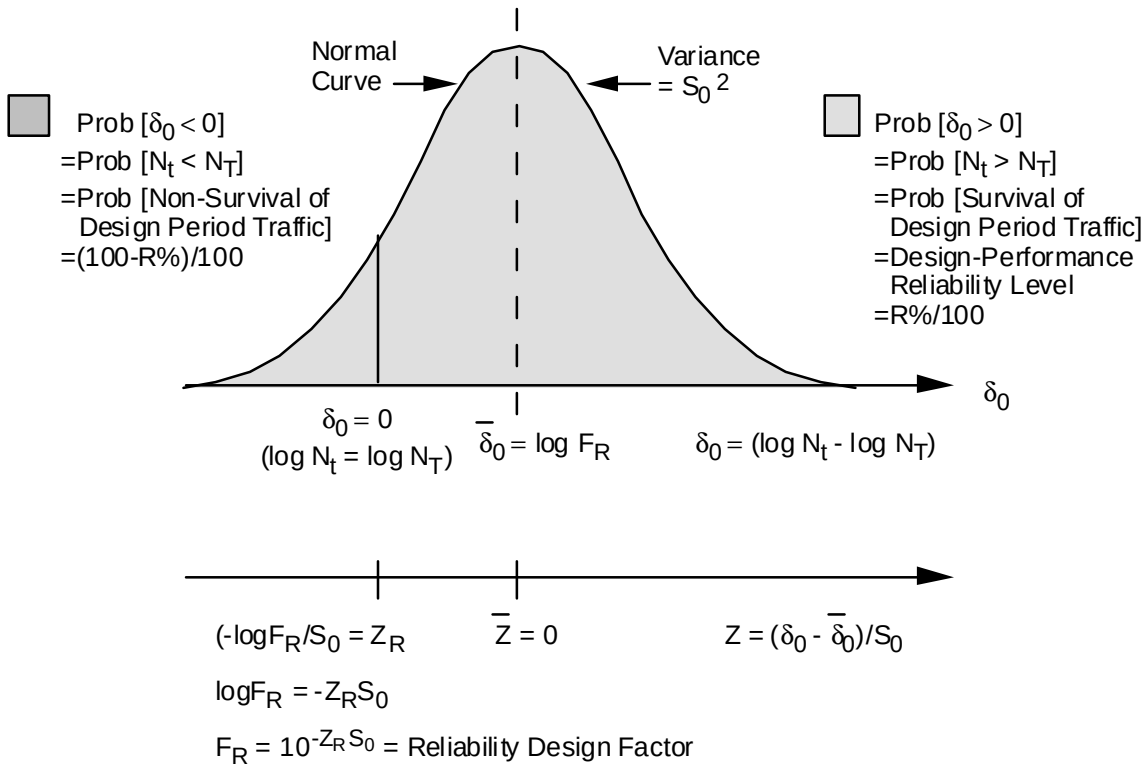


Figure 6. Basic Probability Distributions for Design-Performance Reliability [after Ref. 3}



Note 1. The value of Z_R is determined by the value of R, and is obtained from standard normal curve area tables by entering (100-R%)/100 for the tail area from $-\infty$ to Z_R.

Note 2. If $\log F_R = 0$, $Z_R = 0$. $F_R = 1$, and R = 50%. Thus the probability for design period survival is 50% if the traffic prediction (w_T) is substituted directly for W_t in the performance prediction (design) equation.

Note 3. For fixed R (hence fixed Z_R), F_R increases (or decreases) as $S_0 = \sqrt{S^2_w + S^2_N}$ increases (or drecreases). F_R accounts for the total chance variation in traffic predictions and performance predictions.

Figure 7. Definition of Reliability and Evaluation of Reliability Design Factor [after Ref. 3]

Thus, the S_0 error distribution is a combination of the S_w and S_N error distributions.

Recall, the overall deviation of actual pavement section performance from actual design period traffic:

$$(\log N_t - \log N_T) = \delta_0$$

Transforming

$$\delta_0 = \log\left(\frac{N_t}{N_T}\right)$$

and we want $\frac{N_t}{N_T} \geq 1$

thus $\delta_0 = \log(1) = 0$

Therefore, we need to determine the point $\delta_0 = 0$ in terms of probability (z_R) and S_0.

Thus, let z_R (at $\delta_0 = 0$) = $\frac{\delta_0 - \bar{\delta}_0}{S_0}$ and refer to Figure 7. (Recall $z = \frac{x - \mu}{\sigma}$ which was shown as Equation 2.)

(Again, $\delta_0 = 0$ is the important point to determine in that $\log N_t = \log N_T$ (actual performance = actual traffic).)

Therefore, $z_R = \frac{-\bar{\delta}_0}{S_0}$ since $\delta_0 = 0$.

Further, $\bar{\delta}_0 = \log F_R$

so $z_R = \frac{-\log F_R}{S_0}$

Solving for F_R: $F_R = 10^{-z_R(S_0)}$

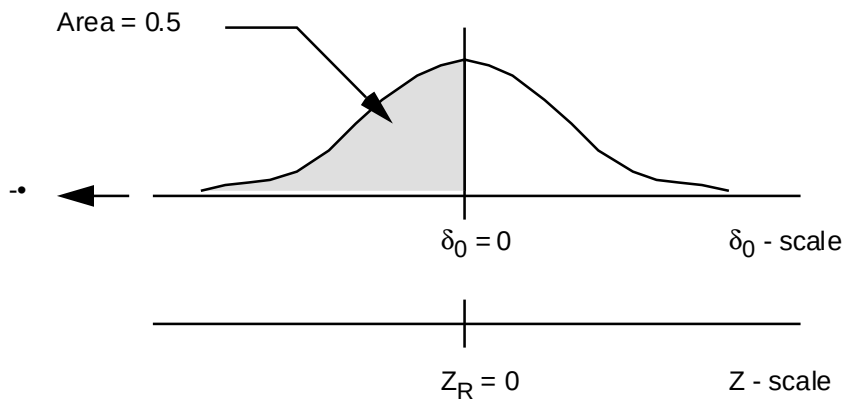
3.3.3 EXAMPLES

3.3.3.1 R = 50%

If we seek a reliability = 50%

$$R = \text{Prob}(N_t \geq N_T) = 1 - (\text{area from } - \text{ to } \delta_0 = 0) = 0.5$$
$$= 1 - (\text{area} = 0.5, \text{ thus } z_R = 0) = 0.5$$

Thus, $F_R = 10^{-z_R(S_0)} = 10^{-0(S_0)} = 1$



Thus, the ESALs used to determine SN or slab thickness = ESALs used to <u>predict</u> the design period traffic.

3.3.3.2 R = 99%

Now, if we seek a reliability = 99%

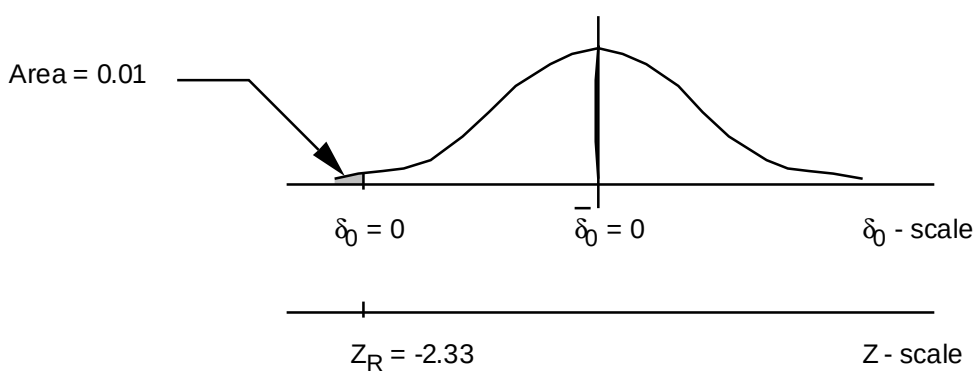
$$R = \text{Prob}\,(N_t \geq N_T) = 1 - (\text{area from } - \text{ to } \delta_0 = 0) = 0.99$$
$$= 1 - (\text{area} = 0.01, \text{ thus } z_R = -2.33) = 0.99$$

(Note: z_R obtained from Table 1.)

Thus, $F_R = 10^{-z_R(S_0)} = 10^{-(-2.33)(0.40)} = 8.55$ (Use $S_0 = 0.40$ for this example)

Therefore, $W_t = (w_T)(F_R)$

(Predicted ESALs by design equation to p_t = predicted design period ESALs x F_R) and ESALs used to <u>determine</u> SN or slab thickness are 8.55 times greater than expected.



If S_0 = 0.25, then F_R = 3.82
S_0 = 0.30, then F_R = 5.00
S_0 = 0.35, then F_R = 6.54
S_0 = 0.45, then F_R = 11.18
S_0 = 0.50, then F_R = 14.62

The above F_R values appear to be rather large but recall that these are for R = 99% (which is also quite large (or high)). If for example, R = 90%, and

S_0 = 0.25, then F_R = 2.09
S_0 = 0.30, then F_R = 2.42
S_0 = 0.35, then F_R = 2.81
S_0 = 0.40, then F_R = 3.25
S_0 = 0.45, then F_R = 3.77
S_0 = 0.50, then F_R = 4.37

(Table 1 reveals that z_R= 1.28 for R = 90%.)

3.4 ANOTHER WAY TO VIEW S_0

As stated previously, S_0 represents the standard deviation of the normal distribution of δ_0. One can view S_0 <u>approximately</u> as:

Estimate design period ESALs

$\pm 1S_0 \cong$ 68.2% of area under a normal distribution
$\pm 2S_0 \cong$ 95.4% of area under a normal distribution
$\pm 3S_0 \cong$ 99.8% of area under a normal distribution

(Recall: 1, 2, 3 = z-statistic.)

Thus, if the design period ESALs = 1,000,000 then for S_0 = 0.25

$10^6 \pm 1(0.25)$ = 562,000; 1,778,000
$10^6 \pm 2(0.25)$ = 316,000; 3,162,000
$10^6 \pm 3(0.25)$ = 178,000; 5,623,000

And if S_0 = 0.5, then

$10^6 \pm 1(0.50)$ = 316,000; 3,162,000
$10^6 \pm 2(0.50)$ = 100,000; 10,000,000
$10^6 \pm 3(0.50)$ = 31,600 31,623,000

3.5 SHA USE OF RELIABILITY

A 1988 survey of SHAs [4] revealed extensive use of the AASHTO reliability concept. Of the SHAs reporting, 28 were using (or planned to use) reliability for rehabilitation design and 33 were using (or planned to use) reliability for new construction. Of those SHAs using reliability, the following ranges were being used:

	Reliability	
Functional Class	Range	Mean
• Interstates		
• Urban	85 - 99%	94%
• Rural	70 - 99%	92%
• Principal Arterials		
• Urban	75 - 99%	88%
• Rural	70 - 95%	84%
• Collectors		
• Urban	50 - 95%	77%
• Rural	50 - 95%	74%
• Local		
• Urban	50 - 85%	70%
• Rural	50 - 85%	70%

Many SHAs use a range of reliability levels for a specific functional class of highway. It appears that "setting" a specific reliability level is often based on anticipated traffic — higher traffic, higher reliability level.

SECTION 5.0, APPENDIX 5.1

REFERENCES

1. Willenbrock, Jack H., "A Manual for Statistical Quality Control of Highway Construction — Volume I," Federal Highway Administration, National Highway Institute, Washington D.C., January 1976.

2. Ulberg, Cy, "Highway Research Manual," Draft Final Report, Research Project Y3399, Task 29, Washington State Department of Transportation, Olympia, WA, August 1987.

3. American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures*, AASHTO, Washington, D.C., 1986.

4. Federal Highway Administration, "Summary of the AASHTO Questionnaire on the *Guide for Design of Pavement Structures, 1986*," Office of Highway Operations, Federal Highway Administration, Washington, D.C., April 1, 1988.

SECTION 5.0, APPENDIX 5.2

EFFECTIVE ROADBED RESILIENT MODULUS

1. GENERAL

The process used to incorporate M_R into the Guide will be presented along with the rationale for seasonal weighting.

2. M_R IN PERFORMANCE EQUATION

2.1 LAYER ELASTIC THEORY

Used layered theory (as discussed in SECTION 4.0 of these notes)

- Calculated response of various pavement sections to 18,000 lb. single axle loads.
- Six surface thicknesses (3 to 10 inches).
- Variable base thickness
- Roadbed soils
  - Poor, CBR ≅ 2, M_R = 3,000 psi
  - Fair, CBR ≅ 5, M_R = 7,500 psi
  - Good, CBR ≅ 10, M_R = 15,000 psi
- Calculated maximum tensile strain bottom of asphalt concrete layer (associated with fatigue cracking in asphalt concrete).
- Calculated maximum compressive strain in roadbed soil (associated with rutting failure).

2.2 REGRESSION EQUATION

Developed from the above results the following regression equation

$$S_i = (6.24)(\log M_R) - 18.72$$

Typical values from this equation follow:

M_R (psi)	Soil Support Value
2,000	2
3,000	3
4,400	4
6,300	5
9,300	6
13,000	7
19,000	8
28,000	9
40,000	10

3. EFFECTIVE ROADBED RESILIENT MODULUS

Basic assumption is that Miner's linear damage hypothesis is valid so that the relative damage values can be added together to define the total overall damage as follows:

$$D = \sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1$$

where D = damage,
n_i = actual loads i experienced,
N_i = allowable loads i to failure, and
n = number of load cases considered.

The performance equation without the reliability term is split into two parts:

Let $T_i = (2.32)(\log M_{R_i}) - 8.07$ which represents the overall M_R effect on predicted performance.

$$\text{Let } Q = (9.36)(\log (SN + 1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}}$$

which represents the non-M_R effect on predicted performance. Further, the i subscript represents the ith time increment during a year. Now, substitute back into the original performance equation:

$$W_{18i} = (10^{Q})(10^{Ti})$$

(recall that if log W_{18} = x, then $W_{18} = 10^{x}$)

where 10^{Ti} represents the relative damage due to resilient modulus on pavement performance.

Now, assuming that Miner's damage hypothesis can be used:

$$\sum_{i=1}^{n} \frac{\left[\frac{W_{18}\ \text{total}}{n}\right]}{W_{18i}} = 1.0 \qquad \text{Eq. 1}$$

where $W_{18total}$ = total 18,000 lb. ESALs <u>predicted</u> during the analysis period,

W_{18i} = <u>allowable</u> 18,000 lb. ESALs to "failure" for ith time period, and

n = number of equal time periods through the year to identify individual seasons.

Now, substituting for $W_{18i} = 10^{Q}\ 10^{Ti}$ in Equation 1,

$$D = \sum_{i=1}^{n} \frac{W_{18total}}{n\ 10^{Q}\ 10^{Ti}} = 1.0$$

Now, define the "relative damage value" as u_{fi} and let it equal $\frac{1}{10^{Ti}}$ or 10^{-Ti} and factor out the constant terms from Equation 1.

$$D = \frac{W_{18total}}{n\ 10^{Q}} \sum u_{fi} = 1.0$$

and by rewriting,

$$W_{18total} = \frac{10^{Q}}{\frac{1}{n}\sum u_{fi}} \qquad \text{Eq. 2}$$

The term in the denominator of Equation 2 indicates that the overall effects of seasonal roadbed soil resilient modulus can be described in terms of <u>average</u> relative damage as follows:

$$\bar{u}_f = \frac{\sum_{i=1}^{n} u_{fi}}{n}$$

The resilient modulus for each "season" of the year is converted to a relative damage value (u_{fi}) and then averaged ($\bar{u}_f$) for pavement design purposes. The equation needed to convert soil resilient moduli to u_{fi} values can be derived as follows:

Starting with $u_f = 10^{-Ti}$ (as previously shown),

then $u_{f_i} = 10^{-Ti} = 10^{(-2.32 \log M_R + 8.07)}$

$= (M_R^{-2.32})(10^{8.07})$

$= (1.17 \times 10^8)(M_R^{-2.32})$ Eq. 3

However, after the u_{f_i} values have been obtained, then the $\bar{u}_f$ can be calculated. In turn, the $\bar{u}_f$ value is converted back to resilient modulus by the following equation:

$$(M_R)_{eff} = (3005)(\bar{u}_f)^{-0.431} \qquad \text{Eq. 4}$$

Equation 4 can be derived as follows:

$$\bar{u}_f = (1.17 \times 10^8)(M_R^{-2.32}) \qquad \text{from Equation 3}$$

Converting to logs:

$$\log u_f = \log (1.17 \times 10^8) - 2.32 \log M_R$$

Thus, $\log M_R = \dfrac{\log u_f + 8.07}{2.32}$

and $M_R = (10^{3.478})(u_f^{-1/2.32}) = (3005)(u_f)^{-0.431}$

or $(M_R)_{eff} = (3005)(\bar{u}_f)^{-0.431}$

The chart and example calculations used in the *AASHTO Guide* for estimating effective roadbed resilient modulus is illustrated in Figure 5.19.

SECTION 6.0

THE AASHTO RIGID PAVEMENT DESIGN PROCEDURE (NEW OR RECONSTRUCTED)

This SECTION is used to describe the AASHTO rigid pavement design procedure (which has been adopted by WSDOT). Information is provided relative to the development of this procedure, as are various design examples. The required design input values are described and illustrated. U.S. customary units are used throughout this SECTION since the AASHTO design procedure currently uses these units.

1. BACKGROUND

1.1 AASHO ROAD TEST

The basic information concerning the AASHO Road Test presented in SECTION 5.0 will not be repeated here; however, relevant information specific to how the rigid pavements performed at the Road Test will be. It is helpful in one's understanding of the AASHTO Design Guide to have such background information.

1.2 AASHTO DESIGN CRITERIA

The primary design criterion is Present Serviceability Index (PSI) (as defined in SECTION 5.0, Paragraph 1.5.1). PSI was estimated for the Road Test pavement sections by use of the following equation [6.1]:

$$PSI = 5.41 - 1.80 \log(1 + \overline{SV}) - 0.9\sqrt{C + P} \qquad \text{Eq. 6.1}$$

where PSI = Present serviceability index,

$\overline{SV}$ = mean of the slope variance in the two wheelpaths (measured with the CHLOE profilometer or BPR Roughometer), and

C, P = measures of cracking and patching in the pavement surface,

C = total linear feet of Class 3 and Class 4 cracks per 1000 ft^2 of pavement area. A Class 3 crack is defined as opened or spalled (at the surface) to a width of 0.25 in. or more over a distance equal to at least one-half the crack length. A Class 4 is defined as any crack which has been sealed.

P = expressed in terms of ft^2 per 1000 ft^2 of pavement surfacing.

The summary statistics for the above equation are:

R^2 = 91.6 percent
RMSE = 0.32
n = 49 data points

The original data used to develop this equation is shown as Table 6.1 and is a good illustration of regression analysis (using actual data to develop a predictive equation). Were other forms of this equation possible? Certainly! Some of the different, possible regression equations follow:

(a) PSR vs. $\log_{10}(1 + \overline{SV})$ Eq. 6.2

$$PSI = 5.92 - 2.61 \log_{10}(1 + \overline{SV})$$

R^2 = 88.8%
RMSE = 0.37
n = 49

(b) PSR vs. $\sqrt{C + P}$ Eq. 6.3

$$PSI = 4.01 - 0.23 \sqrt{C + P}$$

R^2 = 81.2%
RMSE = 0.48
n = 49

(c) PSR vs. $\overline{SV}$ Eq. 6.4

$$PSI = 3.84 - 0.05 \overline{SV}$$

R^2 = 73.9%
RMSE = 0.57
n = 49

(d) PSR vs. C Eq. 6.5

$$PSI = 3.64 - 0.03C$$

R^2 = 59.4%
RMSE = 0.71
n = 49

(e) PSR vs. P Eq. 6.6

$$PSI = 3.08 - 0.02P$$

R^2 = 26.2%
RMSE = 0.95
n = 49

Table 6.1. Data Used in the Development of AASHO Road Test Correlation Between PSR and PSI for Rigid Pavements [from Ref. 6.1]

State	Mean Panel PSR	Mean Slope Variance in Wheelpath $(\overline{SV})$	Cracking — Class 3 and Sealed cracks ft/1000 ft^2 (C)	Patching — Patched Area ft/1000 ft^2 (P)	Transformations	
					log $(1 + \overline{SV})$	$\sqrt{C + P}$
Illinois	2.0	52.0	53	8	1.72	7.8
	4.2	6.5	4	0	0.88	2.0
	2.6	22.2	42	11	1.37	7.3
	2.3	26.2	46	7	1.44	7.3
	1.2	47.8	102	28	1.69	11.4
	2.8	25.5	15	1	1.42	4.0
	4.4	3.2	0	0	0.63	0.0
	1.1	50.8	65	5	1.71	8.4
	0.9	76.8	74	85	1.89	12.6
Minnesota	1.3	43.3	40	59	1.65	10.0
	1.8	24.2	23	66	1.40	9.4
	2.1	24.7	47	41	1.41	9.4
	4.1	2.4	4	0	0.54	2.0
	3.8	4.0	2	0	0.70	1.4
	3.0	7.8	14	1	0.95	3.9
	3.0	7.5	22	0	0.93	4.7
	2.9	9.7	14	0	1.03	3.7
	2.5	17.6	34	0	1.27	5.8
	1.4	59.2	16	12	1.78	5.3
	4.3	3.0	0	0	0 60	0.0
	4.3	4.0	0	0	0.70	0.0
	3.7	5.3	0	0	0.80	0.0
	3.6	4.4	0	0	0.73	0.0
	3.9	5.3	0	0	0.80	0.0
	3.9	6.3	0	0	0.87	0.0
	1.3	32.3	76	1	1.52	8.8
	1.2	27.8	64	0	1.46	8.0
	2.2	25.6	97	1	1.42	9.9
	4.4	4.0	0	0	0.70	0.0
Indiana	4.0	6.6	0	0	0.88	0.0
	3.8	6.6	11	0	0.88	3.3
	3.6	6.8	2	0	0.89	1.4
	3.2	9.8	1	2	1.03	1.7
	2.6	14.6	72	0	1.19	8.5
	2.8	10.4	70	1	1.06	8.4
	1.8	49.4	41	29	1.70	8.4
	1.8	54.5	42	37	1.74	8.9
	2.1	36.6	50	29	1.58	8.9
	2.2	25.1	86	33	1.42	10.9
	1.8	45.4	40	65	1.67	10.2
	2.7	9.9	81	5	1.04	9.3
	4.2	6.1	0	0	0.85	0.0
	4.3	5.2	0	0	0.79	0.0
	4.3	7.1	0	0	0.91	0.0
	1.2	81.9	54	219	1.92	16.5
	2.2	32.2	36	0	1.52	6.0
	4.3	4.6	0	0	0.75	0.0
	2.8	12.6	5	13	1.13	4.2
	2.7	17.8	5	16	1.27	4.6

(f) PSR vs. (C + P) Eq. 6.7

$$PSI = 3.55 - 0.02\,(C + P)$$
$$R^2 = 61.2\%$$
$$RMSE = 0.69$$
$$n = 49$$

(g) PSR vs. log [(C + 1) + (P + 1)] Eq. 6.8

$$PSI = 4.61 - 1.41 \log_{10} [(C + 1) + (P + 1)]$$
$$R^2 = 84.1\%$$
$$RMSE = 0.44$$
$$n = 49$$

(h) PSR vs. $\log_{10} (1 + \overline{SV})$, $\log_{10}[(C + 1) + (P + 1)]$ Eq. 6.9

$$PSI = 5.56 - 1.64 \log_{10} (1 + \overline{SV}) - 0.62 \log_{10}[(C + 1) + (P + 1)]$$
$$R^2 = 92.7\%$$
$$RMSE = 0.30$$
$$n = 49$$

(i) PSR vs. $\log_{10} (1 + \overline{SV})$, $\sqrt{C + P}$ Eq. 6.10

Illinois data only:

$$PSI = 5.73 - 1.73 \log_{10} (1 + \overline{SV}) - 0.13\sqrt{C + P}$$
$$R^2 = 95.5\%$$
$$RMSE = 0.31$$
$$n = 9$$

Minnesota data only: Eq. 6.11

$$PSI = 5.48 - 2.08 \log_{10} (1 + \overline{SV}) - 0.08\sqrt{C + P}$$
$$R^2 = 94.5\%$$
$$RMSE = 0.28$$
$$n = 20$$

Indiana data only: Eq. 6.12

$$PSI = 5.37 - 1.62 \log_{10} (1 + \overline{SV}) - 0.09\sqrt{C + P}$$
$$R^2 = 95.2\%$$
$$RMSE = 0.23$$
$$n = 20$$

From the prior equations, we can conclude:

- If only one independent variable is to be used $\overline{SV}$ (or its transformation) is the single best predictor of PSR (Eq. 6.2 and Eq. 6.4). The worst single predictor is Patching (Eq. 6.6). The combined C + P term as a log (Eq. 6.8) can predict PSR almost as well as $\log_{10} \overline{SV}$ (Eq. 6.2)

- The fit of the data can be improved (Eq. 6.9) over the originally developed equation (Eq. 6.1); however, the improvement is rather small.

- The three separate equations (Eq. 6.10, 6.11, 6.12) based on data from the three states in the form of the original equation (Eq. 6.1, Eq. 6.2), all fit the data rather well.

1.3 AASHTO DESIGN LIMITATIONS

The *AASHTO Guide for Design of Pavement Structures* is largely based on the experimental results of the AASHO Road Test but extended with theoretical concepts. The experimental nature of the AASHO Road Test resulted in certain limitations which, to some degree, have been overcome. At any rate, some of these limitations are:

1.3.1 GENERAL LIMITATIONS

(a) Specific pavement materials and roadbed soil

(b) Single environment

(c) An accelerated two year testing period extrapolated to longer design periods

(d) Operating vehicles with identical axle loads and configurations, as opposed to mixed traffic

1.3.2 LIMITATIONS SPECIFIC TO RIGID PAVEMENTS

(a) For Jointed Plain and Reinforced Concrete Pavements, all transverse joints were the same spacing. JPCP was 15 ft (4.6 m) and JRCP 40 ft (12.2 m). All transverse joints had dowel bars.

(b) All PCC was the same mix design (and aggregate and portland concrete).

1.4 RIGID PAVEMENT MATERIALS AND DESIGNS — AASHO ROAD TEST

1.4.1 MATERIALS [6.1, 6.2]

1.4.1.1 Portland Cement Concrete

(a) Design cement factor (Type I) = 6 bags/yd^3 (564 lb/yd^3)

(b) Design maximum water content = 5.3 gal/bag of cement (W/C ratio = 0.47)

(c) Design strength (14 day cure)

- Minimum compressive strength = 3500 psi
- Minimum modulus of rupture = 550 psi (AASHTO T97 (third point loading))

(d) Design slump range: 1.5 to 2.5 in.

(e) Maximum aggregate sizes: 1.5 and 2.5 in.

(f) "Plastic" concrete results

Parameter	Maximum Aggregate Size	
	1.5 in.	2.5 in.
• Mean slump (in.)	2.7	2.5
• Mean air content (%)	4.2	3.7
• Mean cement factor (lb/yd^3)	6.07	6.08

(g) Hardened Strength Properties

Age	Maximum Aggregate Size			
	1.5 in.		2.5 in.	
	Compressive Strength (psi)	Flex. Strength (psi)	Compressive Strength (psi)	Flex. Strength (psi)
3 days	2860	550	2670	510
7 days	3780	630	3560	620
14 days	4004	668	3966	636
21 days	4250	710	4130	660
1 year	5990	880	5580	790
2 years	6155	873	5818	787

(h) Linear coefficient of thermal expansion

4.6 to 5.1 x 10^{-6} per °F

1.4.1.2 Subbase Course

The subbase course used beneath the PCC slabs was the same material as used for the subbase course in the flexible pavement sections (see SECTION 5.0, Paragraph 1.2.1.3). This material was a sand-gravel mixture which had a CBR specification not to exceed 60. The CBR tests performed indicated a mean CBR of about 35 percent. All subbase material for the rigid pavement sections was placed during May and June 1958.

Typical inplace mean dry densities ranged from 138 to 141 lb/ft^3 with mean moisture contents ranging from about 5.7 to 6.5 percent.

Permeability tests (constant head parameter) were run on the subbase. The mean permeability value was about 4 ft per day.

1.4.1.3 Embankment Soil (Subgrade)

Selected characteristics of the embankment soil upon which all pavement structures were placed was previously described in SECTION 5.0, Paragraph 1.2.1.4.

1.4.2 AASHO RIGID PAVEMENT TEST SECTIONS:

1.4.2.1 Experimental Designs [6.1]

(a) Overall, there were 260 rigid pavement test sections in the main experiment (312 counting replicates).

(b) Reinforced sections were 240 ft long with sawed, doweled transverse contraction joints spaced 40 ft apart.

(c) Nonreinforced sections were 120 ft long with sawed, doweled transverse contraction joints spaced 15 ft apart (no expansion joints).

(d) Size of dowel bars varied with slab thickness.

(e) Reinforcement was welded steel fabric placed about 2 in. from the slab surface (slabs 5 in. to 12.5 in.).

(f) Structural thicknesses for each loop (same for nonreinforced and reinforced sections).

Loop No.	PCC Slab Thickness (in.)	Subbase Thickness (in.)	Transverse Dowel Bars (Diameter x Length)
1	2.5	0, 6.0	3/8" x 12"
	5.0	0, 6.0	5/8" x 12"
	9.5	0, 6.0	1 1/4" x 18"
	12.5	0, 6.0	1 5/8" x 18"
2	2.5	0, 3.0, 6.0	3/8" x 12"
	3.5	0, 3.0, 6.0	1/2" x 12"
	5.0	0, 3.0, 6.0	5/8" x 12"
3	3.5	3.0, 6.0, 9.0	1/2" x 12"
	5.0	3.0, 6.0, 9.0	5/8" x 12"
	6.5	3.0, 6.0, 9.0	7/8" x 18"
	8.0	3.0, 6.0, 9.0	1" x 18"
4	5.0	3.0, 6.0, 9.0	5/8" x 12"
	6.5	3.0, 6.0, 9.0	7/8" x 18"
	8.0	3.0, 6.0, 9.0	1" x 18"
	9.5	3.0, 6.0, 9.0	1 1/4" x 18"
5	6.5	3.0, 6.0, 9.0	7/8" x 18"
	8.0	3.0, 6.0, 9.0	1" x 18"
	9.5	3.0, 6.0, 9.0	1 1/4" x 18"
	11.0	3.0, 6.0, 9.0	1 3/8" x 18"
6	8.0	3.0, 6.0, 9.0	1" x 18"
	9.5	3.0, 6.0, 9.0	1 1/4" x 18"
	11.0	3.0, 6.0, 9.0	1 3/8" x 18"
	12.5	3.0, 6.0, 9.0	1 5/8" x 18"

1.4.2.2 Overview of Rigid Pavement Performance

Of the various axle loads applied to each of the test loops, a total of 1,114,000 load applications were made (not ESALs). Of the 312 rigid test sections, only 67 sections (or 21 percent) had reached a PSI of 1.5 or less at the end of the load application period (from November 1958 to June 1960).

The following overviews the actual performance for Loops 4 and 6 (Loop 4 had an 18,000 lb single axle as one of its standard loads and Loop 6 had the heaviest axle loads used at the Road Test). Refer to Tables 6.2 and 6.3. Table 6.2 reveals that the then standard axles caused failure in up to the 6.5 in. thick PCC slabs (8.0 in. and thicker slabs did not fail). For the heavier axles used in Loop 6, slabs up to 9.5 in. did fail

Table 6.2 AASHO Road Test Loop 4 Results — Rigid Test Sections [after Ref. 6.1]

Slab Thickness (in.)	Axle	Subbase Thickness (in.)					
		3.0		6.0		9.0	
		Plain	Reinforced	Plain	Reinforced	Plain	Reinforced
5.0	18,000 lb single	716,000	415,000	353,000	325,000	291,000	592,000
	32,000 lb tandem	343,000	304,000	328,000	175,000	298,000	408,000
6.5	18,000 lb single	(3.8)	(3.6)	(4.3)	(3.4)	(3.0)	(1.8)
	32,000 lb tandem	687,000	793,000	1,000,000	796,000	722,000	1,036,000
8.0	18,000 lb single	(4.5)	(3.9)	(4.4)	(3.9)	(4.3)	(4.3)
	32,000 lb tandem	(4.2)	(4.0)	(4.2)	(3.8)	(4.1)	(4.2)
9.5	18,000 lb single	(4.2)	(4.0)	(4.5)	(4.5)	(4.1)	(4.8)
	32,000 lb tandem	(4.0)	(4.0)	(4.2)	(4.3)	(4.2)	(4.6)

Notes:
1. Replicate sections not shown
2. Values in parentheses are PSI at end of load applications
3. Values not in parentheses are the numbers of load applications to reach a PSI = 1.5.

Table 6.3 AASHO Road Test Loop 6 Results — Rigid Test Sections [after Ref. 6.1]

Slab Thickness (in.)	Axle	Subbase Thickness (in.)					
		3.0		6.0		9.0	
		Plain	Reinforced	Plain	Reinforced	Plain	Reinforced
8.0	30,000 lb single	878,000	782,000	(3.9)	974,000	(3.4)	768,000
	48,000 lb tandem	618,000	(1.8)	(4.1)	415,000	1,114,000	624,000
9.5	30,000 lb single	(3.6)	(1.6)	(4.3)	(4.0)	(4.2)	(2.2)
	48,000 lb tandem	(3.1)	(4.1)	(4.3)	(4.0)	(4.3)	912,000
11.0	30,000 lb single	(4.4)	(4.4)	(4.2)	(4.3)	(4.3)	(4.2)
	48,000 lb tandem	(4.3)	(4.4)	(4.3)	(4.2)	(4.3)	(4.1)
12.5	30,000 lb single	(4.2)	(4.4)	(4.0)	(4.2)	(4.2)	(4.5)
	48,000 lb tandem	(4.3)	(4.3)	(4.2)	(4.4)	(4.4)	(4.2)

(11.0 in. thick slabs and thicker did not have any failures). Further, subbase thickness did not seem to matter much.

Subsequent work reported by Darter [6.5] in 1977 examined the in-service AASHO Road Test PCC sections incorporated into what became I-80. Measurements obtained in 1974 (16 years after construction) were made for various distress types including contraction joint faulting. Since all joints were doweled and spaced 15 ft apart, such data provide some insight into the effect of slab thickness (only the PCC slabs on 3 in. or more of subbase are presented):

Loop	PCC Slab Thickness (in.)	Dowel Diameter (in.)	Accumulated ESALs (x 10^6)	Joint Faulting (in.)
4	8.0	1	11.2	0.25
	9.5	1-1/4	11.3	0.05
5	9.5	1-1/4	13.8	0.06
	11.0	1-3/8	14.1	0.03
6	9.5	1-1/4	17.8	0.09
	11.0	1-3/8	18.9	0.05
	12.5	1-5/8	19.5	0.03

As a comparison, ESAL levels of about 13,000,000 were estimated for WSDOT I-5, MP 176 (Seattle) over a 22-year period (1965-1987) and 5,600,000 ESALs for WSDOT I-90, MP 279 (Spokane) also for a 20-year period.

Interestingly, I-90 in Spokane (vicinity of MP 276) exhibited joint faulting of about 0.22 in. (an average) in 1987 after accumulating 5,600,000 ESALs from the time of original construction (1965). Further, the I-90 slab thickness was 7.8 in. on a 5.4 in. granular base (the slabs were undoweled with a 15 ft transverse joint spacing). Thus, the I-90 ESALs were 1/2 those for I-80 in Illinois for about the same slab thickness and joint faulting (5,600,000 ESALs for 7.8 in. slab vs. 11,200,000 ESALs for an 8.0 in. slab). Such evidence is far from conclusive but suggests that dowel bars can significantly increase pavement performance (at least as measured by joint faulting).

I-5 in Seattle (vicinity of MP 176) had an average faulting level of about 0.08 in. in 1987 after 13,000,000 ESALs for a 9.0 in. PCC slab on a 7.0 in. granular base. Thus, the faulting was only a bit higher than the I-80/AASHO Road Test section which was 9.5 in. thick PCC (Loop 5) at a similar ESAL level. The I-5 section was undoweled with a 15 ft transverse joint spacing. As of December 1992 and a total accumulated ESALs of about 18,000,000 ESALs, noticeable transverse joint faulting is occurring in the vicinity of MP 176. This only suggests that the faulting performance of I-5 will not be as good as the I-80/AASHO Road Test sections which had a 9.5 in. doweled slab (Loop 6) at similar ESAL levels (≈ 18,000,000 for I-5 and 17,800,000 for I-80).

1.4.3 AASHTO RIGID PAVEMENT PERFORMANCE EQUATIONS

1.4.3.1 Pre-1986 Performance Equations

The following equation was used to produce the rigid pavement design nomograph for the pre-1986 AASHTO Guide [6.3]:

$$\log W_{18} = 7.35 \log (D + 1) - 0.06 - \frac{0.1761}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} \qquad \text{Eq. 6.13}$$

$$+ 3.42 \log \left[\frac{f_t}{690} \left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{(E/k)^{0.25}}} \right) \right]$$

where p_t was fixed at 2.5.

1.4.3.2 1986 Performance Equation

The 1986 version of the performance equation [6.4]:

$$\log W_{18} = (z_R)(S_o) + 7.35 \log (D + 1) - 0.06 \qquad \text{Eq. 6.14}$$

$$+ \frac{\log \left[\frac{\Delta PSI}{4.5-1.5} \right]}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} + (4.22 - 0.32 p_t) \log \left[\frac{(S_c')(C_d)(D^{0.75} - 1.132)}{215.63(J)\left(D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right)} \right]$$

1.4.3.3 Discussion

The pre-1986 performance equation had the p_t fixed at 2.5. The working stress (f_t) was calculated by S_c/C where C = safety factor (generally ranged from 1.33 to 2.0). Further, the performance equation (and hence the design chart) assumed joint load transfer to be equal to the doweled joints at the AASHO Road Test. Thus, J was fixed at 3.2 and was not a variable.

The 1986 performance equation assumes "mean" values will be used for design. The term "$(z_R)(S_0)$" allows for the incorporation of reliability into the process. This was described in some detail previously.

1.5 RESEARCH ON RIGID PAVEMENT DESIGN FEATURES

Background information was provided in SECTION 4.0 Paragraph 6, "Stresses in Rigid Pavements," on basic issues relating to PCC slab thickness, transverse contraction joint spacing, and joint load transfer. Essentially it comes down to

- PCC slabs: thicker is better.
- Joint spacing: shorter is better than longer.
- Joint load transfer: use of dowel bars is better than aggregate interlock.

Recent work performed for the FHWA and reported during 1990 by Smith et al. [6.6] was used to examine 95 in-service rigid pavements located throughout the U.S. These sections included

- Jointed reinforced and jointed plain slabs.
- PCC slab thicknesses ranging from 7 to 13 in. (mostly 9 in.).
- Plain jointed slabs with joint spacings ranging from 5 to 30 ft. The shortest joint spacing was in an experimental project on I-5 in California (Tracy) and the longest in an experimental project on I-95 in North Carolina (Rocky Mount).
- Transverse joint load transfer included both dowel bars and aggregate interlock.
- Base types included
    - Dense graded aggregate
    - CTB
    - ATB
    - ATPB
    - Lean concrete base (LCB)
    - No base (slabs directly on subgrade)

Further, the pavements were located in ten states and one province, which were

- Arizona
- California
- Florida
- Michigan
- Minnesota
- New Jersey
- New York
- North Carolina
- Ohio
- Ontario
- Pennsylvania

The basic conclusions of the research team [6.6] relating to the influence of the various design features follow (conclusions summarized primarily for plain jointed slabs since jointed reinforced is not used by WSDOT).

(a) Slab thickness

Increased slab thickness did not overcome joint spalling or faulting — particularly for undoweled joints; however, transverse and longitudinal slab cracking was reduced. Note that most of the JPCP sections studied had 9 in. thick PCC slabs — more specifically these were

Slab Thickness (in.)	Number of Sections
7.0	1
8.0	3
8.4	6
9.0	27
11.0	1
11.4	1
12.0	1
13.0	2
	Total 42

(b) Base type

- The base types which performed the worst were cement stabilized (CTB and soil-cement) which manifested distresses of pumping, faulting, and cracking. Lean concrete bases also had performance problems contributing to large slab curling and warping stresses which resulted in transverse and longitudinal slab cracking.

- Dense graded asphalt treated bases (ATB) had variable performance ranging from very poor to good. Associated distresses were slab cracking, faulting, and pumping.

- Dense aggregate bases were rated as performing fair to very good. This type of base "contribute(s) the least to … high curling and warping stresses in the slab."

- The best performing bases were permeable; however, at the time of the study (field data were collected in 1987) most permeable bases were relatively "young" with associated low ESAL levels. Some concern was expressed about measured high corner deflections and low load transfer. Apparently, dowel bars may be extra important for achieving adequate long-term performance when permeable bases are used.

(c) Slab length

For the 42 JPCP sections studied, the joint spacings can be summarized as

Joint Spacing (ft)		
Actual	Average	Number of Sections
5-8-11-7	7.75	1
12-13-17-16	14.5	2
13-15-17-15	15.0	6
12-13-19-18	15.5	17
13-16-14-19	15.5	2
20.0	—	6
26.7	—	1
30.0	—	7
		Total 42

The shorter joint spacings had reduced faulting and transverse cracking. However, the shortest joint spacings (Caltrans 5-8-11-7) may be too short. By 1988-1989, the shorter slabs were experiencing corner breaks and longitudinal cracks. Slabs 18 ft or longer (random spacing arrangement) had more transverse cracks than slabs with shorter spacings. Similar observations were made by WSDOT/UW personnel following an examination of California Interstate pavements in 1988 (California currently uses a reduced random spacing (nondoweled)).

(Note: The FHWA recommends in Technical Advisory 5040.30 [6.8] that the maximum slab length be limited by L/ℓ 5.0 (L = slab length and ℓ = radius of relative stiffness).)

(d) Joint orientation

- No definite conclusions were made about the effectiveness of skewed, undoweled transverse joints.
- The research team did <u>not</u> believe there was any benefit for using random, skewed joints with doweled slabs.

(e) Transverse joint load transfer

- Use of dowel bars significantly reduced faulting as compared to similar undoweled sections. This faulting reduction was most significant at higher ESAL levels.
- Dowel bar diameter was important. It was recommended that the minimum dowel bar diameter be 1.25 in. (32 mm).
- Dowel bar coatings appeared to enhance pavement performance by inhibiting dowel corrosion.

(f) PCC tied shoulders

- The performance benefit of tied shoulders was mixed. It was stated that to maximize performance benefits
  - tiebars must be located near slab corners.
  - in frost areas the PCC shoulder should have the same thickness as the main lanes in order to avoid differential frost heave.
- Tied shoulders had no apparent effect in reducing joint faulting in the adjacent main lane.
- One benefit of tied shoulders is a sealable joint reservoir between the shoulder and main lane. This joint is a potential source of substantial infiltration water in traditional PCC/ACP joints.

(g) Widened traffic lanes

This design feature was not evaluated but is believed to potentially provide improved pavement performance.

(h) Subdrainage

Improved subdrainage (either by use of longitudinal edge drains or in combination with a drainage layer) generally reduced faulting.

2. RIGID PAVEMENT DESIGN PROCEDURE FOR NEW CONSTRUCTION OR RECONSTRUCTION (PART II OF AASHTO GUIDE)

2.1 OUTLINE AND BACKGROUND INFORMATION FOR USING PART II (NEW CONSTRUCTION OR RECONSTRUCTION)

(a) Estimate future traffic (ESALs)
(b) Select reliability (R)
(c) Select overall standard deviation (S_o)
(d) Estimate/select design serviceability loss (PSI)
(e) Estimate PCC elastic modulus (E_c)
(f) Estimate PCC modulus of rupture (S'_c)
(g) Select/estimate load transfer coefficient (J)
(h) Select/estimate drainage coefficient (C_d)
(i) Estimate effective modulus of subgrade reaction (k)

Each of the above items will be examined in a bit more detail in the following subsections.

2.1.1 ESTIMATE FUTURE TRAFFIC

The principles for estimating the design period ESALs are the same for rigid pavements as for flexible pavements (refer back to SECTION 5.0, Paragraph 2.2.1.2 ("Traffic")). However, the axle load equivalency factors are a bit different for rigid pavements (refer to Tables D.10 through D.18, Appendix D, *AASHTO Guide for Design of Pavement Structures* and Table 6.4 (this table provides a sample of AASHTO equivalency factors)). As an illustration of this, refer to Table 6.5 which illustrates a comparison of load equivalency factors (LEFs) for both flexible and rigid pavement structures. The data in Table 6.5 strongly suggests that it is ill-advised to assume flexible pavement calculated ESALs can be extrapolated to rigid pavements (and, of course, vice versa).

2.1.2 SELECT RELIABILITY

Refer specifically to SECTION 2.1.3 in Part II of the *AASHTO Guide*. The table in that section offers <u>ranges</u> of reliability levels for various highway functional classifications. The ranges are a bit higher for urban conditions (as compared to rural). This should be no surprise since the "tolerance" of urban traffic (levels) for pavement maintenance and rehabilitation is not high. When designing both flexible and rigid pavements sections (say for a LCC analysis and pavement type selection) use the <u>same</u> reliability level for both.

Table 6.4. A Sample of AASHTO Traffic Equivalency Factors for Rigid Pavement (from Ref. 6.4)

Single Axles

Axle Load		Slab Thickness and p_t								
		D = 6 in.			D = 10 in.			D = 14 in.		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
10	44.5	0.087	0.097	0.111	0.080	0.081	0.081	0.080	0.080	0.080
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.55	1.51	1.46	1.58	1.58	1.57	1.59	1.59	1.59
30	133.4	8.92	8.16	7.27	9.23	8.79	8.25	9.56	9.52	9.46
40	177.9	32.2	29.1	25.6	31.6	27.9	23.8	34.4	33.7	32.9

Tandem Axles

Axle Load		Slab Thickness and p_t								
		D = 6 in.			D = 10 in.			D = 14 in.		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
10	44.5	0.014	0.015	0.018	0.012	0.012	0.013	0.012	0.012	0.012
20	89.0	0.216	0.234	0.260	0.203	0.204	0.205	0.203	0.203	0.203
30	133.4	1.13	1.11	1.09	1.14	1.14	1.14	1.14	1.14	1.14
34	151.2	1.90	1.82	1.72	1.96	1.95	1.93	1.97	1.97	1.97
40	177.9	3.79	3.52	3.21	3.95	3.87	3.76	4.01	4.01	4.00
50	222.5	10.00	9.11	8.09	10.27	9.68	8.96	10.73	10.66	10.58
60	267.0	22.4	20.3	17.9	22.3	20.0	17.4	24.0	23.6	23.2
70	311.5	44.6	40.3	35.4	42.9	37.0	30.6	47.3	45.9	44.1

Triple Axles

Axle Load		Slab Thickness and p_t								
		D = 6 in.			D = 10 in.			D = 14 in.		
kips	**kN**	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0	p_t = 2.0	p_t = 2.5	p_t = 3.0
20	89.0	0.072	0.080	0.091	0.066	0.066	0.067	0.066	0.066	0.066
30	133.4	0.364	0.384	0.411	0.350	0.351	0.353	0.349	0.349	0.349
40	177.9	1.17	1.14	1.11	1.18	1.18	1.18	1.19	1.18	1.18
50	222.5	2.95	2.76	2.53	3.07	3.02	2.96	3.11	3.11	3.10
60	267.0	6.45	5.91	5.28	6.69	6.42	6.08	6.90	6..87	6.84
70	311.5	12.6	11.5	10.2	12.8	11.9	10.8	13.6	13.5	13.3
80	356.0	22.8	20.6	18.2	22.6	20.2	17.4	24.4	24.0	23.5
90	400.5	38.5	34.8	30.5	37.2	32.2	26.8	40.9	39.8	38.3

Table 6.5 Comparison of Flexible and Rigid Pavement LEFs

Pavement Type	Axle Load (lbs)	p_t Level 2.0	2.5	3.0
Rigid	10,000	0.08	0.08	0.08
• Single Axles	18,000	1.00	1.00	1.00
• Slab Thick = 10"	20,000	1.58	1.58	1.57
	30,000	9.23	8.79	8.25
Flexible	10,000	0.08	0.10	0.13
• Single Axles	18,000	1.00	1.00	1.00
• SN = 4	20,000	1.55	1.47	1.38
	30,000	8.60	6.80	5.00
Rigid	30,000	1.14	1.14	1.14
• Tandem Axle	34,000	1.96	1.95	1.93
• Slab Thick = 10"	40,000	3.95	3.87	3.76
	48,000	8.63	8.21	7.69
Flexible	30,000	0.64	0.70	0.77
• Tandem Axle	34,000	1.08	1.11	1.15
• SN = 4	40,000	2.13	2.03	1.89
	48,000	4.58	3.98	3.29

Recall from SECTION 5.0 that reliability as applied in the *AASHTO Guide* is a multiplier onto the design ESALs (w_{18}). For example, <u>if</u> we choose the following reliability levels for

Functional Class	Reliability
• Interstate	
• Urban (e.g., I-5)	95%
• Rural (e.g., I-90)	90%
• Principal Arterial	
• Urban (e.g., US99)	85%
• Rural (e.g. US2)	80%
• Collector	
• Urban (e.g., SR303)	80%
• Rural (e.g., SR172)	75%

then how would these levels <u>affect</u> the ESAL level actually used in the pavement structural design?

Recall from SECTION 5.0 the following:

$$W_t = (w_T)(F_R)$$

where W_t = ESALs predicted by the performance equation to p_t,
w_T = ESALs predicted (or estimated) for the design period,
F_R = reliability design factor.
= $10^{-Z_R(S_0)}$

Now, calculate F_R for the various reliability levels we chose earlier and use S_o values of 0.35 (optimistic) and 0.40 (not so optimistic).

Reliability Level (%)	Z_R	Calculated F_R	
		S_o = .35	S_o = .40
75	-0.67	1.72	1.85
80	-0.84	1.97	2.17
85	-1.04	2.31	2.61
90	-1.28	2.80	3.25
95	-1.65	3.78	4.57

Thus, depending on the reliability and S_o levels, the estimated design ESALs will be increased by factors ranging from 1.72 (a 72 percent increase) to 4.57 (a 357 percent increase).

If we choose an even more extreme reliability level such as 99.9 percent, then F_R becomes 12.06 and 17.22, respectively, for S_o's of 0.35 and 0.40.

2.1.3 SELECT OVERALL STANDARD DEVIATION

For rigid pavements, an optimistic S_o is 0.35 (optimistic in that the prediction and traffic errors are rather modest). Possibly, a more realistic S_o is about 0.40. Of all the required design variables in the 1986 and 1993 *AASHTO Guides*, the S_o value is likely the least understood. As observed in the preceding section, modest differences in S_o has a minimal effect on the design ESALs and hence on slab thickness.

2.1.4 DETERMINE DESIGN SERVICEABILITY LOSS

Two decisions must be made to determine the design serviceability loss (PSI). First, what initial PSI level can be reasonably expected (p_o of 4.0, 4.2, 4.5 or what)? This, undoubtedly, is a function of project specific conditions. If the project is on an urban freeway where a lane is being added, the initial p_o might be a bit on the low side (say 4.0). If the project is in a rural area with unobstructed construction, the p_o should be somewhat higher (say 4.5).

The terminal serviceability index (p_t) is a designer selection. Often values of 2.0, 2.5, or 3.0 are selected. The selection of the p_t is a function of how low the PSI will be allowed

to go following application of the design period ESALs. One might want to consider a p_t of about 3.0 (or more) for freeways.

2.1.5 ESTIMATE PCC ELASTIC MODULUS

The PCC elastic modulus (E_c) can be estimated from relationships such as the following:

$$E_c = 57{,}000\ (f_c')^{0.5}$$

where E_c = PCC elastic modulus (psi),

f_c' = PCC compressive strength as determined by use of ASTM C39

The above equation implies that the PCC modulus increases as the compressive strength increases, albeit somewhat less. For example:

If f_c' = 4,000 psi
then $E_c \cong$ 3,600,000 psi

If f_c' = 10,000 psi
then E_c = 5,700,000 psi

If no compressive strength data are available (or cannot be assumed), assume E_c = 4,000,000 psi (corresponds to an f_c' = 5,000 psi).

2.1.6 ESTIMATE PCC MODULUS OF RUPTURE

The modulus of rupture (flexural strength) for PCC is the <u>mean</u> value (28 day cure) using the third-point loading scheme (AASHTO T 97, ASTM C78). This input value is designated as S'_c. Typically, for WSDOT PCC mixes, this value will range from about 650 to 800 psi.

2.1.7 SELECT LOAD TRANSFER COEFFICIENT

The load transfer coefficient (J Factor) accounts for the relative ability of a concrete pavement to transfer load across discontinuities such as transverse contraction joints. A range of J Factors is shown in SECTION 2.4.2 of the *AASHTO Guide*. Essentially the <u>lower</u> the J Factor, the <u>better</u> the load transfer. The J Factor for the AASHO Road Test (15 ft transverse joint spacing with dowel bars) was estimated to be 3.2.

The development of the J Factor is treated in Appendix 6.1. An initial estimate of J Factor for six in-service WSDOT pavements was made. The J Factors in the Seattle area ranged from 2.9 to 3.8. J Factors in the Snoqualmie Pass area ranged from 3.6 to 3.9 and in the Vancouver area about 3.8. The joint spacing at all five locations examined are at 15 ft at a right angle ("straight"). There are no shape factors on these joints other than the standard width required to create an initial saw cut.

The J Factor for I-90 in Spokane (vicinity of MP 276) was estimated to be within the range of 3.5 to 3.8. This pavement was originally constructed as a 7.8 in. PCC slab on 5.4 in. of granular base in 1965. It accumulated about 5,600,000 ESALs over a period of 22 years. (Note: The J Factor was calculated using a Reliability Level of 50 percent.) A similar treatment was done for I-5 in Seattle (vicinity of MP 176). This pavement was originally constructed as a 9.0 in. PCC slab on a 7.0 in. granular base in 1965. It accumulated about 18,000,000 ESALs over a period of 27 years. The J Factor for this initial performance period was estimated to be within the range of 3.4 to 3.8.

Refer to Appendix 6.2 for additional details on joints.

2.1.8 SELECT DRAINAGE COEFFICIENT

The possible drainage coefficients (C_d) for rigid pavements are shown in Table 2.5, Part II of the AASHTO Guide. C_d ranges from a low of 0.70 (worst case) to 1.25 (best case). The selection of a C_d is a function of "quality of drainage" and "percent of time pavement is exposed to moisture levels approaching saturation." As noted by Seeds and Hicks [6.9], C_d influences rigid pavement performance (hence slab thickness) in the same proportional manner as modulus of rupture (S_c') and the load transfer coefficient (J) (refer to rigid pavement performance equation shown as Equation 6.14).

The influence of C_d values on performance can be illustrated by calculating how the slab support (k) would have to vary to achieve the same effect. The following results were reported by Seeds and Hicks [6.9] (based on a 10 in. PCC slab with E_c = 5,000,000 psi):

Quality of Drainage	Selected C_d Value	Corresponding k-value (pci)
Excellent	1.2	942
Good	1.1	501
Fair	1.0	200
Poor	0.9	44
Very Poor	0.8	1

Clearly, the selection of a C_d value can have a significant impact on the slab design.

If subsurface drainage is expected to be a problem, positive drainage measures should be taken. In general, the use of drainage coefficients to overcome poor drainage conditions is not recommended, i.e., more slab thickness does not necessarily solve water-related problems.

2.1.9 ESTIMATE EFFECTIVE MODULUS OF SUBGRADE REACTION

The modulus of subgrade reaction (k) is used to estimate the "support" of the PCC slab by the layers below (the "effective" k includes the effect of the subbase and subgrade weighted by season). The value of k is in terms of pounds per square inch of deflection (or pci) and ranges from about 50 pci (weak support) to over 1000 pci (strong support). The magnitude of k can be determined by correlation with other tests (such as R-value,

CBR, or modulus of elasticity) or field tests (refer to AASHTO T221 or T222). Refer to Paragraph 2.4, SECTION 2.0, for additional background information.

To determine the effective k by use of the *AASHTO Guide*, the following steps are required (all quoted figures and tables are from the *AASHTO Guide*):

(a) Determine the seasonal roadbed soil resilient moduli — enter into Column 2, Table 3.2, p. II-38 (1993 *AASHTO Guide*). A sketch of AASHTO Table 3.2 is shown in Figure 6.1.)

(b) Determine the subbase elastic moduli for each season — enter into Column 3, AASHTO Table 3.2.

(c) Estimate the composite modulus of subgrade reaction for each season — enter into Column 4, Table 3.2. These k's are determined by use of AASHTO Figure 3.3, p. II-39 (input values are subgrade M_R, subbase thickness (assume), and subbase elastic modulus). (Refer to Step 1 in the Figure 6.2 sketch.)

(d) Adjust the k-value for the effect of a rigid foundation (a rigid foundation is any stiff layer condition (e.g., bedrock) which is within 10 ft of the surface). Use AASHTO Figure 3.4, p. II-40, to make this adjustment if required. Enter the result into Column 5, AASHTO Table 3.2. (Refer to Step 2 in the Figure 6.2 sketch.)

(e) Estimate the thickness of slab, then use AASHTO Figure 3.5, p. II-41, to determine the relative damage (u_r) for each season. Enter the result into Column 6, AASHTO Table 3.2. (Refer to Step 3 in the Figure 6.2 sketch.)

(f) Add all the u_r values and divide by the total number of seasonal increments.

(g) The effective modulus of subgrade reaction corresponds to the average relative damage and projected slab thickness.

(h) Adjust the effective modulus of subgrade reaction to account for the potential Loss of Support (due to subbase erosion). Use AASHTO Figure 3.6, p. II-42, to do this. For guidance on selection of LS, refer to AASHTO Table 2.7, p. II-27. In general, if the PCC slab is to be placed on unbound granular subbase (such as WSDOT CSTC), use a LS within the range of 1.0 to 2.0. If placed on ATB subbase, use LS $\simeq$ 1.0. If placed on ACB (such as Class E), the LS should range between 0.0 and 1.0. (Refer to Step 4 in the Figure 6.2 sketch.)

Column					
(1)	(2)	(3)	(4) Step 1	(5) Step 2	(6) Step 3
Month	Roadbed Modulus (M_R)	Subbase Modulus (E_{SB})	Composite k-value	k-value on Rigid Foundation	Relative Drainage (u_r)
Jan					
Feb					
March					
April					
May					
June					
July					
Aug.					
Sept.					
Oct.					
Nov.					
Dec.					

(AASHTO Table 3.2)

$\sum u_r =$

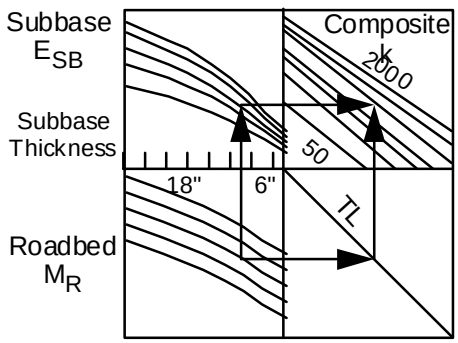
Average $\bar{u}_r = \sum \frac{u_r}{n}$

Effective k (pci) = _____

Corrected for Loss of Support, k(pci) = _____
(Step 4)

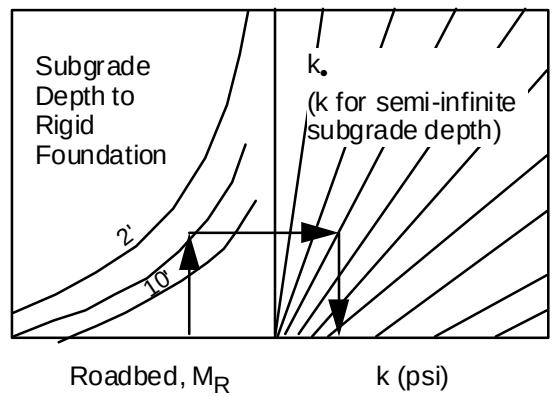
Figure 6.1. Sketch of AASHTO Table for Estimating Effective Modulus of Subgrade Reaction

Step 1
Estimate Composite Modulus of Subgrade Reaction



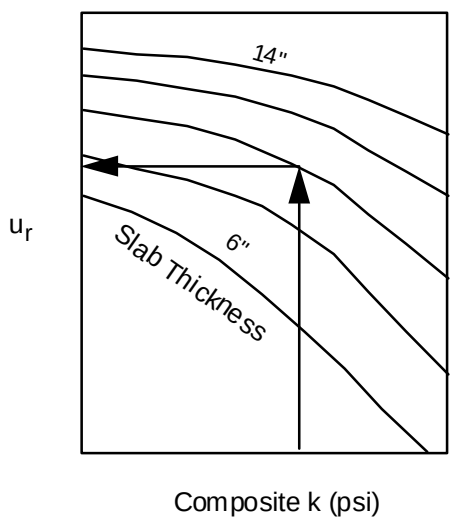
(AASHTO Figure 3.3)

Step 2
Modify k for Effects of Rigid Foundation (if needed)



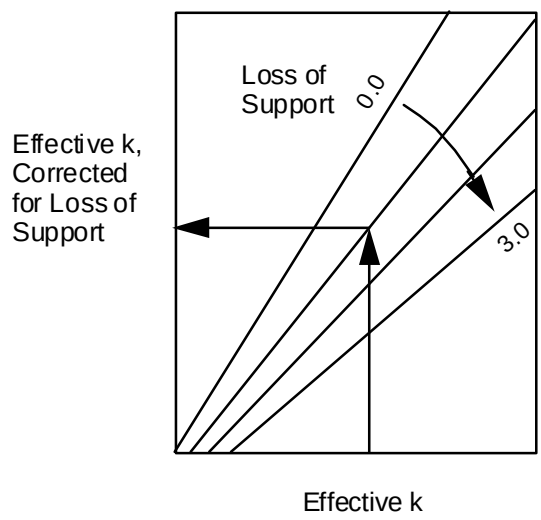
(AASHTO Figure 3.4)

Step 3
Estimate Relative Damage



(AASHTO Figure 3.5)

Step 4
Correct M_R for Loss of Support



(AASHTO Figure 3.6)

(not to scale)

Figure 6.2. Steps Required to Obtain Effective Modulus of Subgrade Reaction

Some of the equations used to generate the various nomographs which are used to calculate an effective modulus of subgrade reaction follow (from Appendix HH, 1986 *AASHTO Guide*).

(a) Modulus of subgrade reaction as a function of resilient modulus

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

(b) Rigid pavement relative damage

(i) General formula

$$u_{r_i} = \left[D^{0.75} - \frac{18.42}{(E_c/k_i)^{0.25}} \right]^{b}$$

where $b = 4.22 - 0.32(p_t)$
D = PCC slab thickness (in.),
E_c = PCC elastic modulus (psi), and
k_i = modulus of subgrade reaction for i^{th} time period (pci).

(ii) Formula used to generate AASHTO Figure 3.5, p. II-41, 1993 *AASHTO Guide*

- $p_t = 2.5$ (fixed)
- $E_c = 5{,}000{,}000$ psi (fixed)

$$u_{r_i} = [\, D^{0.75} - (0.39)\,(k)^{0.25}]^{3.42}$$

(iii) Average relative damage

$$\bar{u}_r = \frac{\sum_{i=1}^{n} u_{r_i}}{n}$$

(c) Effective modulus of subgrade reaction

$$k_{eff} = \frac{E_c}{\left[\dfrac{18.42}{D^{0.75} - (\bar{u}_r)^{1/b}} \right]^4}$$

where $b = 4.22 - 0.32\,(p_t)$

but E_c was fixed at 5,000,000 psi and

p_t = 2.5, thus

$$k_{eff} = \frac{5{,}000{,}000}{\left[\dfrac{18.42}{D^{0.75} - (\bar{u})^{1/3.42}}\right]^4}$$

(d) Example

- PCC slab = 10 in.
- No subbase (simplifying assumption)
- LS = 0.0
- E_c = 5,000,000
- p_t = 2.5
- Roadbed M_r = 10,000 psi (6 months)
  M_r = 15,000 psi (6 months)

- Calculations

  - $k_1 = {}^{10{,}000}/_{19.4} = 515$

  - $k_2 = {}^{15{,}000}/_{19.4} = 773$

  - $u_{r_1} = [\,10^{0.75} - 0.39(515)^{0.25}\,]^{3.42} = 93$

  - $u_{r_2} = [\,10^{0.75} - 0.39(773)^{0.25}\,]^{3.42} = 77$

    If k_1 and k_2 exist for 6 months each, then

$$\bar{u} = \frac{(93 \times 6) + (77 \times 6)}{12} = 85$$

$$k_{eff} = \frac{5{,}000{,}000}{\left[\dfrac{18.42}{D^{0.75} - (85)^{1/3.42}}\right]^4} \cong 638$$

k_{eff} checks with Figure 3.5, p. II-41, 1993 *AASHTO Guide*

If M_R = 10,000 psi for all 12 months,

then $k_1 = k_{eff}$ = 515 pci.

2.1.10 DETERMINE SLAB THICKNESS

Figure 6.3 is used to illustrate how the PCC slab thickness is obtained using the required input values. (Figure 6.3 is a sketch of AASHTO Guide Figure 3.7.) These inputs are the effective k-value, PCC mean modulus of rupture, load transfer coefficient, drainage coefficient, change in PSI, reliability level, overall standard deviation, and ESALs.

The nomograph simply solves the rigid pavement performance equation shown earlier in this SECTION as Equation 6.14. It is expected that nomograph use will be quite limited since all Regional Materials Offices have been provided with the current version of the AASHTO pavement design software (DARWIN).

2.2 WSDOT DESIGN INPUTS

There cannot be a "fixed list" of design inputs for use in the *AASHTO Guide* which can be applied to all WSDOT rigid pavement designs; however, some guidance is offered as a starting point in the design process (typical values and associated ranges; required decisions). Further, knowledge about these inputs will improve and undoubtedly change.

2.2.1 FUTURE ESALS

A primary difficulty in estimating ESALs is the annual growth rates to use for traffic growth (specifically truck growth rates). As an example, for a 40 year ESAL estimate:

Assumed Annual Growth Rate	Multiplier for Initial Annual ESALs[1] (40 year design)	Percent Increase
0%	40.0	0%
1%	48.9	22%
2%	60.4	51%
3%	75.4	88%
4%	95.0	138%
5%	120.8	202%

Note 1: $\frac{(1 + g)^n - 1}{g}$

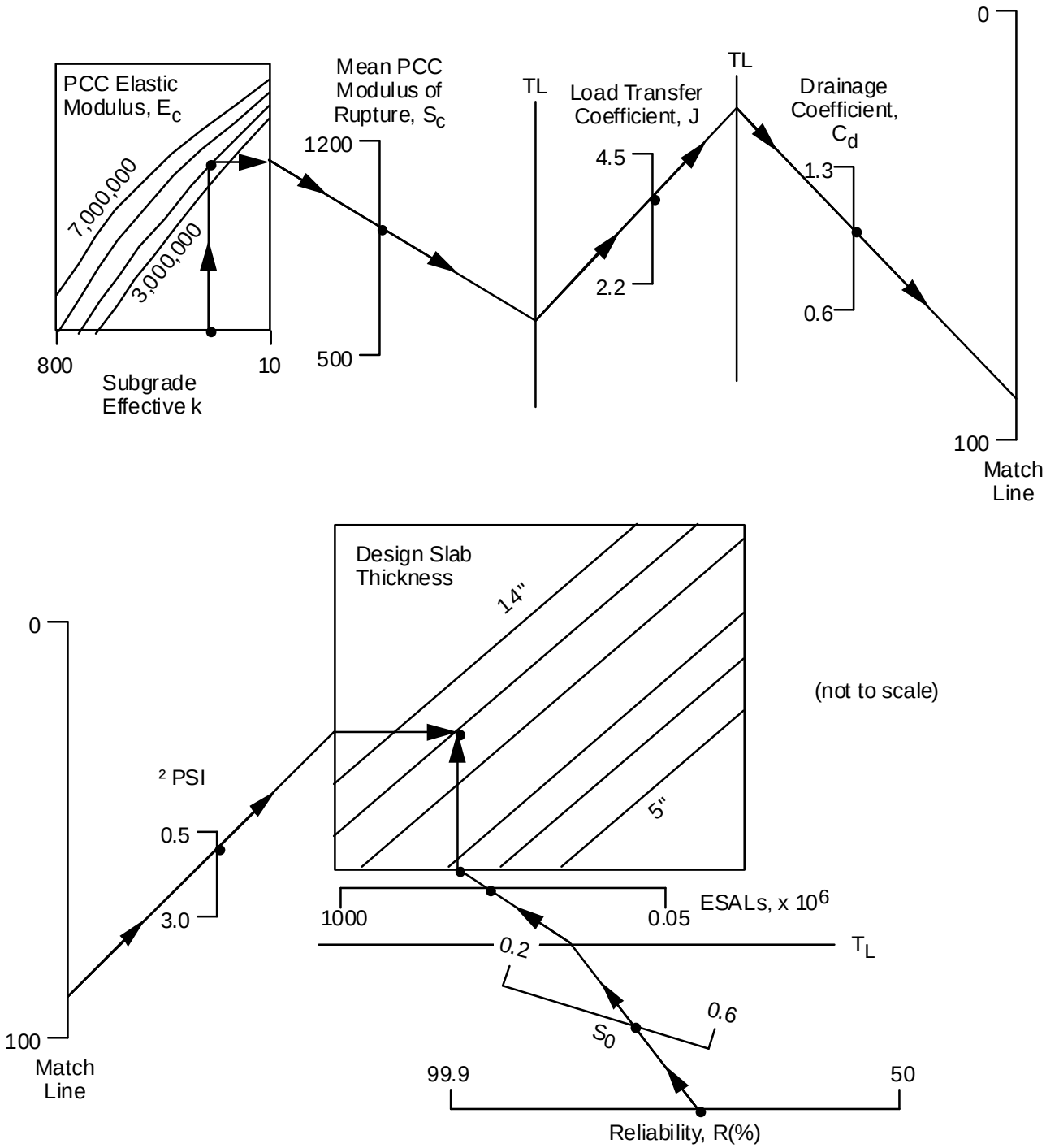


Figure 6.3. Illustration of AASHTO Design Nomograph for Rigid Pavements (Sketch of AASHTO Figure 3.7)

If the initial annual ESALs are 500,000, then the following results:

Assumed Annual Growth Rate	Design Period ESALs
0%	20,000,000
1%	24,000,000
2%	30,000,000
3%	38,000,000
4%	48,000,000
5%	60,000,000

Clearly, care must be taken in selecting annual growth rates.

Typically, annual ESAL accumulations on the WSDOT Interstate system are currently about 1,000,000 per year on I-5 and 500,000 on I-90 (in the vicinity of Spokane). These are only approximate estimates in the "design lane" and vary project-to-project. WSDOT has installed an extensive WIM system, and initial results provide substantial insight into not only initial annual ESAL levels but associated growth rates.

Refer to SECTION 2.0, Paragraph 1.6 and SECTION 5.0, Paragraph 2.2.1.2, for additional information.

2.2.2 RELIABILITY

The following reliability levels are recommended for use (refer to Volume 1, SECTION 2.0, Paragraphs 3.3.2 and 4.2.2):

- Highways of National Significance (HNS)
  - HNS Trunk (Interstate): 95%
  - HNS Branch (Principal Arterial): 85%
- Non-Federal Aid (Minor Arterial, Collector) 75%

2.2.3 OVERALL STANDARD DEVIATION

Unless available project specific information suggests otherwise, use

$S_o = 0.40$

2.2.4 DESIGN SERVICEABILITY LOSS

Two decisions are required, selection of an initial PSI (p_o) and terminal PSI (p_t). The following are suggested:

	p_o	p_t	ΔPSI
• Rural Interstate	4.5	3.0	1.5
• Urban Interstate (minimal construction constraints)	4.5	3.0	1.5
• Urban Interstate (added HOV lanes, major construction constraints)	4.2	3.0	1.2
• Other ACP/PCC Surfaced Pavements	4.5	3.0	1.5
• BST Surfaced Flexible Pavements	4.0	3.0	1.0

A terminal PSI level of 3.0 is based, in part, on the original pavement serviceability-performance data reported by Carey and Irick [6.7]. This information was summarized in SECTION 3.0, Paragraph 3.1.1.

2.2.5 PCC ELASTIC MODULUS

For design purposes (unless project specific information suggests otherwise), use

E_c = 4,000,000 psi

2.2.6 MODULUS OF RUPTURE

The S_c' value typically ranges between 650 to 800 psi. "Local" data obtained in accordance with AASHTO T79 or ASTM C78 should be used if available.

2.2.7 LOAD TRANSFER COEFFICIENT

Unless performance information indicates otherwise, the following J factors are suggested:

Condition	J Factor
• Undoweled PCC on ACB with widened outside lane and drainable shoulder material	3.4
• Undoweled PCC on crushed surfacing with widened outside lane and drainable shoulder material	3.5-3.7
• Undoweled PCC on crushed surfacing (similar to WSDOT designs prior to 1994)	3.8
• Doweled PCC on crushed surfacing orACB	3.2
• Doweled PCC on ATPB with widened outside lane and drainable ACP surfaced shoulders	2.7
• Doweled PCC on ATPB (without widened outside lane) and tied PCC shoulders	2.7

2.2.8 DRAINAGE COEFFICIENT

If no other project specific information is available, use a C_d = 1.0 for PCC sections <u>without ATPB</u> and C_d = 1.2 for PCC sections <u>with ATPB</u>. Other values are certainly possible.

2.2.9 EFFECTIVE MODULUS OF SUBGRADE REACTION

The k_{eff} is a function of
- Roadbed M_R,
- Subbase E (or M_R),
- Subbase thickness,
- Loss of support (LS)

Numerous combinations are possible. A few examples for a 10 in. PCC slab follow (no seasonal variation is assumed for these examples):

Condition	Roadbed M_R (psi)	Subbase) M_R (psi)	Subbase Thickness (in.)	Loss of Support	k_{eff}*
	5,000	30,000	6.0	1.0	100
	5,000	30,000	6.0	2.0	40
Granular Subbase	10,000	30,000	6.0	1.0	160
	10,000	30,000	6.0	2.0	50
	20,000	30,000	6.0	1.0	260
	20,000	30,000	6.0	2.0	80
	5,000	100,000	4.0	0.0	310
ATPB Subbase	10,000	100,000	4.0	0.0	570
	20,000	100,000	4.0	0.0	1040
ACB Subbase	10,000	500,000	4.2	0.0	700
	10,000	500,000	4.2	1.0	210

*rounded to nearest 10 (and calculated using DARWIN Release 2.01)

A large change in k_{eff} has a rather modest impact on PCC slab thickness. For example, calculate the slab thicknesses for the following conditions:

- R = 95%
- J = 3.2
- ΔPSI = 1.5
- S_0 = 0.40
- E_c = 4,500,000 psi
- S_c' = 750 psi
- C_d = 1.0

- ESALs = 5,000,000
  10,000,000
  25,000,000
  50,000,000

- k_{eff} = 50 pci
  200 pci
  800 pci

The corresponding slab thicknesses are to the nearest 0.1 inch

	k_{eff} (pci)		
ESALs	50	200	800
5,000,000	9.9"	9.3"	8.4"
10,000,000	10.9"	10.4"	9.5"
25,000,000	12.5"	12.0"	11.2"
50,000,000	13.9"	13.4"	12.5"

(calculated by use of DARWIN Release 2.01)

Thus, changing k_{eff} from 50 to 200 pci decreases the slab thickness about 0.6 in., from 200 to 800 pci about 1.0 in., and from 50 to 800 pci about 1.6 in. (naturally, these slab thicknesses as a function of k apply to this example only).

2.3 DESIGN EXAMPLE

(Use the example problem originally shown in Paragraph 1.8, Section 5.0, of these notes): Design a rigid pavement structure for a highway in northwestern Washington (mountainous terrain) which primarily supports log hauling trucks. Use only portland cement concrete and granular subbase on subgrade (three layers). The design inputs should not be considered "typical" for WSDOT conditions.

2.3.1 GIVEN (OR KNOWN OR SELECTED)

(a) Pavement location: Rural

(b) Pavement functional classification: Local

(c) Performance period: 20 years (use same as flexible pavement, Paragraph 2.3.2.1, SECTION 5.0)

(d) Materials

(i) PCC: E_{PCC} = 4,000,000 psi
(ii) Subbase: E_{SB} = 28,000 psi (unstabilized crushed surfacing)
(iii) Subgrade: Roadbed resilient modulus = 10,000 psi

(e) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(f) Reliability: Use two levels — 50%
75%

(e) Overall standard deviation: 0.40

2.3.2 RECALCULATE ESALS

Recall, the original ESAL estimate was a log truck haul. Since the LEFs vary a bit, redo the calculation originally done in Paragraph 1.5.4, SECTION 2.0.

Total equivalent damage by the logging truck is (use p_t = 3.0, slab thickness = 8.0 in.):

- Steering axle @ 14,000 lb = 0.36 ESAL
- Drive axle @ 34,000 lb = 1.80
- Pole axle @ 30,000 lb = 1.12

Total = 3.28 ESAL

As before, if the pavement is subjected to 100 of these trucks each day (in one direction) for 20 years (5 days per week), the total ESALs would be:

(5 day/7 day)(365 days/year)(20 years)(100 tk/day)(3.28 ESAL/tk)
= 1,710,286 ≃ 1,700,000 ESALs

2.3.3 SOLUTION FOR RELIABILITY = 50%

(a) Solve nomograph for composite modulus of subgrade reaction (Figure 3.3, p. II-39, 1993 *AASHTO Guide*).

- Use subbase thickness = 6 in. (assumed)
- Road soil resilient modulus = 10,000 psi (given)
- Subbase elastic modulus = 28,000 psi (given)
- $k_\infty \cong 550$ pci (from AASHTO Figure 3.3)

Further, assume there are no subgrade seasonal changes or bedrock within 10 ft of the pavement surface; however, loss of subbase support is estimated to be 2.0 (from Table 2.7, p. II-27, 1993 *AASHTO Guide*).

- k ≅ 50 pci (from Figure 3.6, p. II-42, 1993 *AASHTO Guide*).

(b) Solve nomograph for PCC slab thickness (Figure 3.7, p. II-45-46, 1993 *AASHTO Guide*).

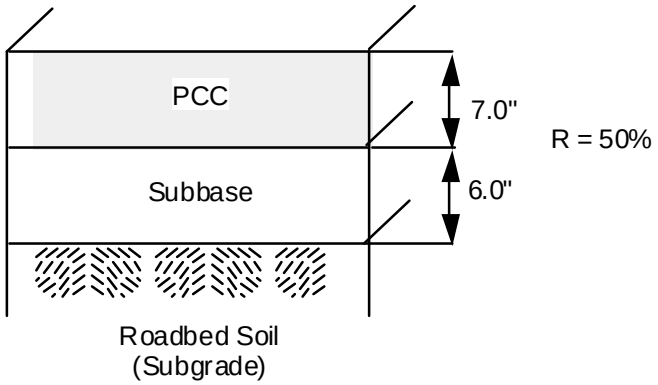
- Inputs

  - k = 50 pci
  - E_c = 4,000,000 psi
  - S_c' = 700 psi
  - J = 3.2 (assume use of dowel bars)
  - C_d = 1.0
  - ΔPSI = 4.5 - 3.0 = 1.5
  - R = 50%
  - S_0 = 0.40
  - ESALs = 1,700,000

- Solution

  PCC slab thickness = 6.6 in. ≃ 7.0 in. (rounding to nearest higher 1/2")

(c) Cross Section



(d) Check for frost design considerations.

2.3.4 SOLUTION FOR RELIABILITY = 75%

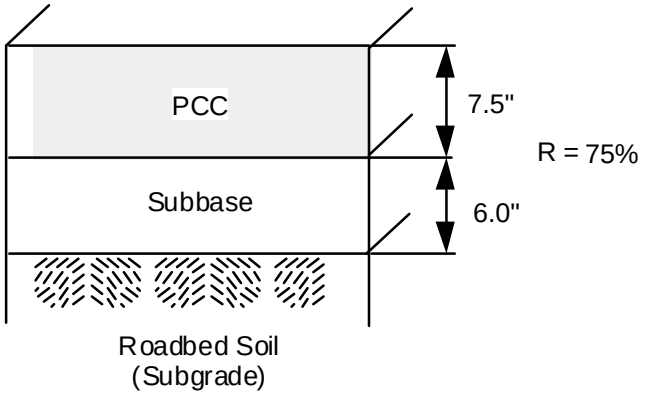
(a) Known

All inputs the same as shown in Paragraph 2.3.3, except R = 75%.

(b) Solution (from Figure 3.7, p. II-45-46, 1993 *AASHTO Guide*)

PCC slab thickness = 7.4 ≃ 7.5 in. (rounding to nearest higher 1/2")

(c) Cross Section



(d) Check for frost design considerations.

2.3.5 TRANSVERSE JOINTS

- Spacing = 12 ft (refer to Paragraph 3.3.2, p. II-49, 1993 *AASHTO Guide*). Thus, does not exceed twice the slab thickness
- Dowels: 1" x 18" spaced every 12" c-c (from Paragraph 2.4.2, p. II-25, 1993 *AASHTO Guide*).

2.4 RIGID DESIGN CASES

The design inputs used in the following four cases do not necessarily reflect actual WSDOT design inputs. The cases are intended to illustrate the AASHTO design process.

2.4.1 RIGID DESIGN CASE NO. 1

Design new rigid pavements (various conditions) for I-5 in the vicinity of N.E. 175th Street (Seattle — MP 176.35 to 177.35) for the following conditions (four lanes each direction):

(a) Pavement location: urban

(b) Pavement functional classification: Interstate

(c) Design period: 20, 30 and 40 years

(d) Construction Materials

(i) PCC E_c = 4,500,000 psi

(ii) PCC S_c' = 750 psi

(iii) Subbase (ACB Class E): 500,000 psi @ 68°F (0.33')

(iv) Subgrade: 15,000 psi (dry months)
12,500 psi (wet months)

(e) Load transfer (J): use J = 3.2

(f) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(g) Reliability: use two levels R = 95%
R = 99%

(h) Overall standard deviation (S_o): use two levels 0.30
0.40

(i) Traffic: Refer to Section 5.0, Paragraph 2.4.1 for traffic information.

(j) Let the drainage coefficient (C_d) = 1.0

(k) Loss of Support = 1.0

(l) Results

(i) ESALs

Use ESALs for the three design periods from Section 5.0, Paragraph 2.4.1. Adjust flexible pavement ESALs to rigid pavement ESALs by

multiplying by 1.5. This is an approximate approach as described in SECTION 2.0, Paragraph 1.6.3.

- 20 years: (21,000,000) (1.5) ≃ 32,000,000
- 30 years: (35,000,000) (1.5) ≃ 52,000,000
- 40 years: (52,000,000) (1.5) ≃ 78,000,000

(ii) Effective Modulus of Subgrade Reaction

Month	Roadbed M_R, psi)	Subbase M_R, psi	Composite k, pci	Relative Damage (u_r)*
Jan	12,500	500,000	950	95
Feb	12,500	500,000	950	95
Mar	12,500	500,000	950	95
Apr	15,000	500,000	1000	92
May	15,000	500,000	1000	92
Jun	15,000	500,000	1000	92
July	15,000	500,000	1000	92
Aug	15,000	500,000	1000	92
Sept	15,000	500,000	1000	92
Oct	15,000	500,000	1000	92
Nov	12,500	500,000	950	95
Dec	12,500	500,000	950	95

Note 1: Subgrade depth assumed to be infinite (no rigid layer)
Note 2: Projected slab thickness = 11"
$\bar{u}_r$ = 93, thus composite k ≃ 980 pci
If LS = 1.0, then effective k ≃ 250 pci
If LS = 0.0, then effective k ≃ 1000 psi
Use k ≅ 250 pci

(k's and u_r's estimated by graphical techniques, AASHTO Figures 3.3, 3.5, and 3.6)

(iii) Summary of Results[1, 2, 3] (only for S_0 = 0.40)

Design Period ESALs	Pavement Layer	Reliability	
		95% (S_0 = 0.40)	99% (S_0 = 0.40)
32,000,000 (20 years)	PCC	12.3" (315 mm)	13.6" (345 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
52,000,000 (30 years)	PCC	13.3" (340 mm)	14.7" (375 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
78,000,000 (40 years)	PCC	14.2" (360 mm)	15.6" (395 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)

[1]Standard or "fixed" layer thicknesses include:
- ACB = 4.0" (100 mm)
- CS = 5.4" (135 mm)

[2]Slab thicknesses calculated by use of DARWIN Release 2.01 and rounded up to nearest 0.1"

[3]Frost design requirements must be checked.

2.4.2 RIGID DESIGN CASE NO. 2

(a) Use the same basic conditions as noted for Case No. 1 (I-5) <u>except</u> use an asphalt treated permeable base (ATPB) beneath the PCC slab. Assume that the underlying CS will serve as an adequate separator layer between the ATPB and subgrade soil.

(b) Effective Modulus of Subgrade Reaction

Month	Roadbed M_R, psi)	Subbase M_R, psi	Composite k, pci	Relative Drainage (u_r)*
Jan	12,500	100,000	810	110
Feb	12,500	100,000	810	110
Mar	12,500	100,000	810	110
Apr	15,000	100,000	900	105
May	15,000	100,000	900	105
Jun	15,000	100,000	900	105

Month	Roadbed M_R, psi)	Subbase M_R, psi	Composite k, pci	Relative Drainage (u_r)*
July	15,000	100,000	900	105
Aug	15,000	100,000	900	105
Sept	15,000	100,000	900	105
Oct	15,000	100,000	900	105
Nov	12,500	100,000	810	110
Dec	12,500	100,000	810	110

$\bar{u}_r$ = 107, thus composite k $\simeq$ 850 pci
If LS = 0.0, then effective k $\simeq$ 900 pci
(k's and u_r's estimated by graphical techniques, AASHTO Figures 3.3, 3.5, and 3.6)

(iii) Summary of Results[1, 2, 3] (only for S_0 = 0.40)

Design Period ESALs	Pavement Layer	Reliability	
		95% (S_0 = 0.40)	99% (S_0 = 0.40)
32,000,000 (20 years)	PCC	9.4" (240 mm)	10.6" (270 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
52,000,000 (30 years)	PCC	10.3" (260 mm)	11.5" (290 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
78,000,000 (40 years)	PCC	11.1" (280 mm)	12.3" (310 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)

[1]Standard or "fixed" layer thicknesses include: ATPB = 4.0" (100 mm) and CS = 5.4" (135 mm)

[2]Other design inputs used which differ from Case No. 1:
- C_d = 1.20
- J = 2.7 (dowels with tied shoulders and ATPB)

[3]Slab thicknesses calculated by use of DARWIN Release 2.01 and rounded up to nearest 0.1"

2.4.3 RIGID DESIGN CASE NO. 3

Design new rigid pavements (various conditions) for I-90 (Spokane — MP 278 to 279) for the following conditions (three lanes each direction):

(a) Pavement location: urban

(b) Pavement functional classification: Interstate

(c) Design period: 20, 30 and 40 years

(d) Construction Materials

(i) PCC E_c = 4,000,000 psi

(ii) PCC S_c' = 700 psi

(iii) Subbase (ACB Class E): 600,000 psi @ 68°F (0.33')

(iv) Subgrade: 20,000 psi (dry months)
15,000 psi (wet months)

(e) Load transfer (J): use J = 3.2

(f) Overall serviceability loss: $p_o - p_t = 4.5 - 3.0 = 1.5$

(g) Reliability: use two levels R = 95%
R = 99%

(h) Overall standard deviation (S_o): use two levels 0.30
0.40

(i) Traffic: Refer to Section 5.0, Paragraph 2.4.3 for traffic information.

(j) Drainage coefficient (C_d) = 1.0

(k) Loss of Support = 0.5

(l) Results

(i) ESALs

Use ESALs for the three design periods from SECTION 5.0, Paragraph 2.4.3. Adjust flexible pavement ESALs to rigid pavement ESALs by multiplying by 1.5. This is an approximate approach as described in SECTION 2.0, Paragraph 1.6.3.

- 20 years: (17,000,000) (1.5) ≅ 26,000,000
- 30 years: (29,000,000) (1.5) ≅ 44,000,000
- 40 years: (45,000,000) (1.5) ≅ 68,000,000

(ii) Effective Modulus of Subgrade Reaction

Month	Roadbed M_R, psi)	Subbase M_R, psi	Composite k, pci	Relative Drainage (u_r)
Jan	15,000	600,000	1000	95
Feb	15,000	600,000	1000	95
Mar	15,000	600,000	1000	95
Apr	15,000	600,000	1000	95
May	20,000	600,000	1400	80
Jun	20,000	600,000	1400	80
July	20,000	600,000	1400	80
Aug	20,000	600,000	1400	80
Sept	20,000	600,000	1400	80
Oct	20,000	600,000	1400	80
Nov	20,000	600,000	1400	80
Dec	15,000	600,000	1000	95

Note 1: Subgrade depth assumed to be infinite (no rigid layer)
Note 2: Projected slab thickness = 11"
$\bar{u}_r$ = 86, thus composite k ≃ 1200 pci
If LS = 1.0, then effective k ≃ 300 pci
If LS = 0.5, then effective k ≃ 600 pci
If LS = 0.0, then effective k ≃ 1250 psi
Use k ≅ 600 pci

(k's and u_r's estimated by graphical techniques, AASHTO Figures 3.3, 3.5, and 3.6)

(iii) Summary of Results[1, 2] (only for S_0 = 0.40)

Design Period ESALs	Pavement Layer	Reliability	
		95% (S_0 = 0.40)	99% (S_0 = 0.40)
26,000,000 (20 years)	PCC	11.8" (300 mm)	13.1" (330 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
44,000,000 (30 years)	PCC	12.9" (325 mm)	14.2" (360 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
68,000,000 (40 years)	PCC	13.8" (350 mm)	15.2" (385 mm)
	ACB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)

[1]Standard or "fixed" layer thicknesses include:
- ACB = 4.0" (100 mm)
- CS = 5.4" (135 mm)

[2]Slab thicknesses calculated by use of DARWIN Release 2.01 and rounded up to nearest 0.1"

(e) Check for frost design considerations.

2.4.4 RIGID DESIGN CASE NO. 4

(a) Use the same basic conditions as noted for Case No. 3 (I-90) <u>except</u> use an ATPB beneath the PCC slab. Assume that the underlying CS will serve as an adequate separator layer between the ATPB and subgrade soil.

(b) Effective Modulus of Subgrade Reaction

Month	Roadbed M_R, psi	Subbase M_R, psi	Composite k, pci	Relative Drainage (u_r)
Jan	15,000	100,000	900	105
Feb	15,000	100,000	900	105
Mar	15,000	100,000	900	105
Apr	15,000	100,000	900	105
May	20,000	100,000	1200	85
Jun	20,000	100,000	1200	85

Month	Roadbed M_R, psi	Subbase M_R, psi	Composite k, pci	Relative Drainage (u_r)
July	20,000	100,000	1200	85
Aug	20,000	100,000	1200	85
Sept	20,000	100,000	1200	85
Oct	20,000	100,000	1200	85
Nov	20,000	100,000	1200	85
Dec	15,000	100,000	900	105

Note 1: Subgrade depth assumed to be infinite (no rigid layer)

Note 2: Projected slab thickness = 11"

$\bar{u}_r$ = 93, thus composite k ≅ 1000 pci

If LS = 0.0, then effective k ≃ 1000 pci

If LS = 0.5, then effective k ≅ 490 pci

If LS = 1.0, then effective k ≅ 260 pci

Use k ≅ 1000 pci.

(k's and u_r's estimated by graphical techniques, AASHTO Figures 3.3, 3.5, and 3.6)

(c) Summary of Results[1, 2, 3] (only for S_0 = 0.40)

Design Period ESALs	Pavement Layer	Reliability 95% (S_0 = 0.40)	Reliability 99% (S_0 = 0.40)
26,000,000 (20 years)	PCC	9.2" (235 mm)	10.4" (265 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
44,000,000 (30 years)	PCC	10.2" (260 mm)	11.4" (290 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)
68,000,000 (40 years)	PCC	11.0" (280 mm)	12.3" (310 mm)
	ATPB	4.0" (100 mm)	4.0" (100 mm)
	CS	5.4" (135 mm)	5.4" (135 mm)

[1]Standard or "fixed" layer thicknesses include:
- ACB = 4.0" (100 mm)
- CS = 5.4" (135 mm)

[2]Other design inputs used which differ from Case No. 3:
- C_d = 1.20
- J = 2.7 (dowels with tied shoulders and ATPB)

[3]Slab thicknesses calculated by use of DARWIN Release 2.01 and rounded up to nearest 0.1"

(d) Check for frost design considerations.

SECTION 6.0

REFERENCES

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SECTION 6.0, APPENDIX 6.1

AN EXAMINATION OF THE "J FACTOR" IN THE 1986 (1993) AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES

INTRODUCTION

The question of the appropriate "J Factor" to use for plain, jointed, portland cement concrete pavements (JPCP) is examined. A J Factor of 3.2 was recommended for "protected corners" (doweled, transverse joints) in the 1981 *AASHTO Design Guide* [4].

The use of the J Factor stems from the use of the Spangler equation to extend the rigid pavement performance equation developed from AASHO Road Test data. To examine the overall issue and ultimately recommend a specific J Factor to use, the development of the original Spangler equation will be reviewed and followed by a discussion of how it was integrated into the AASHTO performance equation (from which the AASHTO thickness design chart was developed).

DEVELOPMENT OF THE SPANGLER EQUATION

The original Spangler equation is as follows [1]:

$$\sigma_c = \frac{3.2P}{h^2}\left(1 - \frac{a_1}{\ell}\right) \qquad \text{Eq. 1}$$

where σ_c = maximum stress in the corner region of a slab, (psi)

P = load applied at the slab corner including an impact increment, (lb)

h = thickness of the slab, (in.)

a_1 = distance from the corner of the slab along the corner bisector to the center of gravity of the wheel load, (in.)

ℓ = radius of relative stiffness, (in.)

$$= \left|\frac{Eh^3}{12(1-\mu^2)k}\right|^{0.25}$$

where E = modulus of elasticity of concrete, (psi)
μ = Poisson's ratio of concrete,
k = modulus of subgrade reaction (psi).

Spangler based this specific equation (Eq. 1) in part on laboratory and field studies in Iowa conducted from about 1929 to 1940. His formula was patterned after the formula developed by Kelley [2].

Kelly (then Chief of the Division of Tests, U.S. Public Roads Administration) based his formula on measured stress observed during the Arlington tests. The Arlington tests were started in 1930 by the Public Roads Administration at the Arlington Experiment Farm, Virginia (now known as the Turner-Fairbank Research Center). These tests were made on a group of 40 concrete pavement slabs, each of which were 20 ft long and 10 ft wide with different cross sections and various types of transverse and longitudinal joints [6,7].

The results from the Arlington tests indicated that the slab stresses at a corner were significantly larger than those calculated by Westergaard's corner load formula (Kelley's formula results in stresses about 25 to 50 percent greater than Westergaard's formula). For reference, both formulas follow:

Kelley's formula [2]:

$$\sigma_c = \frac{3P}{h^2}\left|1-\left(\frac{a_1}{\ell}\right)^{1.2}\right| \qquad \text{Eq. 2}$$

Westergaard's formula [3]:

$$\sigma_c = \frac{3P}{h^2}\left|1-\left(\frac{a_1}{\ell}\right)^{0.6}\right| \qquad \text{Eq. 3}$$

Equation 2 by Kelley was based on stress measurements at the free corners of slabs warped upward due to temperature differentials (a severe condition). Spangler modified the Kelley equation based on the following justification [1]:

> "...since this expression (Kelley's formula) is purely empirical and simplification is both desirable and permissible, the fractional exponent of the quantity a_1/ℓ may be eliminated advantageously and the numerical coefficient of the expression raised to 3.2..."

Thus, the constant "3" (Kelley and Westergaard) became "3.2" primarily to simplify the overall relationship (probably to simplify the use of a sliderule). Further, the Iowa laboratory tests were based on slabs with free corners (no dowels or aggregate interlock). A comparison of the three corner formulas by Spangler, Kelley and Westergaard for AASHTO Parameters [4] is shown in Table 1.

The calculated stresses in Table 1 illustrate that the Spangler and Kelley formulas are in close agreement. Further, both results in significantly higher corner stresses than the Westergaard formula. (This comes as no surprise since Spangler stated that this would be the case.)

Table 1. Comparison of Corner Stresses Calculated by Use of the Spangler, Kelley and Westergaard Formulas

Slab Thickness (in.)	Spangler P(lb)			Kelley P(lb)			Westergaard P(lb)		
	2,500	5,000	10,000	2,500	5,000	10,000	2,500	5,000	10,000
5	211	423	846	218	436	872	143	286	573
6	156	313	626	160	320	640	108	216	432
7	120	241	481	122	244	489	84	169	337
8	95	190	381	96	192	385	68	135	271
9	77	154	309	78	155	311	55	111	222
10	64	128	256	64	128	256	46	93	185
11	54	107	215	54	107	215	39	79	157
12	46	92	183	46	91	182	34	67	135

From Ref. 4 (AASHO Road Test data) using the following parameters:

$k = 60$ pci
$E = 4.2 \times 10^6$ psi
$a_1 = 10$ in.
$\mu = 0.20$

$$\text{hence } \ell = \left| \frac{(E/k)h^3}{12(1 - \mu^2)} \right|^{0.25} = [6076.4\ h^3]^{0.25}$$

SPANGLER EQUATION IN AASHTO PERFORMANCE EQUATION

The basic pre-1986 AASHTO performance equation [5] is:

$$\log W_{t18} = 7.35 \log (D+1) - 0.06 + G_t/\beta \qquad \text{Eq. 4}$$

where $W_{t_{18}}$ = number of 18,000 lb axle loads (ESALs) at end of time t,

D = thickness of slab, (in.)

G_t = a function of the ratio of loss in serviceability at time t to the potential loss taken to a point where $P_t = 1.5$.

$$= \log\left(\frac{4.5 - p_t}{4.5 - 1.5}\right)$$

β = a function of design and load variables that influence the shape of the serviceability vs. load applications (p vs. W) curve

$$= 1.00 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}$$

Based on the above equation (Eq. 4), thickness design charts (and hence pavement designs) would be appropriate only if conditions were similar to those at the AASHO Road Test. Items such as those listed below would need to be similar or identical to the road test pavements [after Ref. 4]:

1. Modulus of elasticity of concrete, (4.2×10^6 psi)
2. Modulus of rupture of concrete, (690 psi)
3. Modulus of subgrade reaction, (60 pci)
4. Environmental conditions in Illinois
5. Life span (only two years at the Road Test)
6. Load transfer characteristics (for unreinforced slabs the transverse joint spacing was 15 ft with dowels and J was taken as equal to 3.2).

To accommodate conditions other than those at the Road Test, the AASHO performance equation was modified using experience and theory. Stresses obtained from Road Test slabs were compared to the stresses calculated by the use of the formulas of Spangler, Westergaard and Pickett. Ultimately, the Spangler equation was chosen to extend the performance equation to other stress conditions; however, the absolute values of stress did vary (measured at Road Test vs. calculated by the Spangler equation). The basic change in the performance equation needed to accommodate the

different stress conditions can be illustrated as follows (derived in Appendix D in Reference 4):

$$\log W_t' = \log W_t + b_{p_t} (\log F' - \log F) \qquad \text{Eq. 5}$$

where W_t' = number of load applications to reach serviceability p_t for pavements with physical properties described by F'

W_t = number of load applications to reach serviceability p_t for a pavement whose physical properties are similar to Road Test pavements

F = S_c/σ for Road Test pavements

F' = S_c'/σ' for physical properties different from those at the Road Test

S_c, S_c' = modulus of rupture, (psi)

σ = pavement stress at Road Test

σ' = pavement stress calculated from Spangler equation, (psi)

b_{p_t} = $4.22 - 0.32\, p_t$

The above modification to the basic performance equation and further simplifying assumptions resulted in the following:

$$\log W_{t18}' = 7.35 \log (D + 1) - 0.6 + \frac{G_t}{1 + \dfrac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32p_t) \log \left| \left(\frac{S_c'J}{S_cJ'}\right)\left(\frac{D^{0.75} - 18.42/Z^{0.25}}{D^{0.75} - 18.42/Z'^{\,0.25}}\right)\right| \qquad \text{Eq. 6}$$

by substituting the following results in Equation 7.

Substitutions:

Z = E/k
E = 4.2×10^6 psi
k = 60 pci
S_c = 690 psi
J = 3.2

$$\log W_{t18}' = 7.35 \log (D + 1) - 0.6 + \frac{G_t}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32p_t) \log \left| \left(\frac{S_c}{215.63J} \right) \left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{Z'^{\,0.25}}} \right) \right| \qquad \text{Eq. 7}$$

Equation 7 was used to develop the thickness design chart shown in Reference 4. In fact, the design equation actually shown at the top of the design chart (Figure III-1, p. 29, Reference 4) was simplified to the following:

$$\log W_{18} = 7.35 \log (D+1) - 0.06 - \frac{0.1761}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ 3.42 \log \left| \frac{f_t}{690} \left(\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{(E/k)^{0.25}}} \right) \right| \qquad \text{Eq. 8}$$

Equation 8 was simplified from Equation 7 with the use of the following:

f_t = S_c/C = working stress
S_c' = 28 day flexural strength of third point loading, (psi)
C = constant to determine design working stress
J' = 3.2
G_t = $\log [0.333 (4.5 - 2.5)] = - 0.1761$
p_t = 2.5

INFLUENCE OF J FACTOR ON PAVEMENT THICKNESS

To further examine the influence of the J Factor on thickness design (or performance) Table 2 was prepared. To develop this table it was assumed that a pavement would have the same k, E, S_c, and p_t as described for the Road Test pavements (or k = 60 pci, E = 4.2 x 10 psi, S_c = 690 psi, p_t = 2.5). Thus, Equation 7 reduces to the following:

$$\log W_{18} = 7.35 \log (D + 1) - 0.6 - \frac{0.1761}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}} + 3.42 \log \left(\frac{3.2}{J} \right) \qquad \text{Eq. 9}$$

In Equation 9, the J' term from Equation 7 is now shown as J. The estimated number of 18,000 lb equivalent axle loads (ESALs) is now only a function of slab thickness and load transfer.

For J Factors ranging from 4.4 to 2.2 (based on the 1986 *AASHTO Design Guide*), the W_{18} were calculated for a fixed 9 in. thick slab (a thickness which is assumed to be typical for existing Washington State PCC pavements). It is recommended in the 1986 *AASHTO Design Guide* that J = 3.2 be used for JPCP or JRCP "...with some type of load transfer device (such as dowel bars)." For CRCP without tied shoulders the AASHTO Guide recommends a J somewhere between 2.9 and 3.2 and 2.6 for CRCP with tied PCC shoulders. Clearly, the proper selection of J for whatever the design case is significant (referring to Table 2 again).

The question now becomes (for typical Washington design parameters) which J Factor level appears reasonable. The data in Table 2 suggest that we should get at least 13,000,000 ESALs for 9 in. PCC pavements if J = 3.2 is appropriate for our doweled slabs; however, a k = 60 pci was used for those calculations (possibly a bit low for Washington conditions). To further examine this and for J = 3.2, k was varied for 9 in. PCC slabs and W_{18} was calculated. The results are shown in Table 3. Table 4 more specifically illustrates how the predicted ESALs change for 9 in. PCC slabs and various J Factor levels. The same treatment was made for 8 in. PCC slabs (Tables 5 and 6) which are typical for slab thicknesses built during the 1960s, such as I-90 (Spokane) and I-405 (Bellevue).

If we have some idea of how many ESALs our 8 and 9 in. PCC pavements have experienced without failure (or equivalent to p_t = 2.5 or higher), then one can decide what J Factor was appropriate for our past conditions. This is discussed more fully in SECTION 6.0, Paragraphs 2.1.7 and 2.2.7.

SUMMARY

This appendix hopefully has illustrated how the Spangler equation was used to expand the Road Test performance equation and how different "J Factors" and subgrade support values affect the predicted ESALs. The predicted ESALs reported were made a function of 8 and 9 in. thick PCC slabs (typical, prior construction for Washington). One can compare actual and predicted ESAL values to provide insight on appropriate load transfer values (J Factor) to use with the AASHTO rigid pavement design procedure.

Table 2. Influence of J Factor on W_{18} for a 9 in. Thick PCC Pavement

J	W_{18} (ESALs)	Percent Change (J = 3.2 baseline)
4.4	4,469,970	-66%
4.2	5,240,830	-61%
4.0	6,192,520	-53%
3.8	7,379,940	-44%
3.6	8,878,880	-33%
3.4	10,795,800	-19%
3.2	13,283,120	0%
3.0	16,563,730	+25%
2.8	20,971,630	+58%
2.6	27,021,160	+103%
2.4	35,529,600	+167%
2.2	47,843,970	+260%

Note: W_{18} is based on the following equation:

$$\log W_{18} = 7.1233 + 3.42 \log\left(\frac{3.2}{J}\right)$$

Table 3. Influence of k on W_{18} for a 9 in. Thick PCC Pavement

k (pci)	W_{18} (ESALs)	Percent Change (k = 60 pci baseline)
60	13,283,120	0%
100	15,168,810	+14%
150	17,141,200	+29%
200	18,899,580	+42%
250	20,533,010	+55%

Note: W_{18} is based on the following equation:

$$\log W_{18} = 7.1233 + 3.42 \log \left(\frac{4.0642}{5.1962 - \frac{0.4069}{(1/k)^{0.25}}} \right)$$

Table 4. Influence of J Factor on W_{18} for a 9 in. Thick PCC Pavement (k = 200 pci)

J	W_{18} (ESALs)	Percent Change (J = 3.2 baseline)
4.4	6,359,870	-66%
4.2	7,456,650	-61%
4.0	8,810,720	-53%
3.8	10,500,180	-44%
3.6	12,632,870	-33%
3.4	15,360,260	-19%
3.2	18,899,580	0%
3.0	23,566,880	+25%
2.8	29,838,440	+58%
2.6	38,445,700	+103%
2.4	50,551,510	+167%
2.2	68,072,400	+260%

Note: W_{18} is based on the following equation:

$$\log W_{18} = 7.1233 + 3.42 \log \left| \left(\frac{3.2}{J}\right)\left(\frac{4.0642}{5.1962 - \frac{0.4069}{(1/k)^{0.25}}}\right) \right|$$

$$= 7.1233 + 3.42 \log \left(\frac{3.55}{J}\right)$$

Table 5. Influence of J Factor on W_{18} for an 8 in. Thick PCC Pavement (k = 60 pci)

J	W_{18} (ESALs)	Percent Change (J = 3.2 baseline)
4.4	2,117,900	-66%
4.2	2,483,140	-61%
4.0	2,934,050	-53%
3.8	3,496,660	-44%
3.6	4,206,860	-33%
3.4	5,115,110	-19%
3.2	6,293,610	0%
3.0	7,847,990	+25%
2.8	9,936,480	+58%
2.6	12,802,770	+103%
2.4	16,834,120	+167%
2.2	22,668,740	+260%

Note: W_{18} is based on the following equation:

$$\log W_{18} = 6.7989 + 3.42 \log \left| \left(\frac{3.2}{J}\right)\left(\frac{3.6248}{4.7568 - \frac{0.4089}{(1/k)^{0.25}}}\right) \right|$$

$$= 6.7989 + 3.42 \log \left(\frac{3.2}{J}\right)$$

Table 6. Influence of J Factor on W_{18} for an 8 in. Thick PCC Pavement (k = 200 pci)

J	W_{18} (ESALs)	Percent Change (J = 3.2 baseline)
4.4	3,138,450	-66%
4.2	3,679,680	-61%
4.0	4,347,880	-53%
3.8	5,181,590	-44%
3.6	6,234,020	-33%
3.4	7,579,930	-19%
3.2	9,326,320	0%
3.0	11,629,690	+25%
2.8	14,724,570	+58%
2.6	18,972,050	+103%
2.4	24,945,980	+167%
2.2	33,592,210	+260%

Note: W_{18} is based on the following equation:

$$\log W_{18} = 6.7989 + 3.42 \log \left| \left(\frac{3.2}{J}\right)\left(\frac{3.6248}{4.7568 - \frac{0.4069}{(1/k)^{0.25}}}\right) \right|$$

$$= 6.7989 + 3.42 \log \left(\frac{3.59}{J}\right)$$

APPENDIX 6.1

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SECTION 6.0, APPENDIX 6.2

WSDOT PCC PAVEMENT JOINTS

INTRODUCTION

This appendix provides additional, needed details relative to joints used in PCC pavement (details on joint construction are provided in SECTION 10.0, CONSTRUCTION CONSIDERATIONS, Paragraphs 5.3 and 5.4).

TYPES OF JOINTS

There are four basic joint types which must be considered:

- construction joints,
- longitudinal joints,
- contraction joints, and
- expansion joints.

CONSTRUCTION JOINTS

Construction joints are used where paving is stopped (say at the end of a workday). Due to the use of transverse header boards to form the terminus of paving, the two slab faces at the joint are relatively smooth. Thus, little or no aggregate interlock exists and dowel bars are required as noted in the WSDOT Construction Manual and Standard Plan A-1 (also refer to Figure 1 in this Appendix).

LONGITUDINAL CONTRACTION AND CONSTRUCTION JOINTS

Longitudinal contraction joints are sawed between adjacent lanes that are paved together; longitudinal construction joints are constructed between adjacent lanes that are paved separately. In both, deformed steel bars are used to "tie" the lanes together (No. 5 bars, 30 in. long, spaced 18 in. center-to-center). Refer to WSDOT Standard Plan A-1 and Figure 1 (these notes).

TRANSVERSE EXPANSION JOINTS

Expansion joints are not frequently used on WSDOT mainline pavement projects; however, they are required at the transverse joint separating the PCC pavement and reinforced concrete bridge approach slabs and at utility holes, storm drains, etc. Refer to WSDOT Standard Plans A-1 and A-2 and Figures 1 and 2 (these notes). In Figure 1, an expansion cap and expansion joint filler may be required depending upon temperature conditions when paved.

CONTRACTION JOINTS

The contraction joint spacing used by WSDOT (as of 1993) is a repeating spacing of 9, 10, 14, and 13 ft skewed 2 ft in 12 ft counterclockwise (refer to WSDOT Standard Plan A-1 and Figure 3 (these notes)). As noted in SECTION 6.0, Paragraph 1.4.2.2, WSDOT PCC performance will likely be improved by the use of dowel bars (reduced potential for joint faulting). The use of dowel bars eliminates the need for skewed joints as shown in Figure 3. A reasonable joint spacing when dowels are used is 12 ft; however, contraction joint spacings up to 15 ft can be used. These contraction joint spacings are, in part, based on prior PCCP performance in Washington State and elsewhere and calculations first described in SECTION 4.0. For example:

- Contraction joint spacings of 12 to 15 ft result in lower slab stresses due to thermal gradients (see SECTION 4.0, Paragraphs 6.2.1 and 6.2.2)

- A contraction joint spacing of about 12 ft conforms to the FHWA criterion $L/\ell \leq 5.0$ (L = slab length, ℓ = radius of relative stiffness) for "thinner" slabs (9 in.) on stiff subbases. A spacing of about 15 ft conforms to the same criterion for "thicker" slabs (13 in.) on stiff subbases. Refer to Section 4.0, Paragraph 6.5 for additional background information.

- In general, as discussed in Section 4.0, Paragraph 6.5, annual joint openings should be limited to no more than 0.025 to 0.035 in. to insure long term joint performance. Using Eq. 4.12 and PCC slabs on stabilized base for annual temperature ranges estimated for eastern and western Washington, the resulting joint movements are:

12 ft slab:	Eastern Washington:	0.031 in.
	Western Washington:	0.021 in.
15 ft slab:	Eastern Washington	0.039 in.
	Western Washington	0.026 in.

These estimated openings generally conform with recommended limits.

The minimum dowel bar diameter should be about D/8 where D = slab thickness. Generally, for mainline PCC pavement, a minimum dowel bar diameter is 1.25 in.; other thicknesses are

Slab Thickness (in.)	Dowel Diameter (in.)
9	1.250 (1 1/4)
10	1.250 (1 1/4)
11	1.375 (1 3/8)
12	1.500 (1 1/2)
13	1.625 (1 5/8)
14	1.750 (1 3/4)
15	1.875 (1 7/8)

All dowels are 18 in. long and spaced 12 in. center to center. (Recall that the "D/8 rule" was used at the AASHO Road Test with respect to dowel diameter (refer to SECTION 6.0, Paragraph 1.4.2.1).)

Joints, in general, should intersect with pavement radii at "right angles" where possible. An illustration of this is shown in Figure 3. Naturally, there can be numerous exceptions to this. The principal issue is to preclude unplanned cracking which may negatively influence pavement performance.

PCCP INTERSECTIONS

In 1995, WSDOT began replacing selected flexible pavement intersections with PCCP. These intersections were severely rutted and distressed due to loads from slow moving vehicles and warm temperatures. Statewide, ruts of 50 mm or more occasionally have occurred and resurfacing to restore the intersection to an acceptable level recurred at intervals of eight years or less. Though WSDOT has numerous PCCP intersections, a unique feature was the replacement of existing flexible with rigid pavement only at intersections.

The Eastern Region has been proactive in rehabilitating several urban intersections with PCCP in the Spokane area. A major advantage is that once the PCCP is placed rehabilitation should not be necessary for 40 years. The major disadvantage with PCCP intersections is the initial construction cost; however, these costs appear to be coming down, particularly as contractors become familiar with this type of construction.

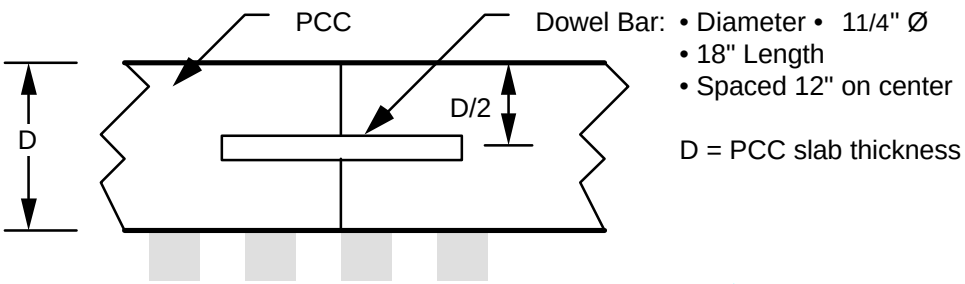
The following is a overview of some considerations for PCCP intersection design:

- The area of pavement rutting or distress must be well defined. The major arterial approach legs to intersections may require PCCP from 60 to 150 meters back of the crosswalk. The approach legs on the minor arterial typically require 60 meters or less but may extend farther. It is desirable to extend the leave legs as far as the adjacent approach legs but, at a minimum, the leave leg should extend at least 30 meters out from the crosswalk.

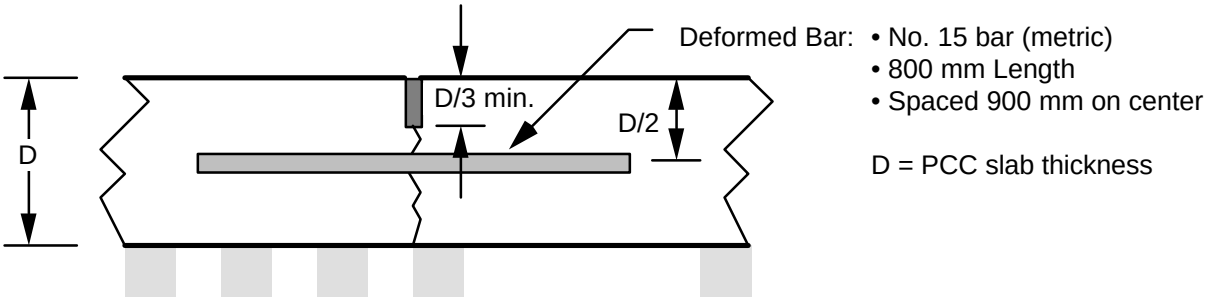
- PCCP intersection plans should include intersection joint layout detail. In 1996,

the American Concrete Pavement Association prepared a useful step by step procedure entitled "Concrete Information - Intersection Joint Layout."

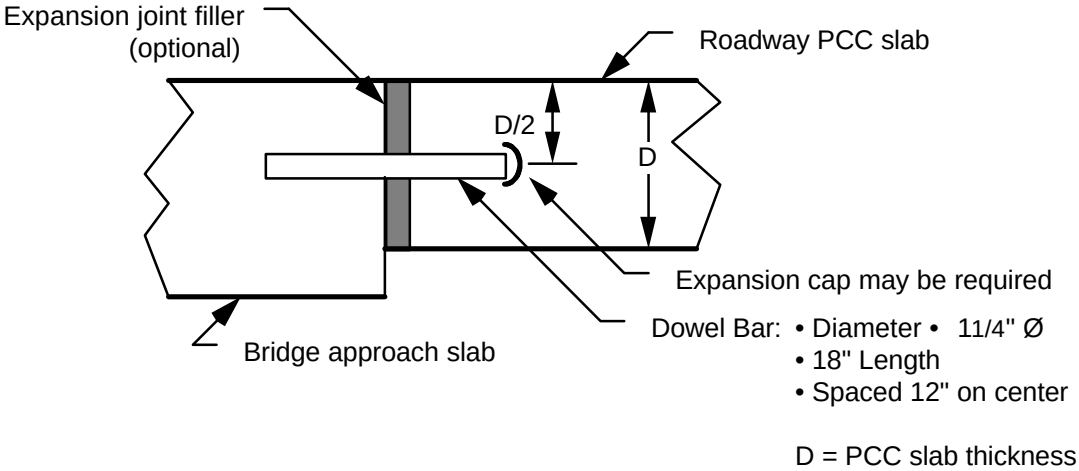
- Transverse joint spacing should be within a range of 3.6 to 4.6 meters.
- Roadway sections should be designed to meet frost penetration criteria.
- 38 mm diameter x 450 mm dowel bars placed at 300 mm centers should be used for all joints with cross traffic. For slabs without cross traffic, use 38 mm diameter x 450 mm dowel bars placed at 300 mm centers on the transverse joints and, for longitudinal joints, No. 5 by 800 mm tie bars placed at 900 mm centers. Dowel bars and tie bars should not be placed within 0.61 meters of new signal detection loops. Tie bars should not be placed within 0.46 meters of dowel bars.
- Provide an isolation joint around all valves and utility holes located within the PCCP unless the valve or utility hole flange is located below the bottom of the concrete slab.
- If a transverse or longitudinal joint is within 1.22 meters of a valve, utility hole, or catch basin, the joint should be skewed to pass through the center of the valve, manhole or catch basin. Care should be taken in the jointing detail to place joints across valves, manholes, or catch basins wherever possible.
- Transverse joints, as much as possible, should line up with existing curb jointing. A lightweight roofing paper should be used between the curbing and the PCCP.
- Prior to opening to traffic, the PCCP should obtain a minimum compressive strength of 17.2 MPa.
- Traffic staging plans should be submitted to WSDOT for approval.



Transverse Construction Joint Cross Section

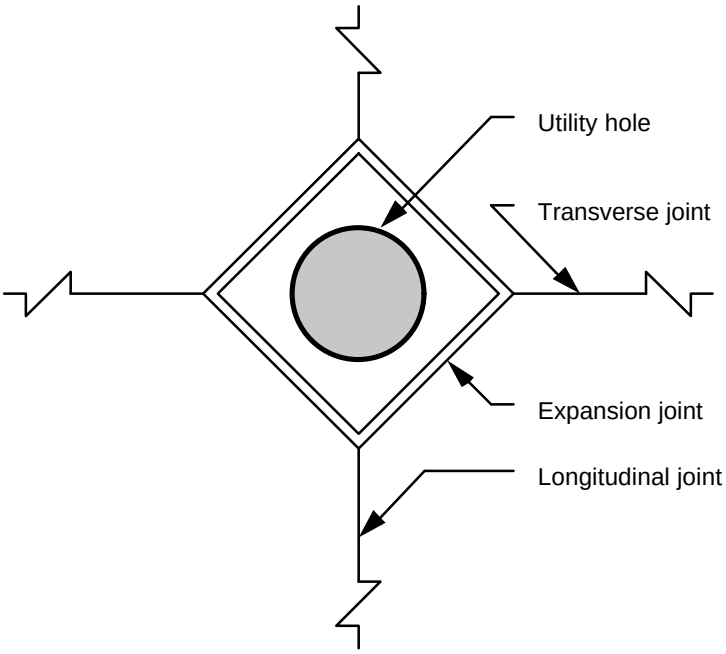


Longitudinal Contraction Joint Cross Section

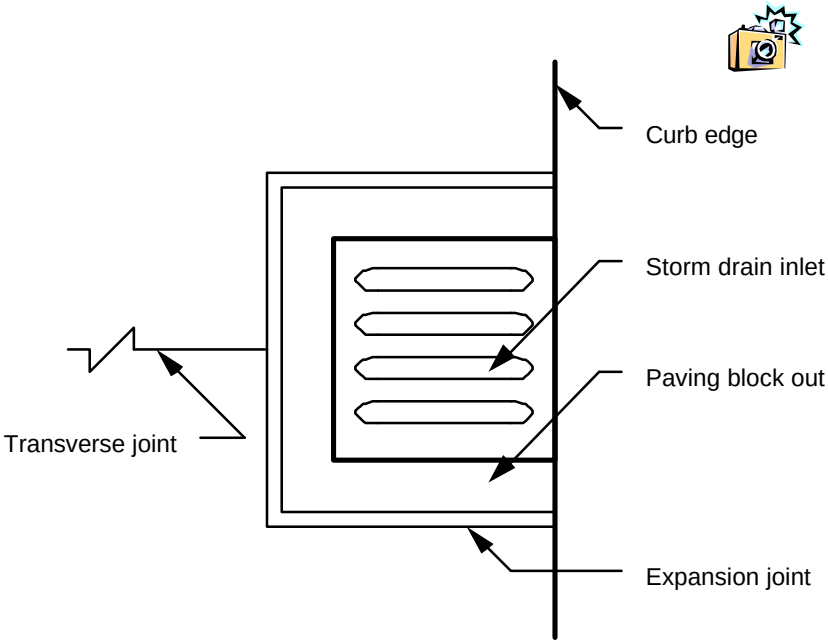


Transverse Expansion Joint Cross Section

Figure 1. Typical PCC Joint Cross Sections

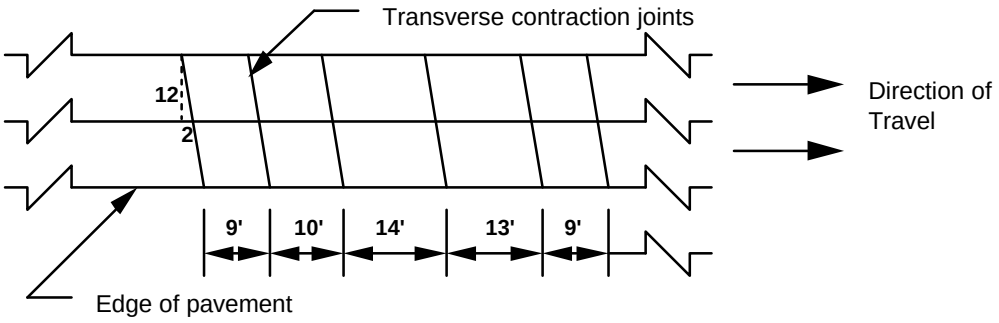


Expansion Joint for Utility Hole on Centerline

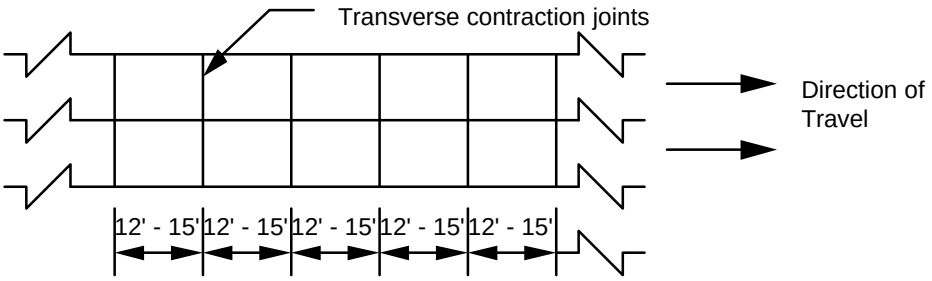


Expansion Joint for Catch Basin

Figure 2. Typical PCC Expansion Joint Layouts



WSDOT Undoweled Transverse Contraction Joints



WSDOT Doweled Transverse Contraction Joints

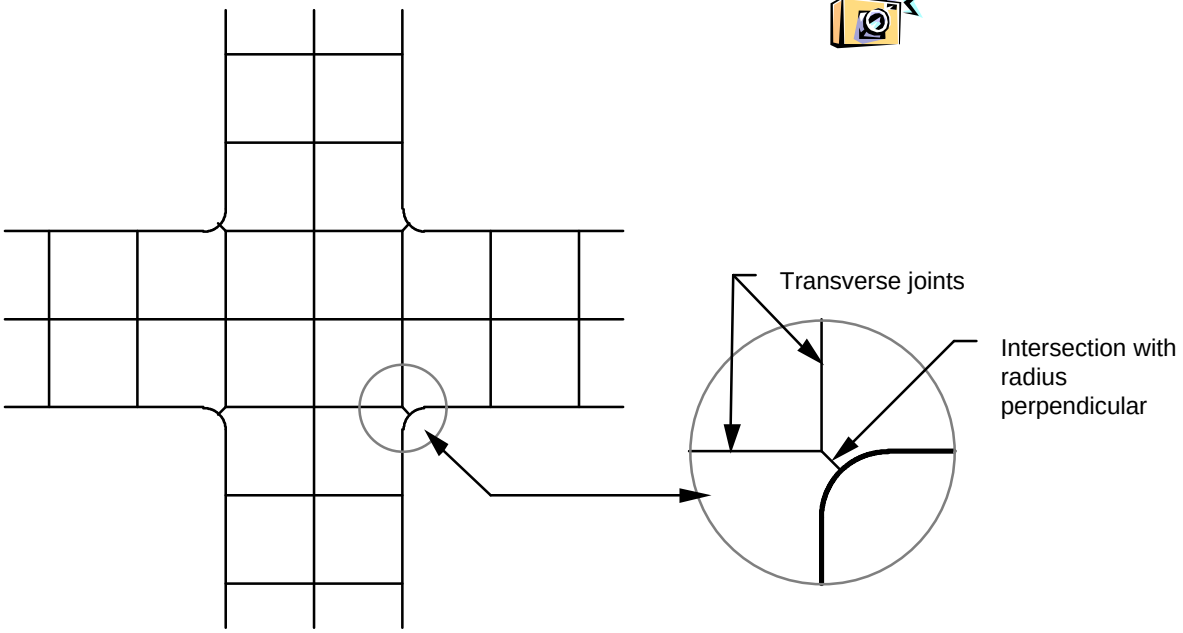


Illustration of Joints Intersecting Radii Perpendicularly

Figure 3. Typical PCC Contraction Joint Layouts

SECTION 7.0

PAVEMENT REHABILITATION

This SECTION is used to describe the basic concepts frequently used in pavement rehabilitation. Several asphalt concrete overlay design procedures are described, with specific emphasis placed on the WSDOT and AASHTO overlay design procedures. The rehabilitation of plain-jointed rigid pavement is described, along with recent examples of WSDOT projects. The SECTION concludes with a description of nondestructive testing interpretation techniques, with specific emphasis on WSDOT uses of such information. U.S. Customary units are primarily used in this SECTION since those are the principal units used in the design procedures which are described.

1. REHABILITATION WITH OVERLAYS

1.1 INTRODUCTION

1.1.1 SCOPE

Overlays are generally used either to improve pavement surface irregularities (such as roughness, studded tire wear, etc.) or increase the pavement structural capacity or both. Both improvements fall under the FHWA term of "resurfacing." Further, overlay design practice often includes two basic engineering design elements:

(a) What is economically sensible?
(b) What does an existing structure need to provide adequate service?

This paragraph will be focused on that part of "resurfacing" which increases the pavement structural capacity or stated another way, "What does a specific pavement structure need to meet the expected service conditions?"

The last portion of this paragraph (subparagraph 1.10) describes a process for thickness design of granular overlays ("cushion courses").

1.2 TYPES OF OVERLAY DESIGN PROCEDURES

The design of asphalt concrete overlays can be broadly classified as follows [modified after Ref. 7.1]:

(a) engineering judgment,

(b) component analysis,

(c) nondestructive testing with limiting deflection criteria, and

(d) mechanistic analysis based on interpretation of nondestructive testing or laboratory data with appropriate failure criteria.

Each of the above classifications will be briefly described.

1.2.1 ENGINEERING JUDGMENT

This classification of overlay design is the most subjective of the four listed and can be heavily influenced by political and budget constraints. Selection of overlay thickness and the associated materials is often based on local knowledge of existing conditions which can result in cost effective solutions; however, local expertise is fragile and subject to retirements, agency reorganizations, etc. Currently, more agencies appear to be relying on quantifiable overlay design approaches but tempered with local expertise.

1.2.2 COMPONENT ANALYSIS

This approach to overlay design essentially requires that the total pavement structure be developed as a new design for the specified service conditions and then compared to the existing pavement structure (taking into account pavement condition, type, and thickness of the pavement layers). A review of current component design procedures quickly reveals that substantial judgment is required to effectively use them. This judgment is mainly associated with selection of "weighting factors" to use in evaluating the structural adequacy of the existing pavement layers (i.e., each layer of the pavement structure is assigned a layer coefficient often on the basis of experience).

The Asphalt Institute procedure will be used to illustrate asphalt concrete overlays on existing flexible pavements using the component analysis method.

1.2.3 NONDESTRUCTIVE TESTING WITH LIMITING DEFLECTION CRITERIA

Nondestructive testing of pavement structures has been used extensively for at least 30 years; however, it mostly was used as a research tool. The term "nondestructive testing" generally translates to measurement of pavement surface deflections.

At the present time, the objective of deflection testing is to measure the structural properties of the pavement. This is done by imposing a known load on the pavement and measuring its response (i.e., surface deflections). Thus, an overall or effective strength is measured which combines all influencing factors such as material properties (including subgrade), thickness of pavement layers, and environmental effects. Most currently used deflection based overlay design procedures do not attempt to isolate material properties of individual pavement layers.

The dominant type of measurement used for nondestructive overlay design procedures is surface deflection (or deflection basins) obtained with known load conditions (i.e., contact pressure, force, and time of loading). Each of these factors can influence the pavement response to loading.

A widely used deflection based overlay design procedure is The Asphalt Institute method [7.2], which is described in Section 1.4.

1.2.4 MECHANISTIC ANALYSIS

Significant interest has developed in the use of mechanistic overlay design procedures. The term "mechanistic" as defined in most dictionaries (such as *Webster's Seventh New Collegiate*) is "mechanically determined" or "relating to the doctrine of mechanism." In turn, "mechanism" is defined as "the fundamental physical...processes involved in or responsible for an action, reaction, or other natural phenomenon." This roughly translates to pavement engineers as determining the fundamental stresses, strains, and deflections caused by traffic and/or the environment in pavement structures. Knowledge of these stresses, strains or deflections can in turn be used with limiting criteria to evaluate not only the need for an overlay but remaining pavement life as well. Section 1.6 will be used to further illustrate this concept.

1.3 COMPONENT ANALYSIS — ASPHALT INSTITUTE

This component analysis design approach (termed "effective thickness" by the Asphalt Institute) uses relationships between subgrade strength, pavement structure, and traffic [7.2]. The existing structural integrity of the pavement is converted to an equivalent thickness of asphalt concrete which is then compared to that required for a new design. The structural evaluation procedure developed by the Asphalt Institute allows for determining the required thickness of asphalt concrete overlay or to estimate the length of time until an overlay is required.

The essential parts of this overlay design procedure will be briefly described:

(a) subgrade analysis,
(b) pavement structure thickness analysis,
(c) traffic analysis, and
(d) example.

1.3.1 SUBGRADE ANALYSIS

Testing of the subgrade materials is encouraged even if original design records are available. The resilient modulus (M_r), soaked CBR or R-value tests appear to be the easiest to use with this procedure. For actual design, the design strength of the subgrade must be characterized in terms of resilient modulus. Associated correlations for CBR and R-value are:

$$M_r \text{ (psi)} = 1500 \text{ (CBR)} \qquad \text{(Eq. 7.1)}$$

$$= 1155 + 555 \text{ (R-value)} \qquad \text{(Eq. 7.2)}^*$$

If test data in terms of M_r , CBR, or R-value are not available, subgrades can be placed into one of three classes for design purposes as follows:

(a) Poor soils. Soft and plastic when wet, generally composed of silts or clays. Typical properties: M_r = 4,500 psi, CBR = 3, R-value = 6.

(b) Medium soils. Include soils such as loams, silty sands, and sand-gravels which contain moderate amounts of clay and silt. These soils can be expected to lose only a moderate amount of strength when wet. Typical properties: M_r = 12,000 psi, CBR = 8, R-value = 20.

(c) Good soils. These soils can be expected to retain a substantial amount of their strength when wet and include clean sands and sand-gravels. Typical properties: M_r = 25,000 psi, CBR = 17, R-value = 43.

Assuming that at least six to eight individual subgrade tests are available, a conservative value is chosen as a function of the design traffic (ESALs). To do this a plot is prepared of the percent equal to or greater than (y axis) versus resilient modulus test results (x axis). Basically, one must create a cumulative frequency plot. Following this, the design subgrade resilient modulus is selected from the plot as follows:

Design ESALs	Design Subgrade Percentile Value (%)
10,000 or less	60
10,000 to 1,000,000	75
greater than 1,000,000	87.5

For more information, refer to The Asphalt Institute'ss MS-1, Chapter V, Paragraph 5.06 (September 1981 Edition).

* Note: R-value results can vary due to differences in test procedures

1.3.2 PAVEMENT STRUCTURE THICKNESS ANALYSIS

The goal of this portion of the design method is to determine the "Effective Thickness (Te)" of the existing pavement structure. The Asphalt Institute has two approaches that can be used; only one will be illustrated here. First, the significant pavement layers are identified and their condition determined. Second, "Conversion Factors" are selected for each layer (judgment by the designer is very important at this point). Third, the Effective Thickness for each layer is determined by multiplying the actual layer thickness by the appropriate Conversion Factor. The Effective Thickness of the complete pavement structure is the sum of the individual Effective Thicknesses. Typical layer thickness Conversion Factors are shown in Table 7.1.

1.3.3 TRAFFIC ANALYSIS

The Asphalt Institute treatment of traffic includes consideration of volume composition, and axle weights, with the goal being to develop the equivalent number of 18,000 lb equivalent single axle loads (ESALs).

Table 7.1 Example of Asphalt Institute Conversion Factors for Estimating Thickness of Existing Pavement Components to Effective Thickness [after Ref. 7.2]

Description of Layer Material	Conversion Factor*
1. Native subgrade	0.0
2. a. Improved subgrade — predominantly granular materials b. Lime modified subgrade of high PI soils	0.0
3. a. Granular subbase or base — CBR not less than 20 b. Cement modified subbases and bases constructed from low PI soils	0.1 - 0.3
4. a. Cement or lime-fly ash bases with pattern cracking b. Emulsified or cutback asphalt surfaces and bases with extensive cracking, rutting, etc. c. PCC pavement broken into small pieces	0.3 - 0.5
5. a. Asphalt concrete surface and base that exhibit extensive cracking	0.5 - 0.7
6. a. Asphalt concrete — generally uncracked b. PCC pavement — stable, undersealed and generally uncracked pavement	0.9 - 1.0
7. Other categories of pavement layers listed in Ref. 7.2	

*Equivalent thickness of new asphalt concrete

To estimate the ESALs for the overlay design period, at least two approaches can be used, depending on availability of site specific traffic information. One approach provides broad traffic classifications and the associated 18,000 lb (80 kN) ESAL amounts, as illustrated in Table 7.2. The second approach includes the use of "truck factors" along with the number and type of trucks that are expected to use the facility. This approach can accommodate a wide variety of truck information ranging from only an estimate of the percent of the Average Daily Traffic (ADT) that constitutes trucks to estimates of trucks broken into the categories of single and multi-units (as illustrated by "vehicle type" in Table 7.3).

The term "truck factor" represents the average 18K ESAL per truck. Truck factors are shown in Table 7.3 for a variety of vehicle types, with the average being 0.4 ESAL per truck averaged over all highway and truck types. Thus, if a given "average" highway is expected to have 1,000,000 trucks during the design period, the resulting ESALs would be 400,000.

Table 7.2. Asphalt Institute Traffic Classifications [after Ref. 7.2]

Type of Street or Highway	Estimated 18,000 lb (80 kN) ESALs
1. Parking lots 2. Light traffic residential streets and farm roads	5,000
1. Residential streets 2. Rural farm and residential roads	10,000
1. Urban and rural minor collectors	100,000
1. Urban minor arterial and light industrial streets 2. Rural major collector and minor arterial highways	1,000,000
1. Urban freeways and other principal arterial highways 2. Rural interstate and other principal arterial highways	3,000,000
1. Some interstate highways 2. Some industrial roads	10,000,000

Table 7.3. Average Truck Factors Compiled from FHWA Data [after Ref. 7.2]

Vehicle Types	Truck Factors				
	Rural Highways			Urban Highways	Combined
	Interstate	Other	All	All	All
1. Single-units					
(a) 2-axle, 4-tire	0.02	0.02	0.03	0.03	0.02
(b) 2-axle, 6-tire	0.19	0.21	0.20	0.26	0.21
(c) 3-axles or more	0.56	0.73	0.67	1.03	0.73
(d) All single-units	0.07	0.07	0.07	0.09	0.07
2. Tractor semi-trailers					
(a) 3-axle	0.51	0.47	0.48	0.47	0.48
(b) 4-axle	0.62	0.83	0.70	0.89	0.73
(c) 5-axles or more	0.94	0.98	0.95	1.02	0.95
(d) All multiple units	0.93	0.97	0.94	1.00	0.95
3. All trucks	0.49	0.31	0.42	0.30	0.40

1.3.4 EXAMPLE OF ASPHALT INSTITUTE PROCEDURE

A two-lane highway has the following characteristics and resulting overlay requirement:

(a) Traffic

- Average Daily Traffic = 4,000
- Percent trucks (total all units) = 10%
- Traffic growth rate = 4%

(b) Existing pavement structure and condition

- Asphalt concrete = 3 in.
- Crushed stone base = 8 in.
- Subgrade design strength value: CBR = 8 or $M_r \cong 12{,}000$ psi
  Refer to Equation 7.1
- Overall, the pavement structure is in poor condition, with asphalt concrete exhibiting well defined crack patterns

(c) Determine overlay thickness for a 20-year period

- 18,000 lb equivalent single axle loads

  - number of trucks in the design lane per day = (4,000)(0.50)(0.10) = 200

  - ESALs/day = 200(0.4 ESAL/truck) = 80 ESALs/day

  - 18K ESAL for design period = (80 ESALs/day) (365 days/yr)(29.8) ≅ 870,000 after adjustment for design period and traffic growth rate

    (Note: $\frac{(1 + g)^n - 1}{g} = \frac{(1 + .04)^{20} - 1}{0.04} = 29.8$)

- Effective pavement thickness

Layer Thickness (in.)		Conversion Factor (Table 7.1)		Effective Thickness (in.)
3	x	0.5	=	1.5
8	x	0.2	=	1.6
		Total Te	=	3.1

- Required new "full-depth" asphalt concrete pavement thickness (Tn) = 7.7 in. (refer to Figure 7.1).

- Thickness of asphalt concrete overlay = Tn - Te = 7.7 - 3.1 in. = 4.6 in.

1.4 ASPHALT INSTITUTE — LIMITING PAVEMENT SURFACE DEFLECTIONS [7.2]

1.4.1 DESCRIPTION

The basic approach of the overlay design procedure is to identify continuous pavement sections of uniform performance, obtain "static" pavement surface deflections with the Benkelman Beam and an 18,000 lb single axle, and determine the expected traffic by use of equivalent axle loads (previously described).

The Asphalt Institute recommends that a minimum of 20 deflection measurements be taken each mile and randomly located in the outer wheelpath. From this data for each "uniform" pavement section, a "representative rebound deflection" is determined as follows:

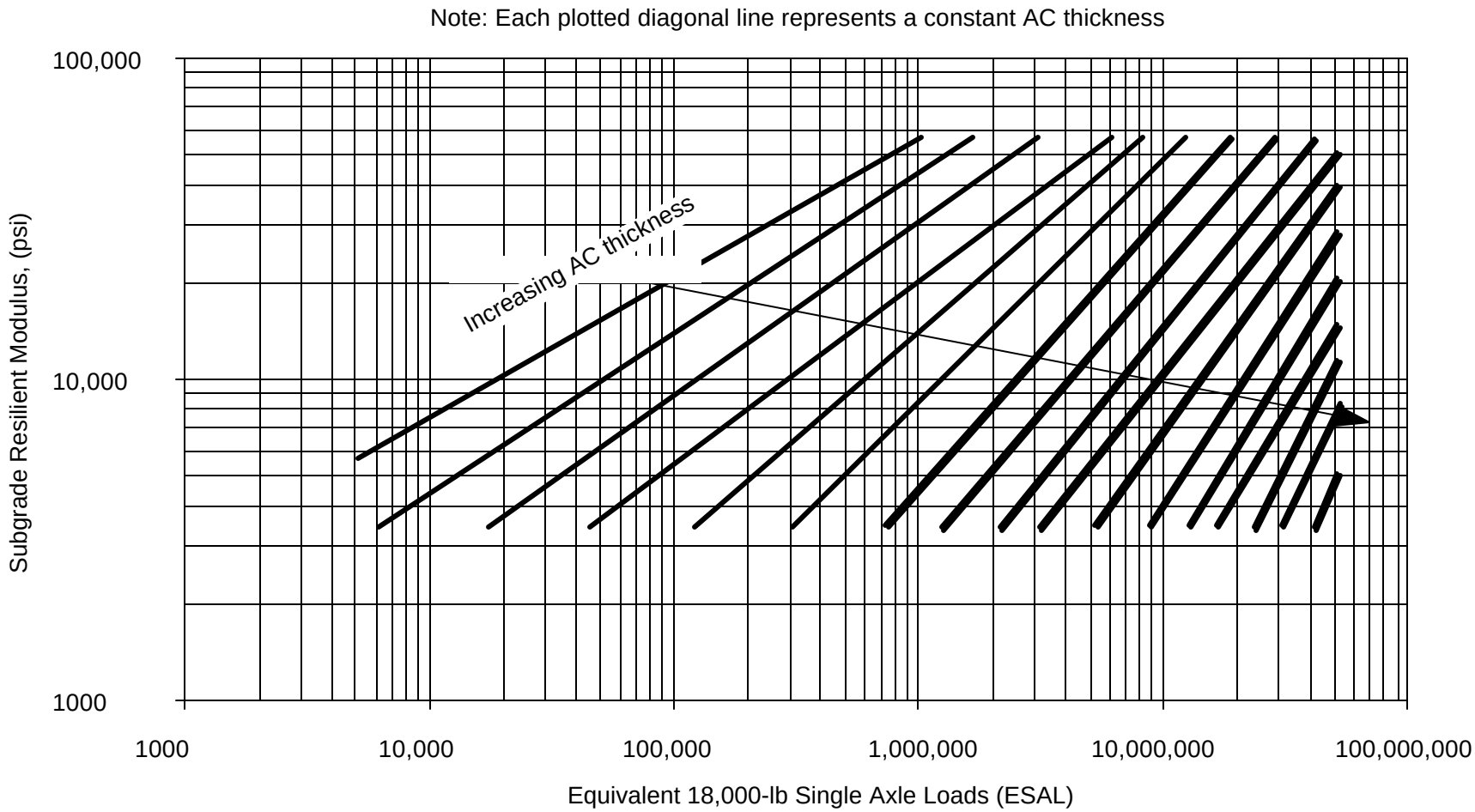


Figure 7.1. Sketch of Asphalt Institute Design Chart for Full-Depth Asphalt Concrete Pavement [Redrawn from 7.2]

$$RRD = (\bar{x} + 2s)(f)(c)$$

where RRD = representative rebound deflection (in),

$\bar{x}$ = mean of the individual deflections (in),

s = standard deviation of the deflections (in),

f = temperature adjustment factor, and

c = critical period adjustment factor (where c = 1 if deflection tests made during the most critical period (highest pavement deflections)).

This calculation of RRD represents an upper bound of about 97 percent of all deflections measured. The temperature adjustment factor adjusts the existing asphalt concrete surfacing to a standard temperature of 70 °F (refer to Figure 7.2).

A study for the U.S. Forest Service completed during 1994 (Uhlmeyer et al. [7.33]) reviewed deflection data from paved roads in the Willamette National Forest (Central Oregon), the Olympic National Forest (both aggregate and asphalt surfaced pavements) and the Kootenai National Forest (western Montana). The critical period adjustment factor (c) ranged from about 1.0 to 1.7 (with the exception of one forest road in the Kootenai which ranged from 3.0 to 8.0). Overall, a c = 1.5 appeared to be widely applicable. Further, if the deflection data are collected for different time periods, the following adjustment factors could be used:

Time Period When Deflectlion Data Obtained	Deflection Data Adjustment Factor
January - March	1.00
April - June	1.25
July - September	1.50
October - December	1.25

The above "c" values are generally representative for flexible pavements located in areas with modest annual freezing and thawing (a Freezing Index of say less than 700 °F-days) or a wet climate.

A higher "c" is likely needed in areas with severe winter freezing and thawing. The Roads and Transportation Association of Canada (RTAC) [7.34] uses a multiplier of 2.5 to convert Benkelman Beam measurements taken during the fall period (September 1 to October 15) to "maximum spring values." This ratio of 2.5 is a bit higher than generally observed in the U.S. possibly reflecting the generally more severe winter and thaw periods.

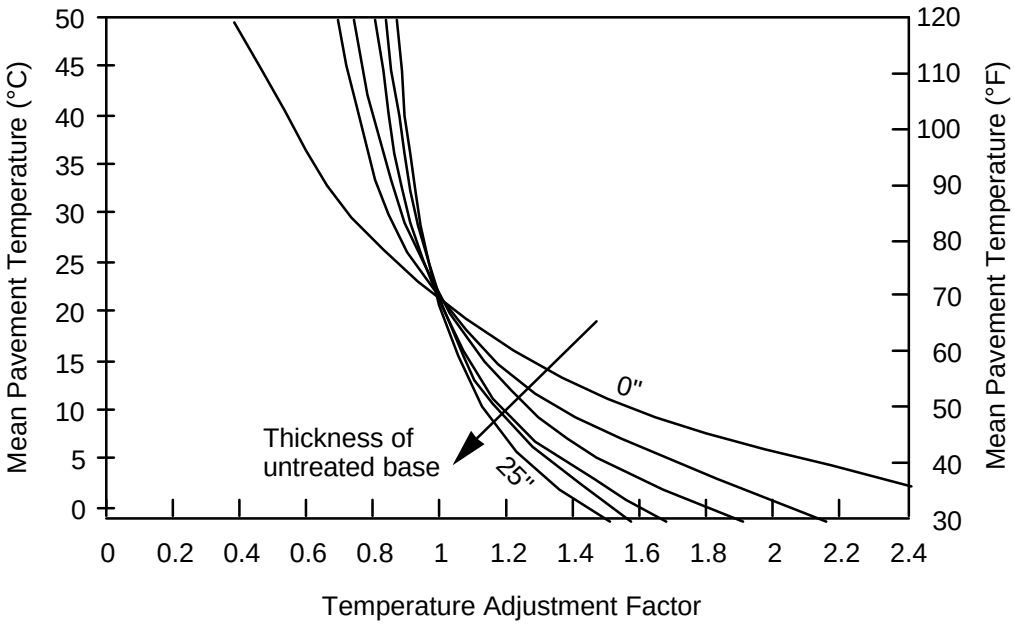


Figure 7.2. Sketch of Asphalt Institute Temperature Adjustment Factors for Benkelmen Beam Deflections [after Ref. 7.2]

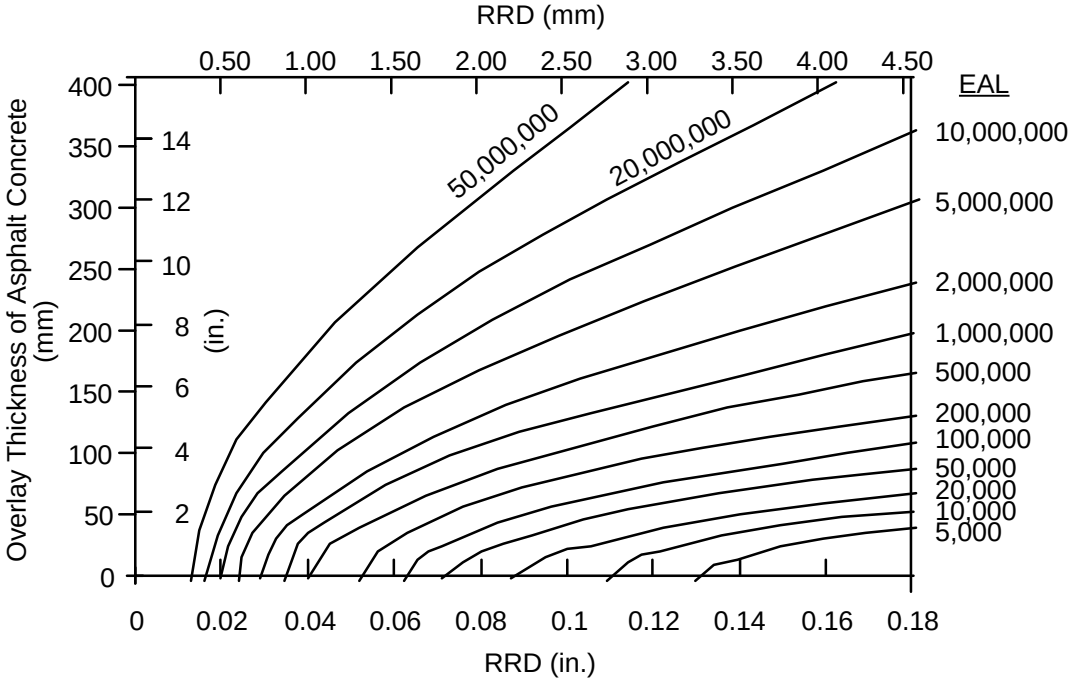


Figure 7.3. Sketch of Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflection from a Measured to a Design Deflection Value [after Ref. 7.2]

The deflection measurements obtained by use of the Benkelman Beam can be used to estimate the remaining life of the pavement or the needed thickness of asphalt concrete overlay. To determine the required overlay thickness, Figure 7.3 is used with the RRD and ESALs as the required input.

1.4.2 EXAMPLE

The previous information contained in the Asphalt Institute example for component analysis will be used.

(a) Traffic

ESALs for design period 870,000

(b) Deflection data

$\bar{x}$ = 0.061 in. s = 0.004 in.

f: If the mean asphalt concrete temperature during deflection = 60 °F, then f = 1.10 (refer to Figure 7.2).

c: If tests made during the critical period, then c = 1.0.

(c) Representative rebound deflection

$RRD = (\bar{x} + 2s)(f)(c)$

$RRD = [0.061 + 2(0.004)](1.10)(1.0)$

$RRD = 0.076$ in

(d) Required asphalt concrete overlay thickness = 3.8 in (refer to Figure 7.3)

1.5 MECHANISTIC

1.5.1 INTRODUCTION

The greatest advantage of mechanistic based overlay design methods is the versatility provided in evaluating different materials under various environments and pavement conditions. The mechanistic procedures provide a basis for rationally modeling pavement systems. As these models improve, better correlations can be expected between design and performance parameters. It is anticipated that these procedures will replace limiting deflection overlay methods, since the latter do not account for subsurface material properties [7.1].

Mechanistic overlay design should, at a minimum, encompass the elements illustrated in Figure 7.4. Selected elements shown in this figure will be separately discussed.

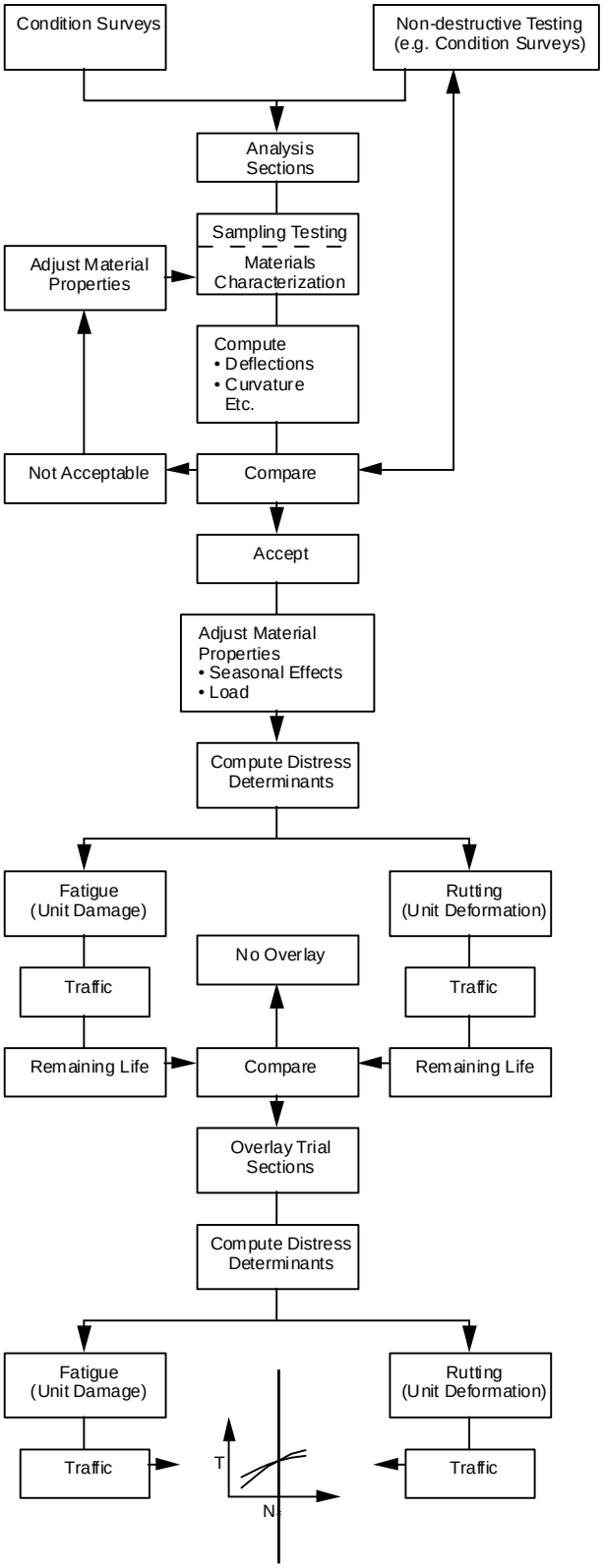


Figure 7.4 Overlay Design Based on Mechanistic analysis [after 7.1]

1.5.2 ANALYSIS SECTIONS

A reasonable amount of uniformity should exist within a given pavement segment being considered for overlay. These sections can be initially identified by use of condition surveys and finally deflection measurements. There exist numerous methods for determining the required number and location of such measurements. A minimum sample generally consists of deflection measurements every 250 to 500 ft. After collection of the deflection measurements, statistical measures can be used to delineate between analysis sections (along with the condition surveys).

1.5.3 LAYER CHARACTERISTICS

The mechanistic approach to overlay design can encompass both material characterization from the laboratory and nondestructive test data collected in the field. Total reliance on either laboratory or field data is generally felt to be inappropriate at the current stage of development. However, recent developments provide for estimates from field data of in situ moduli of the pavement layers. Illustrations of these approaches include

(a) FHWA — Resource International overlay design procedures [7.3],

(b) BISDEF computer program developed by Bush at the Waterways Experiment Station [7.4] (not an overlay design system),

(c) ELMOD computer program by Ullidtz [7.5],

(d) several other analysis procedures which use deflection basins from the Falling Weight Deflectometer, Dynaflect, or Road Rater.

Laboratory testing for mechanistic analysis generally implies the determination of resilient moduli (essentially a "modulus of elasticity" for pavement materials). Standard test methods such as ASTM D4123-82 [7.6] are used for bituminous mixtures and triaxial procedures such as recommended by Kalcheff and Hicks [7.7] can be used for unbound granular materials. Laboratory determined moduli from unbound base, subbase, and subgrade materials are stress-sensitive and as such must be recognized.

The laboratory derived moduli are often "adjusted" by using layered elastic analysis to calculate the resulting maximum deflection or deflection basin for a specified loading condition. In turn, field deflections are compared to the estimated deflections. If differences exist, the laboratory values are modified to reasonably match field measurements. More recently, computer programs such as BISDEF [7.4] can be used to estimate layer moduli for up to four pavement layers. Input data for this program include the NDT load, measured deflection basin, layer thicknesses, and limiting ranges and expected values of moduli for each pavement layer. The program then estimates the moduli for each layer, which results in a best fit of the field deflection basin (within a user specified error range).

1.5.4 LIMITING FAILURE CRITERIA

Pavement sections deteriorate with time due to a progression of defects (due to traffic loads, environment, and other factors acting on the pavement structure). The pavement reaction to its total loading condition can be characterized by estimating the induced stress, strain, and deflections. When these pavement responses reach a cumulative limiting value, distress results. The resulting serviceability loss can occur as the result of the accumulation of a single distress type (often fatigue cracking) or rutting or a combination of several types.

Fatigue related distress can be defined as the phenomenon of load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material [7.8]. The classic type of fatigue failure is commonly described as "alligator" cracking, due to the pattern or cracks that appear on the pavement surface. These cracks appear to be best associated with tensile strains at the bottom of the asphalt concrete layers. A common expression used to relate the number of loads to fatigue failure as a function of tensile strain is:

$$N_f = K_1 \left(\frac{1}{\varepsilon}\right)^{K_2}$$

where N_f = load repetitions to failure,
ε = initial tensile strain, and
K_1, K_2 = fatigue parameters.

The fatigue relationship developed by Majidzadeh and Ilves [7.3] for the FHWA-RII overlay design system is used to illustrate "typical" K_1 and K_2 parameters:

$$N_f = 7.56 \times 10^{-12} \left(\frac{1}{\varepsilon}\right)^{4.68}$$

Analogous criteria have been developed for rutting failure, whereby the number of load repetitions to failure is generally made a function of vertical strain in the subgrade (in lieu of tensile strains as for asphalt concrete fatigue).

In practice, flexible pavements are subjected to a variety of loads. Miner's rule is used for evaluating cumulative damage. The rule states that the condition at failure is given by:

$$\sum_{i=1}^{r} \frac{n_i}{N_i} = 1$$

where n_i = actual number of cycles of stress or strain applied to the pavement,

N_i = allowable number of cycles to failure based on failure criteria (such as fatigue or rutting), and

r = number of loading conditions considered.

1.6 WASHINGTON STATE DEPARTMENT OF TRANSPORTATION — A MECHANISTIC/EMPIRICAL APPROACH

1.6.1 INTRODUCTION

A mechanistic-empirical overlay design procedure (EVERPAVE) was developed by the Washington State Department of Transportation that is based on the backcalculation of material properties and fatigue and rutting failures. In this approach, layer moduli can be calculated for each deflection test point. The asphalt concrete modulus is corrected for temperature according to data for typical Washington mixtures (Figure 7.5). Next, an iterative process is used to determine an appropriate overlay thickness for each deflection test point as shown in Figure 7.6a.

Both the unstabilized base course (subbase) and subgrade moduli can be non-linear with stress state, i.e., the base, subbase, and subgrade layer moduli can take the following form:

$$E = K_1 (\theta)^{K_2} \quad \text{or} \quad E = K_3 (\sigma_d)^{K_4}$$

with the exponents being either positive or negative.

1.6.2 FAILURE CRITERIA

The failure criteria used in EVERPAVE are based on two basic criteria: rutting and AC fatigue cracking (refer to Figure 7.6b for specific locations). The rutting criterion was adopted from the Asphalt Institute [7.9, 7.10]:

$$N_f = \left[\frac{1.05 \times 10^{-2}}{\varepsilon_v} \right]^{4.4843}$$

where N_f = allowable number of 18,000 lb (80 kN) single axles so that rutting at the pavement surface should not exceed 0.5 in. (12.7 mm), and

ε_v = vertical compressive strain at the top of the subgrade layer.

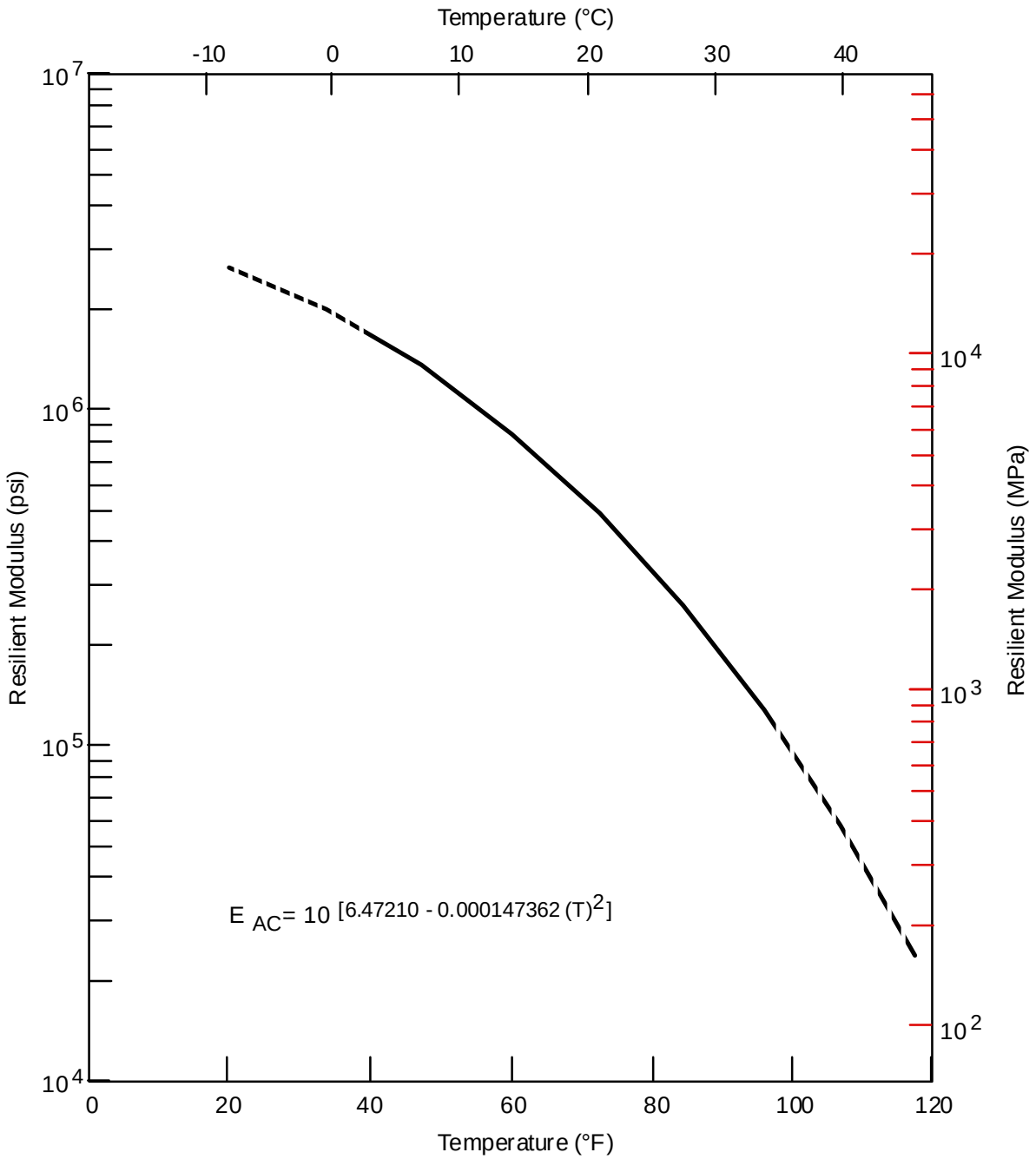


Figure 7.5. General Stiffness-Temperature Relationship for Class B (Dense Graded) Asphalt Concrete in Washington State [2.13]

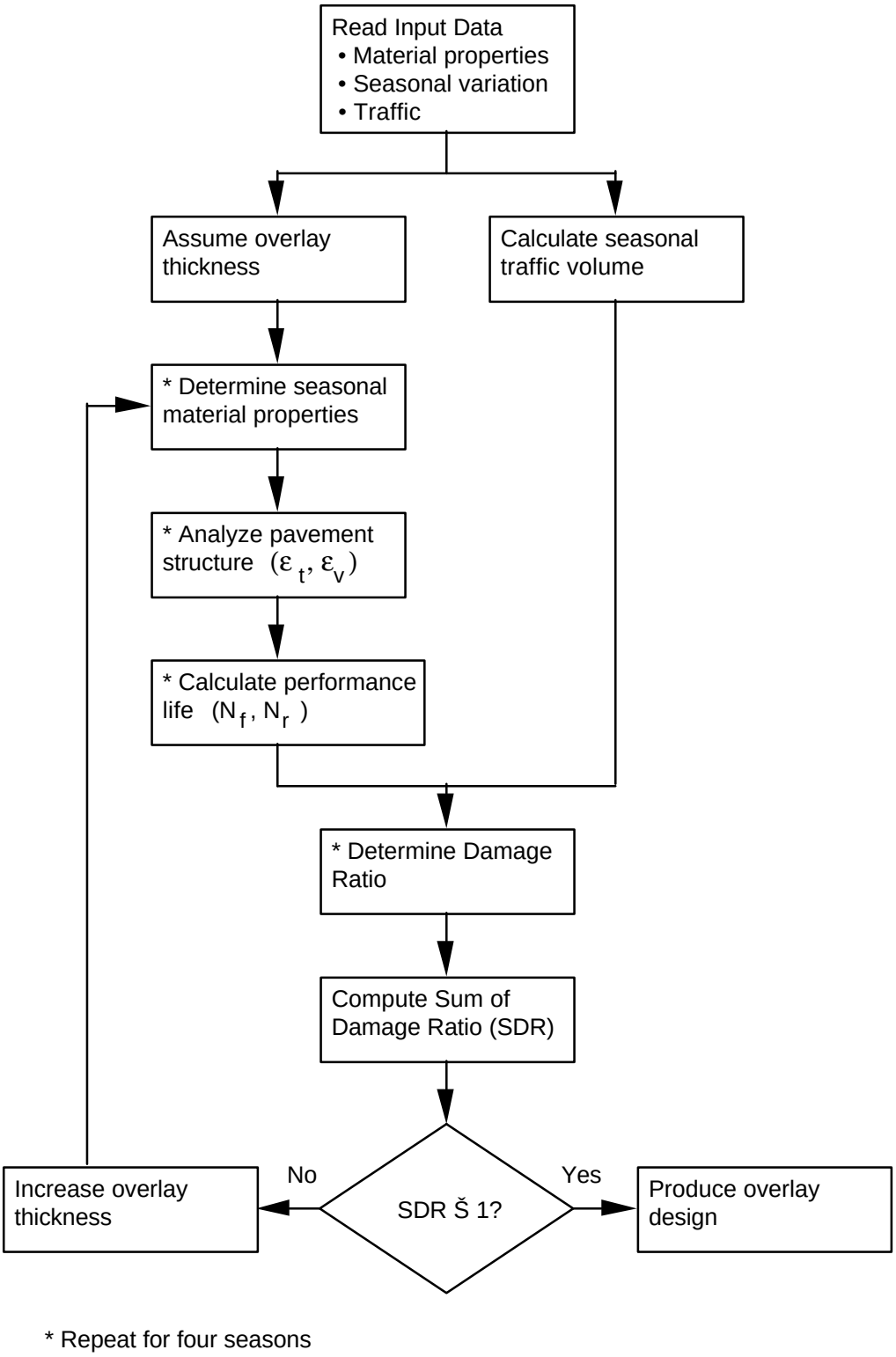
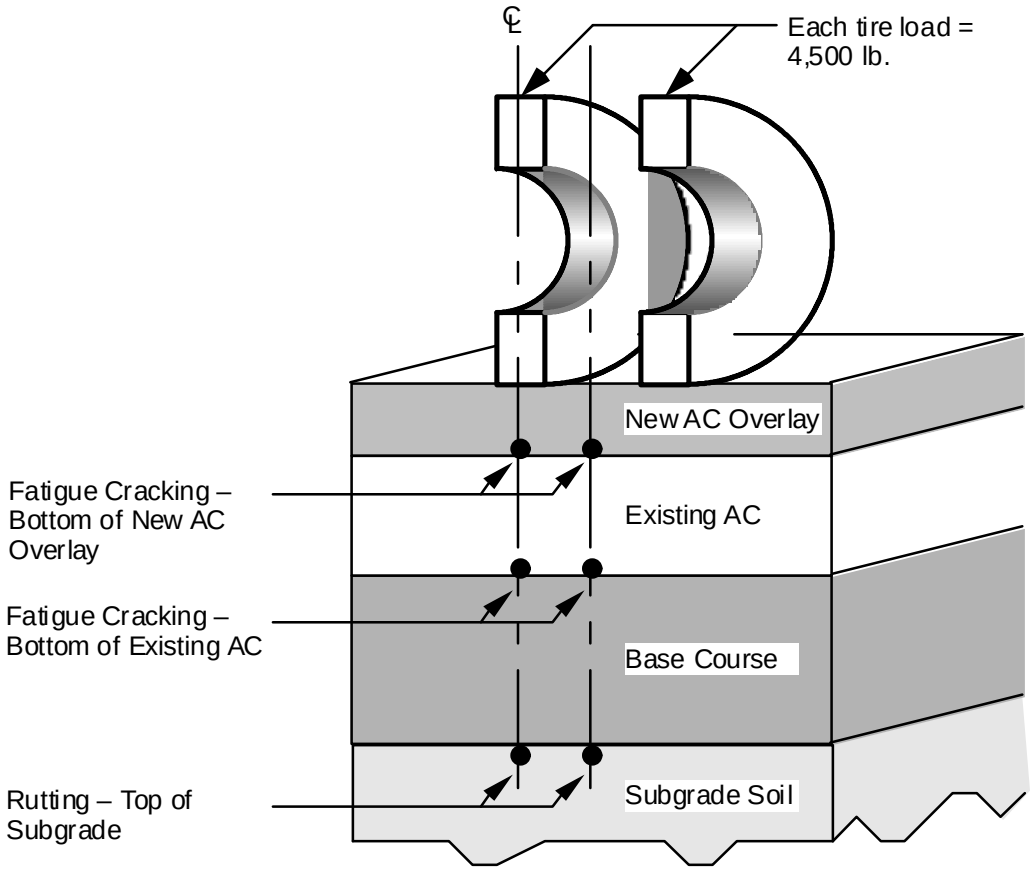


Figure 7.6a. WSDOT Overlay Design Flow Chart



Note: Failure criteria checked beneath one tire and between the two tires.

Figure 7.6b. Failure Criteria Evaluation Points – EVERPAVE Program

The fatigue cracking failure criterion is based on Monismith's laboratory based model [7.11] and the subsequent work by Finn, et al. [7.12] and Mahoney, et al. [7.13].

Fatigue cracking: $N_{field} = (N_{lab})\ (SF)$

where N_{field} = number of load applications of constant stress to cause fatigue cracking,

N_{lab} = relationship from laboratory data [2.13, 2.14],

$$= 10^{\left(14.82 - 3.291 \log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log\left(\frac{E_{ac}}{10^3}\right)\right)}$$

SF = can range from about 4 to 10, depending on AC thickness, ESAL level, climate.

The N_{field} applications is estimated to result in about 10 percent or less fatigue cracking in the wheelpath area. The original Finn, et al. [7.12] model based on the Monismith laboratory work [7.11] and the results of the AASHO Road Test is:

$$N_f = 10^{\left(15.947 - 3.291 \log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log\left(\frac{E_{ac}}{10^3}\right)\right)}$$

where N_f = number of axle applications to result in 10 percent or less fatigue cracking in the wheelpath area,

ε_t = horizontal tensile strain at the bottom of the AC layer, and

E_{ac} = modulus of the AC layer (psi).

The difference between the Finn equation above and the Monismith laboratory based relationship is about 13.4. Thus, the laboratory fatigue relationship was "shifted" by a factor of 13.4 to more realistically represent a <u>field</u> fatigue prediction for the accelerated loading conditions at the AASHO Road Test. WSDOT studies [7.13] have shown, as stated above, that realistic shift factors for in-service WSDOT pavements are less than 13.4 (more like 4 to 10). Generally, the shift factor is increased for high traffic conditions on say 100 to 150 mm of AC. The shift factor is lower for flexible pavements with AC thicknesses of about 175 to 200 mm or thicker. WSDOT personnel have often observed for these thick AC pavements that the cracking starts at the pavement surface, not the bottom of the AC. It is appropriate to note that Finn, et al. [7.12] only analyzed the 100, 125, 150 mm thick AC flexible pavement sections from Loop 4 (7 sections) and Loop 6 (10 sections) from the AASHO Road Test data.

Specifically, WSDOT recommends the following shift factors:

Existing AC Thickness	Shift Factors
< 100 mm	10
100-175mm	7
>175 mm	4

1.6.3 RELIABILITY

Reliability can be incorporated into the design procedure in a very appropriate and similar manner as used in the AASHTO Guide and previously discussed in SECTIONS 5.0 and 6.0. Basically, reliability is simply a multiplier to the estimated design period ESALs, as follows:

$$\text{Design ESALs} = (\text{Estimated ESALs})\ (F_R)$$

where F_R = Reliability design factor (≥ 1)

$= 10^{(-z_R)(s_o)}$

and z_R = z-statistic associated with normal distributions

S_o = overall standard deviation of normal distribution of errors associated with traffic prediction and pavement performance.

An S_o value of 0.5 will be assumed to be applicable for the design of WSDOT overlays (i.e., a greater uncertainty than experienced at the AASHO Road Test for <u>new</u> design). Thus, F_R for various reliability levels are:

Reliability Level (%)	**z_R**	**Reliability Design Factor (F_R)**
50	0.00	1.00
60	-0.25	1.33
70	-0.52	1.82
75	-0.67	2.16
80	-0.84	2.63
85	-1.04	3.31
90	-1.28	4.37
95	-1.65	6.68
99	-2.33	14.62
99.9	-3.10	35.48

For example, if the estimated design period ESALs = 1,000,000, then the following design ESALs would be used as the ESAL input into the EVERPAVE program (for various reliability levels):

Reliability Level (%)	Design ESALs
50	1,000,000
60	1,330,000
70	1,820,000
75	2,160,000
80	2,630,000
85	3,310,000
90	4,370,000
95	6,680,000
99	14,620,000
99.9	35,480,000

The above illustrates how the reliability process works. In very general terms, doubling the design ESALs will increase the AC overlay thickness about 1 in. (for example, going from R = 50% to R = 75 % would add about one inch of AC to the overlay). WSDOT currently uses a reliability level of 50 percent for the design of all AC overlays.

Note that the use of reliability can never be a substitute for proper mix design, construction, maintenance, etc. Further, WSDOT does not currently use this reliability procedure for overlay design, in effect, using an F_R = 1.0 (or reliability level of 50 percent). This is due to at least two reasons. First, using this reliability approach is not entirely appropriate in a non-AASHTO design process. Second, with AC overlays, environmental as well as traffic considerations limit overlay life.

1.6.4 REQUIRED INPUTS — EVERPAVE

The required input data are:

(a) Design ESALs — Estimated ESALs for the design period adjusted for reliability.

(b) Design Load — Normally a dual tire load of 4500 lb on each tire at 100 psi inflation pressure speed 15 in. c-c; however, this can be varied to suit the design conditions.

(c) Seasonal Variation of Layer Moduli — The unstabilized base, subbase, and subgrade layers can be adjusted for seasonal effects. If no adjustment is necessary, use 1.0.

(d) Seasonal Temperatures — Used to adjust the AC moduli for seasonal temperature changes. Refer to Appendix 7.1 for specific recommendations.

(e)	**Shift Factor for Fatigue Criterion**	Ranges from about 4 to 10 (typically). Discussed in Paragraph 1.6.2.
(f)	**Minimum Overlay Thickness**	Generally start with a 0.5 to 1.0 in. thickness.
(g)	**Overlay Incremental Thickness**	Generally use 0.25 to 0.5 in.
(h)	**Unstabilized Layer Moduli**	These can be fixed as linear, elastic (e.g., E_{BS} = 30,000 psi) or non-linear as described in Paragraph 1.6.1 and 1.6.4.3.

1.6.4.1 Seasonal Variation of Unstabilized Moduli

Based on FWD deflections obtained over a three-year period (1985 to 1987), the ratio of the moduli for the different seasons were estimated. These initial estimates are shown in Table 7.4.

The data in Table 7.4 suggest a greater variation in the base course (seasonally) than the subgrade. Clearly, this phenomenon can be quite site-specific. As such, the ratios are only, at best, "rules of thumb."

For dense graded base materials, numerous sources have suggested that moisture levels exceeding 85 percent of saturation can result in significant moduli reductions.

1.6.4.2 Seasonal Temperatures

Seasonal air temperatures are required inputs. These temperatures are used to adjust the AC moduli seasonally. These mean monthly air temperatures (MMAT) are converted to mean monthly pavement temperatures (MMPT) by use of the following equation:

$$\text{MMPT} = \text{MMAT}\left(1 + \left(\frac{1}{z+4}\right)\right) - \left(\frac{34}{z+4}\right) + 6$$

where MMPT = mean monthly pavement temperature (°F),
MMAT = mean monthly air temperature (°F), and
z = depth below pavement surface (in.).

For example, if the MMAT = 65 °F and z = 3 in., then

$$= (65)\left(1 + \left(\frac{1}{3+4}\right)\right) - \left(\frac{34}{3+4}\right) + 6$$

$$= 75.4\ °F$$

Table 7.4. Design Moduli Ratios for Western and Eastern Washington Base Course and Subgrade Materials[1]

Region		Seasonal Period			
		Spring	Summer	Fall	Winter
Western Washington	Climate:	Cool/Wet	Warm/Dry	Cool/Damp	Cool/Wet
	Months:	March April May	June July August September	October November	December January February
	Base	0.85	1.00	0.90	0.75
	Subgrade	0.85	1.00	0.90	0.85
Eastern Washington	Climate:	Thaw	Hot/Dry	Cool/Dry	Freeze
	Months:	February March April May	June July August September	October November December	January
	Base	0.65	1.00	0.90	1.10
	Subgrade	0.90	1.00	0.90	1.10

[1]Design moduli ratios are appropriate for use if stress sensitive moduli relationships are not used. If stress sensitive moduli relationships are used (e.g., $E = k_1 \theta^{k_2}$), then use of these ratios may overestimate seasonal effects.

1.6.4.3 Unstabilized Layer Moduli

Based on extensive laboratory testing, a general model for WSDOT CSTC is:

$$E_{BS} = 8500\ (\theta)^{0.375}$$

where E_{BS} = Modulus of base course (psi),
θ = Bulk Stress = sum of principal stresses under the design load.

If no other data are available, the above model can be used as a default.

1.7 RST SWEDEN AB

The method used by RST Sweden is based on the mechanistic-empirical approach (somewhat similar to the WSDOT approach (Paragraph 1.6)). It has been developed from Swedish and foreign procedures.

Determination of the stiffness of the existing pavement structure is performed by using the CLEVERCALC program (the EVERCALC program modified for Swedish conditions). Normally all points (or stations) measures are processed in order to determine layer moduli, but sometimes the project is subdivided and design-points are chosen.

Temperature corrections on deflections, when needed, are performed by using the TEMPKORR program. If moduli temperature corrections are used, they are performed within the CLEVERCALC program.

Depth to a stiff layer (apparent or actual) is calculated with a program called DJUP or by repeated backcalculations using different depths and a search for the best fit between measured and calculated deflections basin.

The estimated overlay thickness is dependent on the design traffic, existing strains, type of overlay, and failure criteria.

The design traffic is calculated both for last and future design period. The AC and unbound layers are checked to decide whether it will last for the next design period without any reinforcement (AC overlay). If the strains are too high a simulation is performed within the ELSYM5 program trying different thicknesses of overlay material to meet the design criteria.

The AC fatigue cracking criterion used for roads with relatively high traffic is preliminary one under development at VTI. The unbound material is considered (base and subgrade) the Asphalt Institute criterion was recently adopted [7.10].

The results are normally presented as a profile of the calculated overlay thickness. Figure 7.7 overviews the design system.

The temperature adjustment is made with the following relationship:

$$E = e^{9.1242 - 0.04456\,(T)}$$

where E = AC modulus (MPa), and
T = mean asphalt concrete temperature (°C).

For example, the resulting moduli for 5 °C (41 °F), 25 °C (77 °F), and 40 °C (104 °F) are:

T	=	5 °C (41 °F),	E	=	7342.2 MPa (1,064,600 psi)
T	=	25 °C (77 °F),	E	=	3011.5 MPa (436,700 psi)
T	=	40 °C (104 °F),	E	=	1543.5 MPa (223,800 psi)

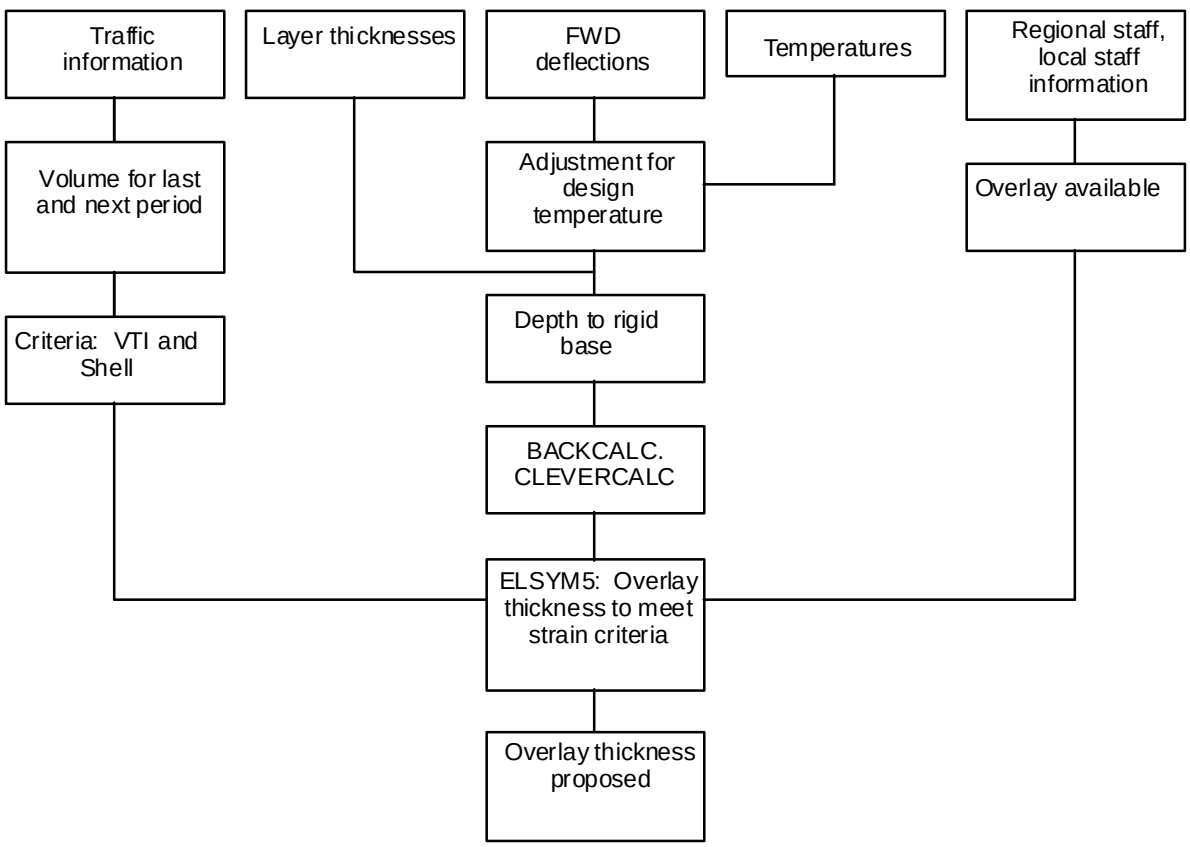


Figure 7.7 RST–Sweden AB Design Flow Chart

As a comparison, the WSDOT modulus-temperature relationship (Figure 7.5) for the same temperatures result in the following:

T	=	41 °F (5 °C),	E	=	1,064,600 psi (7342.2 MPa)
T	=	77 °F (25 °C),	E	=	436,700 psi (3011.5 MPa)
T	=	104 °F (40 °C),	E	=	223,800 psi (1543.5 MPa)

The fatigue cracking failure criterion (preliminary relationship from VTI (Väg-och Trafik Institutet (Swedish Road and Traffic Research Institute), Linköping) is as follows:

$$N_{100} = \frac{1.179 \times 10^{14}}{(\varepsilon_t)}$$

where N_{100} = allowable number of 100 kN (22,500 lb) single axles, and
ε_t = horizontal tensile strain a the bottom of the AC layer (x 10^{-6})

The relationship between 100 kN (22,500 lb) and 18,000 lb (80 kN) single axle loads is approximately (based on AASHTO load equivalency factors [7.15] equal to 2.0 (e.g., a 22,500 lb (100 kN) axle is twice as damaging as a 18,000 lb (80 kN)) axle — or only one-half as many of the heavier axles can be accommodated). A comparison of the WSDOT and VTI fatigue cracking criteria follows:

Criterion	Strain Level	Number of Axles	
		100 kN (22,500 lb)	80 kN (18,000 lb)
VTI (RST Sweden)	100 x 10^{-6}	5,146,500	10,293,000
	200 x 10^{-6}	401,500	803,000
	300 x 10^{-6}	90,300	180,600
WSDOT (E = 400 ksi, SF = 5)	100 x 10^{-6}	2,592,800	5,185,500
	200 x 10^{-6}	265,000	530,000
	300 x 10^{-6}	69,800	139,500
WSDOT (E = 1270 ksi, SF = 5)	100 x 10^{-6}	966,700	1,933,300
	200 x 10^{-6}	98,800	197,500
	300 x 10^{-6}	5,200	10,400

Note: WSDOT normally uses a "design" AC overlay temperature of 25 °C (77 °F), which corresponds to E ≈1,270,000 psi (8759 MPa) at a design AC temperature of 10 °C (50 °F), which corresponds to RST Sweden Practice.

1.8 REVISED AASHTO OVERLAY DESIGN PROCEDURE (1993)

1.8.1 INTRODUCTION

The revised AASHTO overlay design procedure (replaces Chapter 5, Part III, of the 1986 *AASHTO Guide*) has provisions for the following types of overlays [7.16]:

Overlay	Existing Pavement
AC	AC
AC	Break/crack and seat and rubblized PCC
AC	JPCP, JRCP, CRCP
AC	AC/JPCP, AC/JRCP, AC/CRCP
Bonded PCC	JPCP, JRCP, CRCP
Unbonded PCC	JPCP, JRCP, CRCP
PCC	AC

The primary WSDOT use of AASHTO overlay design procedures as currently envisioned are AC overlay over existing AC and AC overlay over JPCP. The other design options will be used as needed on specific projects.

1.8.2 CONSIDERATIONS OF OVERLAY DESIGN

The following important considerations in overlay design were listed by Darter et al. [7.16] and the 1993 *AASHTO Guide* [7.27]:

(a) Preoverlay Repair

If the existing condition of the pavement will likely affect the performance of the overlay within a few years, repair should be considered. Cost tradeoffs between overlay type, thickness, and repairs should be made.

(b) Reflection Crack Control

Most overlay thickness design procedures <u>do not</u> directly account for potential reflection cracking. Reflection crack control measures should be considered if this distress is anticipated.

(c) Traffic Loadings

The design traffic is in terms of 18,000 lb (80 kN) ESALs. The point is made by Darter et al. [716] that rigid pavement ESALs and flexible pavement ESALs will differ for the same project specific traffic. A rule-of-thumb is

Flexible ESALs = 0.67 (Rigid ESALs)
Rigid ESALs = 1.50 (Flexible ESALs)

(d) Subdrainage

The benefits of correcting poor drainage conditions should be examined.

(e) Rutting in AC Pavements

Basically, the cause of rutting must be understood in order to properly design an AC overlay. Are the ruts due to permanent deformation within the AC layer or more within the base and subgrade?

Numerous other considerations can be listed.

1.8.3 AC OVERLAY OF AC PAVEMENT

The thickness design process can be divided into several steps which will be briefly described. The Structural Number required for the overlay is as follows [7.16, 7.27]:

$$SN_{ol} = (a_{ol})\,(D_{ol}) = SN_f - SN_{eff} \qquad \text{(Eq. 7.3)}$$

where SN_{ol} = required overlay Structural Number,
a_{ol} = structural coefficient for the AC overlay,
D_{ol} = required overlay thickness (in.),
SN_f = Structural Number required to carry future traffic, and
SN_{eff} = effective Structural Number of the existing pavement.

1.8.3.1 SN_f

To determine SN_f, the following inputs are required:

- estimate of future ESALs,
- design reliability (R)
- overall standard deviation (S_o),
- effective roadbed resilient modulus (M_R), and
- design serviceability loss (ΔPSI).

1.8.3.2 M_R

The resilient modulus of the subgrade can be determined. by

- NDT backcalculation,
- laboratory testing, or
- other sources/correlations.

WSDOT will normally use the FWD and appropriate equations to estimate M_R. The basic equation for subgrade modulus is [from Ref. 7.16]:

$$M_R = \frac{P\,(1 - \mu^2)}{(\pi)\,(r)\,(d_r)} \qquad \text{(Eq. 7.4)}$$

where M_R = elastic modulus of subgrade (psi),
μ = Poisson's ratio of subgrade,
P = load (lbs)
r = radial distance from the center of the load plate (in.), and
d_r = pavement deflection at distance r from the applied load (in.).

If $\mu = 0.5$, then Equation 7.4 reduces to

$$M_R = \frac{0.24\,(P)}{(r)\,(d_r)} \qquad \text{(Eq. 7.5)}$$

One constraint on the use of either Equation 7.4 or 7.5 is that d_r must be taken at some <u>minimum</u> distance from the center of the load. This minimum distance should be a function of the effective radius (a_e) of the applied load stress bulb at the subgrade pavement interface. If $\mu = 0.5$, then the equation for a_e is

$$a_e = \sqrt{a^2 + \left(D\sqrt[3]{\frac{E_p}{M_R}}\right)^2}$$

where a_e = effective radius of the stress bulb at the subgrade-pavement interface (in.),

a = load plate radius (in.),
D = actual pavement structure thickness (in.),
E_p = effective modulus of the pavement structure (psi), and
M_R = subgrade resilient modulus (psi).

Further, it was found that d_r should be taken at a distance

$$r \geq (0.7)\,(a_e)$$

1.8.3.3 SN_{eff}

To determine SN_{eff}, one has three options

- based on visual survey/material testing,
- based on nondestructive testing, or
- based on remaining life.

WSDOT, for now, will use either the <u>first</u> or <u>second</u> approach. The second approach (based on NDT) takes the following form:

$$SN_{eff} = (0.0045)\,(D)\,(E_p)^{0.33}$$

where D = total thickness of pavement structure above the subgrade (i.e., surfacing, base, subbase) in inches, and

E_p = effective modulus of the pavement structure above the subgrade (psi).

Thus, the problem now becomes one of how to solve for E_p by use of NDT obtained deflections. To achieve a solution for E_p, the NDT data are used to measure d_o (total surface deflection) and solve for M_R. Thus, an equation is needed which "links" d_o, E_p, and M_R together. This equation is

$$d_o = 1.5\,(p)\,(a)\left[\frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{\left(1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right)}{E_p}\right] \qquad \text{(Eq. 7.6)}$$

where d_o = deflection at the pavement surface and adjusted to a standard temperature of 68 °F (in.),

p = contact pressure under the loading plate (psi),
a = load plate radius (in.),
D = actual pavement structure thickness (in.),
M_R = subgrade resilient modulus (psi), and
E_p = effective modulus of the pavement structure (psi).

The derivation of Equation 7.6 follows and is based on the overview provided by Darter et al. [7.16].

First, start with the Boussinesq equation for deflection at any depth for a one layer system.

$$d = \frac{1.5\,(p)\,(a)}{E}\left[\frac{1}{\sqrt{1+\left(\frac{z}{a}\right)^2}}\right]$$

where p = contact pressure (psi),
a = load radius (in.),
E = elastic modulus (psi), and
z = depth below pavement surface (in.).

Next, the concept of "equivalent thickness" is required and is illustrated in Figure 7.8. This scheme was originally developed by Odemark in 1949 and is an approximate method for determining the deflection of a two layer system by use of one elastic modulus value.

The total pavement surface deflection (d_o) is composed of d_p (deflection between z = 0 and z = D) and d_s (deflection at the top of the subgrade), i.e.,

$$d_o = d_p + d_s$$

Thus, we need to estimate both d_p and d_s.

The deflection of d_p is estimated by transforming the two layer system to a one layer system of modulus E_p (refer to top portion of Figure 7.9). Further, the surface deflection with z = 0 is

$$d_o = \frac{1.5\,(p)\,(a)}{E_p}$$

and the deflection at z = D is

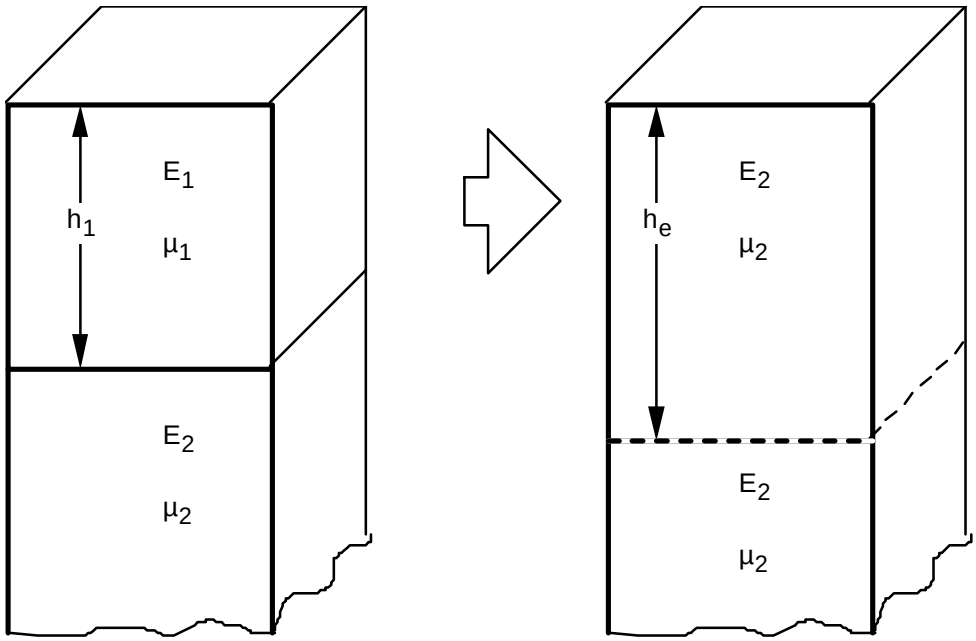
$$d_o = \frac{1.5\,(p)\,(a)}{E_p}\left[\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]$$

Thus d_p is equal to the difference between z = 0 and z = D as follows.

$$d_p = d_o - d_D = \frac{1.5\,(p)\,(a)}{E_p}\left[1 - \frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]$$

The deflection at the top of the subgrade is estimated by transforming the two layer system to a one layer system with modulus M_R (refer to bottom portion of Figure 7.9). The deflection is taken at z = D_e:

$$d_s = \frac{1.5\,(p)\,(a)}{M_R}\left[\frac{1}{\sqrt{1+\left(\frac{D_e}{a}\right)^2}}\right]$$

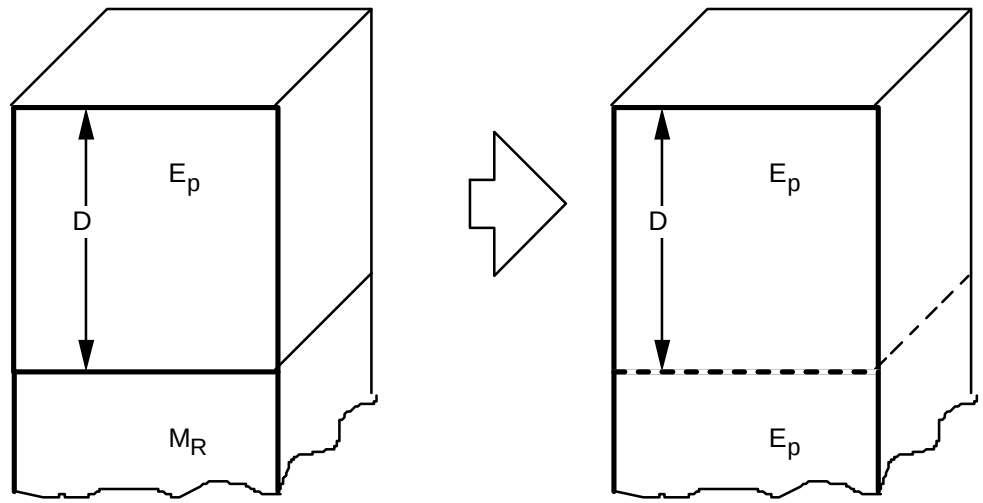


$$\frac{(I)(E)}{I - \mu_2} = \text{constant stiffness}$$

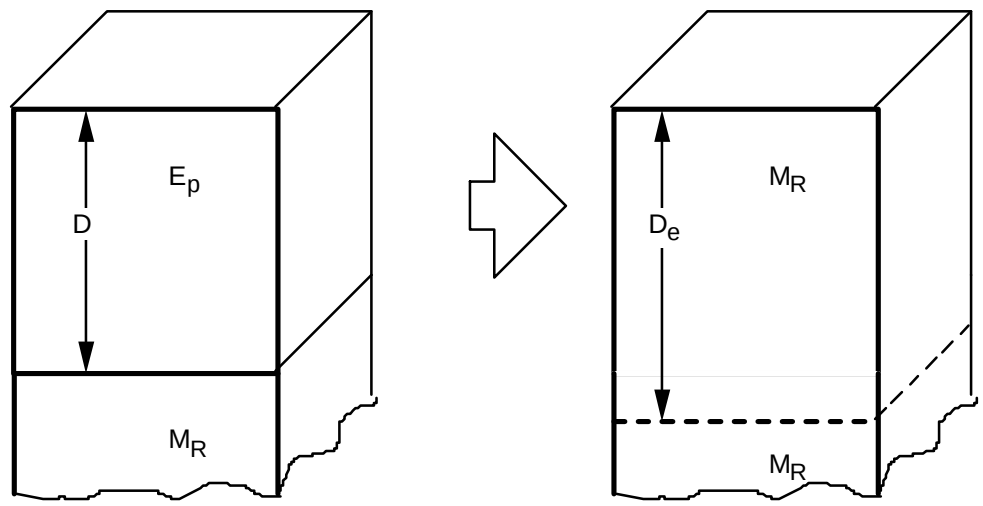
$$\frac{h_e^3 \; E_2}{I - \mu^2{}_2} = \frac{h^3{}_1 E_1}{I - \mu^2{}_1}$$

Figure 7.8. Concept of "Equivalent Thickness"

all $\mu = 0.5$



To obtain deflection of pavement structure (d_p)



To obtain deflection at top of subgrade (d_s)
Then $d_0 = d_p + d_s$ and solve for E_p with M_R known.

Figure 7.9. Transformations Required to Solve for E_p

and $$D_e = (D)\sqrt[3]{\frac{E_p}{M_R}}$$

which results in $$d_s = \frac{1.5\,(p)\,(a)}{M_R}\left[\frac{1}{\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}}\right]$$

Thus, adding d_p and d_s results in Equation 7.6.

A solution for Equation 7.6 (i.e., to solve for E_p) is iterative due to its form. The solution lends itself to the use of PC "spreadsheet" software or the AASHTO supported DARWin 2.0 software. Manual calculations would be lengthy as the following illustrates (use data for "cracked" condition for d_o, D, and M_R from Figure 7.10:

Solve for E_p in Equation 7.6

if:

d_o	=	0.03529 in.
D	=	5.0 in. + 8.0 in. = 13.0 in.
M_R	=	7,500 psi
a	=	5.9 in.
p	=	109.4 psi

<u>Assume</u> E_p = 100,000 psi

From Equation 7.6, <u>calculated</u> d_o = 0.03015 in. The <u>measured</u> d_o = 0.03529 in. Thus, decrease the <u>assumed</u> E_p a bit and redo until convergence achieved between assumed E_p and calculated E_p.

1.8.3.4 Steps

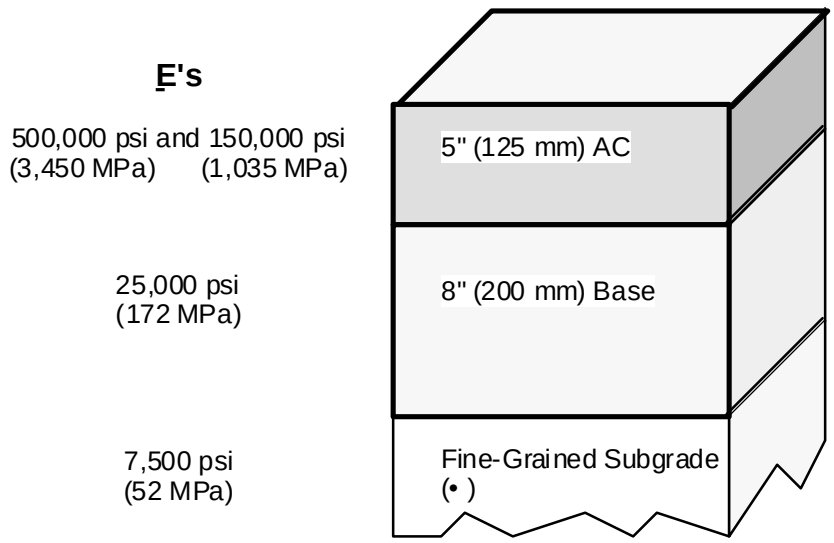
The following steps can be used to design AC overlays [partially after Ref. 7.27, Paragraph 5.5.5, 1993 *AASHTO Guide*]:

(a) Step 1: Existing pavement design and construction

Need:
- thickness and material type for each pavement layer,
- available subgrade information.

(b) Step 2: Traffic analysis

Estimate predicted future ESALs in the design lane for the design period (N_f).



Load	AC Moduli (psi (MPa))	Calculated Deflections (in. (µm))					
		$D_{0"}$	$D_{8"}$	$D_{12"}$	$D_{24"}$	$D_{36'}$	$D_{48"}$
P = 9,000 lb (40 kN)	150,000 (1,035)	0.03529 (896.4)	0.02697 (685.0)	0.02241 (569.2)	0.01383 (351.3)	0.00934 (237.2)	0.00681 (173.0)
h = 5.9 in. (150 mm)	500,000 (3,450)	0.02718 (690.4)	0.02319 (589.0)	0.02039 (517.9)	0.01371 (348.2)	0.00953 (242.1)	0.00698 (177.3)

* Pavement responses obtained from ELSYM5 program

Figure 7.10. Pavement Response Summary for the Medium Thickness Section (Section B)*

(c) Step 3: Condition survey

Obtain information on

- percent of surface area with fatigue (alligator) cracking (Classes 1, 2, and 3 which correspond to low, medium, and high severities),
- number of transverse cracks per mile,
- mean rut depth, and
- evidence of pumping at cracks and pavement edges.

(d) Step 4: Deflection testing

The use of an FWD is strongly encouraged at some fixed spacing in the design lane. Such data (with no temperature adjustment needed) can be used to estimate M_R with Equation 7.4 or 7.5. The deflection sensor used in such equations must be measured at a minimum distance from the center of the load plate as described earlier (r (0.7)(a e)).

The effective modulus of the pavement (E_p) can be determined from Equation 7.6 if M_R and D are known. The value of d_o is obtained from the deflection under the center of the load plate adjusted to a standard temperature of 68 °F (20 °C). Thus, the temperature of the AC layer must be measured during deflection testing or otherwise estimated.

(e) Step 5: Coring and materials testing

The resilient modulus of the subgrade (M_R) can be, presumably, determined by triaxial testing in the laboratory. The WSDOT Materials Laboratory is equipped to do this. For now, AASHTO T 274 (or presumably AASHTO T294) would be used at a deviator stress of 6 psi. This matches the deviator stress used in establishing the 3,000 psi for the AASHO Road Test soil (which is used in the flexible performance equation).

Samples of all stabilized layers should be examined for stripping, etc.

(f) Step 6: Determination of required Structural Number of future traffic (SN_f)

Elements which must be addressed to determine SN_f include the following:

(i) Effective <u>design</u> subgrade resilient modulus

Determine by
- laboratory testing,
- backcalculation from NDT deflection data.

Seasonal adjustments to M_R may be needed. If so, use the procedures in Part II, Section 2.3.1 of the *AASHTO Guide* (also described in SECTION

5.0 of these notes). The recommended design M_R is [from Refs. 7.16 and 7.27]:

$$\text{Design } M_R = C \left(\frac{0.24\ P}{(r)\ (d_r)} \right)$$

where $C = 0.33$

The C = 0.33 is for use in determining SN_f and assumes a NDT load of 9,000 lb (40 kN). Typical <u>design</u> M_R values are 2,000 to 10,000 psi for fine-grained soils and 10,000 to 20,000 psi for coarse-grained soils.

(ii) Design PSI loss

The ΔPSI is the PSI immediately after overlay minus the terminal PSI (or p_t). Ranges from about 1.2 to 2.5.

(iii) Overlay design reliability (R)

Refer to the 1993 *AASHTO Guide*, Part I, Section 4.2, and Part III, Section 5.2.15. Ranges from 50 to 99 percent.

(iv) Overall standard deviation (S_o) for flexible pavement.

Refer to the *AASHTO Guide*, Part I, Section 4.3. Typically S_o about 0.50.

(v) Compute SN_f

Use <u>mean</u> input values.

(g) Step 7: Determine the effective Structural Number (SN_{eff}) of the existing pavement

Two methods are currently being used by WSDOT to estimate SN_{eff}.

(i) NDT Method

As previously shown, SN_{eff} can be estimated from

$$SN_{eff} = (0.0045)\ (D) \left(\sqrt[3]{E_p} \right)$$

where E_p is "backcalculated" from deflection data.

(ii) Condition survey

The approach uses essentially a component analysis and starts with

$$SN_{eff} = (a_1)(D_1) + (a_2)(D_2)(m_2) + (a_3)(D_3)(m_3)$$

where D_1, D_2, D_3 = thicknesses of existing surface, base and subbase layers,

a_1, a_2, a_3 = current structural layer coefficients, and

m_2, m_3 = drainage coefficients for granular base and subbase.

The key element is to estimate the in situ structural layer coefficients. Table 7.5 was prepared by Darter et al. [7.16] to assist in making these estimates. (Note: Table 7.5 (these notes) is shown as Table 5.2, p. III-105, 1993 *AASHTO Guide*).

(h) Step 8: Determination overlay thickness

Estimated by

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}}$$

where SN_{ol} = required overlay Structural Number,

a_{ol} = structural layer coefficient for the AC overlay,

D_{ol} = required overlay thickness (in.),

SN_f = Structural Number from Step 6,

SN_{eff} = effective Structural Number for the existing pavement, from Step 7.

Table 7.5. Suggested Layer Coefficients for Existing AC Pavement Layer Materials [from Ref. 7.16]

Material	Surface Condition	Coefficient
AC Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 to 0.40
	< 10 percent low-severity alligator cracking and/or < 5 percent medium- and high-severity transverse cracking	0.25 to 0.35
	> 10 percent low-severity alligator cracking and/or < 10 percent medium-severity alligator cracking and/or > 5-10 percent medium- and high-severity transverse cracking	0.20 to 0.30
	> 10 percent medium-severity alligator cracking and/or < 10 percent high-severity alligator cracking and/or > 10 percent medium- and high-severity transverse cracking	0.14 to 0.20
	> 10 percent high-severity alligator cracking and/or > 10 percent high-severity transverse cracking	0.08 to 0.15
Stabilized Base	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 to 0.35
	< 10 percent low-severity alligator cracking and/or < 5 percent medium- and high-severity transverse cracking	0.15 to 0.25
	> 10 percent low-severity alligator cracking and/or < 10 percent medium-severity alligator cracking and/or > 5-10 percent medium- and high-severity transverse cracking	0.15 to 0.20
	> 10 percent medium-severity alligator cracking and/or < 10 percent high-severity alligator cracking and/or > 10 percent medium- and high-severity transverse cracking	0.10 to 0.20
	> 10 percent high-severity alligator cracking and/or > 10 percent high-severity transverse cracking	0.08 to 0.15
Granular Base or Subbase	No evidence of pumping, degradation, or contamination by fines	0.10 to 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.00 to 0.10

1.9 EXAMPLE

1.9.1 INTRODUCTION

The medium AC thickness section (B) shown in Figure 4.9 (SECTION 4.0) will be used to illustrate the use of some of the various AC overlay design procedures. Even though this is only a hypothetical pavement, we will make the necessary assumptions to make use of this "pavement section." We assume that section B can have two levels of AC stiffness, E_{AC} = 500,000 psi (3450 MPa), which assumes no cracking, and E_{AC} = 150,000 (1035 MPa), which implies extensive fatigue cracking of the AC surfacing. The necessary material properties, layer thicknesses, and deflections (calculated from ELSYM5) are summarized in Figure 7.10. Further, we will design AC overlays for three assumed ESAL levels: 1,000,000; 2,000,000; and 5,000,000.

1.9.2 ASPHALT INSTITUTE EFFECTIVE THICKNESS PROCEDURE

(a) Subgrade M_R

M_R = 7500 psi (52 MPa), a given

(b) Traffic analysis

Use ESAL levels of: 1,000,000
2,000,000
5,000,000

Therefore, obtain three overlay thicknesses.

(c) Effective thickness of existing pavement structure (T_e)

Use Table 7.1 for equivalency factors.

Layer	AC @ 500,000 psi (3450 MPa)	AC @ 150,000 (1035 MPa)
AC	(5") (1.0) = 5.0	(5") (0.5) = 2.5
Base	(8") (0.2) = 1.6	(8") (0.2) = 1.6
	= 6.6" (168 mm)	= 4.1" (104 mm)

(d) Determine thickness of new pavement (T_n). Use Figure 7.1 and appropriate M_R and ESAL levels.

M_R	ESAL Level	T_n
7500 psi	1,000,000	9.1" (231 mm)
(52 MPa)	2,000,000	10.3" (262 mm)
	5,000,000	12.3" (312 mm)

(e) Determine AC overlay thickness (T_o)

ESAL Level	E_{AC}, psi (MPa)	$T_o = T_n - T_e$
1,000,000	500,000 (3450)	9.1" - 6.6" = 2.5" (64 mm)
2,000,000	500,000 (3450)	10.3" - 6.6" = 3.7" (94 mm)
5,000,000	500,000 (3450)	12.3" - 6.6" = 5.7" (145 mm)
1,000,000	150,000 (1035)	9.1" - 4.1" = 5.0" (127 mm)
2,000,000	150,000 (1035)	10.3" - 4.1" = 8.2" (208 mm)
5,000,000	150,000 (1035)	12.3" - 4.1" = 8.2" (208 mm)

1.9.3 ASPHALT INSTITUTE DEFLECTION PROCEDURE

(a) E_{AC} = 150,000 psi (1035 MPa)

Assume that D_o from Figure 7.10 represents the mean deflection with a standard deviation about 1/2 as large as the mean. Therefore,

$\bar{x}$ = 0.03529" (896.4 μm)
s = 0.01764" (448.2 μm)

Further, assume the deflections were obtained for an average pavement temperature of 60 °F and the critical period adjustment factor (c) = 1.25. The temperature adjustment factor (f) ≈ 1.1 from Figure 7.2.

RRD = (0.03529 + 2(0.01764)) (1.1) (1.25)
= 0.09703" (2464.7 μm)

Overlay thickness = 4.8" (122 mm) for 1,000,000 ESALs
(from Figure 7.3) = 5.8" (147 mm) for 2,000,000 ESALs
= 7.5" (190 mm) for 5,000,000 ESALs

(b) E_{AC} = 500,000 psi (3450 MPa)

All calculations and estimations will be the same as (a) except D_o = 0.02718" (690.4 μm). Thus,

$\bar{x}$ = 0.02718" (690.4 μm)
s = 0.01359" (345.2 μm)
f = 1.1
c = 1.25

Therefore, RRD = (0.02718 + 2(0.01359)) (1.1) (1.25)
= 0.07475" (1898.5 μm)

Overlay thickness = 3.8" (97 mm) for 1,000,000 ESALs
(from Figure 7.3) = 4.8" (122 mm) for 2,000,000 ESALs
= 6.2" (157 mm) for 5,000,000 ESALs

(c) E_{AC} = 150,000 Psi (1035 MPa) and E_{AC} = 500,000 psi (3450 MPa) Revised

If the pavement deflections had been taken during the critical period and no temperature adjustment was needed (i.e., measurements obtained at 70 °F (21 °C)) and all measurements were the same (i.e., s = 0, which is highly unlikely), then the resulting overlays would be the following:

E_{AC} = 150,000 psi (1035 MPa)
RRD = 0.03529" (896.4 mm)

E_{AC} = 500,000 psi (3450 MPa)
RRD = 0.02718" (690.4 mm)

E_{AC} = 150,000 psi (1035 MPa)

Overlay thickness = 0.4" (10 mm) for 1,000,000 ESALs
(from Figure 7.3) = 1.7" (43 mm) for 2,000,000 ESALs
= 2.9" (74 mm) for 5,000,000 ESALs

E_{AC} = 500,000 psi (3450 MPa)

Overlay thickness = 0.0" (0 mm) for 1,000,000 ESALs
(from Figure 7.3) = 0.0" (0 mm) for 2,000,000 ESALs
= 1.8" (46 mm) for 5,000,000 ESALs

1.9.4 WSDOT MECHANISTIC-EMPIRICAL

(a) The EVERPAVE program was used with the following inputs (assumed location for seasonal effects is Spokane, Washington).

Case 1

- Existing pavement moduli
  - E_{AC} = 150,000 psi (1035 MPa)
  - E_{BS} = 25,000 psi (172 MPa)
  - E_{SG} = 7,500 psi (52 MPa)
- New AC modulus = 500,000 psi (3450 MPa)
- Fatigue shift factor = 10
- ESAL levels 1,000,000
  2,000,000
  5,000,000

Case 2

Same as Case 1 but existing EAC = 500,000 psi (3450 MPa) (i.e., no initial fatigue cracking).

Case 3

- Existing pavement moduli
  - E_{AC} = 150,000 psi (1035 MPa)
  - E_{BS} = 8,000 $(\theta)^{0.375}$
  - E_{SG} = 7,500 psi (52 MPa)
- New AC modulus = 400,000 psi (2760 MPa)
- Fatigue shift factor = 10
- ESAL levels 1,000,000
  2,000,000
  5,000,000

Case 4

Same as Case 3 but existing EAC = 500,000 psi (3450 MPa) (i.e., no initial fatigue cracking).

(b) Results

Case 1 (Original surfacing E_{AC} = 150,000 psi (1035 MPa)) "cracked AC"

ESAL Level	AC Overlay Thickness, inches (mm)
1,000,000	1.2" (30 mm)
2,000,000	2.0" (51 mm)
5,000,000	3.5" (89 mm)

Case 2 (Original surfacing E_{AC} = 500,000 psi (3450 MPa)) "uncracked AC"

ESAL Level	AC Overlay Thickness, inches (mm)
1,000,000	0" (0 mm)
2,000,000	0.5" (13 mm)
5,000,000	1.7" (43 mm)

Case 3 (Original surfacing E_{AC} = 150,000 psi (1035 MPa)) "cracked AC"

ESAL Level	AC Overlay Thickness, inches (mm)
1,000,000	2.2" (56 mm)
2,000,000	3.2" (81 mm)
5,000,000	5.0" (127 mm)

Case 4 (Original surfacing E_{AC} = 500,000 psi (3450 MPa)) "uncracked AC"

ESAL Level	AC Overlay Thickness, inches (mm)
1,000,000	0.2" (5 mm)
2,000,000	1.2" (30 mm)
5,000,000	2.8" (71 mm)

1.9.5 RST SWEDEN

(a) AC overlay thicknesses were estimated for two cases as follows:

Case 1

- Existing pavement moduli
  - E_{AC} = 150,000 psi (1035 MPa)
  - E_{BS} = 25,000 psi (172 MPa)
  - E_{SG} = 7,500 psi (52 MPa)
- New AC modulus
  - Prior examples used 500,000 psi (3450 MPa); however, this was changed to 851,400 psi (5876 MPa) due to the fatigue cracking criterion being based on 50 °F(10 °C). The E was calculated as follows:

$$E = e^{9.1242 - 0.04456 (T)}$$

for T = 10 °C (50 °F)
then E = 5876 MPa (851,400 psi)

- ESAL levels
  - 80 kN 1,000,000
    2,000,000
    5,000,000
  - 100 kN 410,000
    820,000
    2,050,000

Case 2

- Existing pavement moduli

  - E_{AC} = 662,200 psi (4570 MPa) (approximately 50 °F (10 °C))
  - E_{BS} = 25,000 psi (172 MPa)
  - E_{SG} = 7,500 psi (52 MPa)

- New AC modulus

  $E_{AC/ov}$ = 851,400 psi (5876 MPa) as for Case 1

- ESAL levels

  - 80 kN 1,000,000
    2,000,000
    5,000,000

  - 100 kN 410,000
    820,000
    2,050,000

(b) Results

General

ESALs (80 kN)	Allowable Tensile Strain
1,000,000	199 x 10-6
2,000,000	165 x 10-6
5,000,000	128 x 10-6

Case 1 "Cracked AC"

ESALs (80 kN) Level	AC Overlay Thickness, inches (mm)
1,000,000	4.3" (110 mm)
2,000,000	5.2" (133 mm)
5,000,000	6.5" (165 mm)

Case 2 "Uncracked AC"

ESALs (80 kN) Level	AC Overlay Thickness, inches (mm)
1,000,000	0.5" (13 mm)
2,000,000	1.6" (41 mm)
5,000,000	3.2" (81 mm)

1.9.6 REVISED AASHTO

(a) AC overlay thicknesses were estimated for the two cases (use NDT Method only):

Case 1 (uncracked AC layer)

D_o = 0.02718 in.
$D_{24"}$ = 0.01371 in.
C = 3
D = 5.0 in. + 8.0 in. = 13.0 in.
p_t = 2.5
S_o = 0.49

Case 2 (cracked AC layer)

D_o = 0.03529 in.
$D_{24"}$ = 0.01383 in.
C = 3
D = 13.0 in.
p_t = 2.5
S_o = 0.49

(b) Results

Case 1 (uncracked AC, M_R based on deflection @ 24")

- SN_f

ESALs (80 kN) Level	SN_f*
1,000,000	4.25
2,000,000	4.69
5,000,000	5.32

*Assumes p_i = 4.5, p_t = 2.5, R = 50%, M_R = 2188 psi

- Calculation of M_R: assumed that only the deflection data are available to characterize the subgrade, thus we do not know that the actual subgrade M_R = 7500 psi (this assumption is quite conservative and will result in thick AC overlays).

$$M_R = \frac{(0.24)\ (P)}{(r)\ (d_r)} \qquad \text{(from Eq. 7.5)}$$

$$= \frac{(0.24)\ (9000)}{(24)\ (0.01371)} = 6{,}564 \text{ psi}$$

If C = 3, then $M_R = \frac{6{,}564}{3} = 2{,}188$ psi

Check $r \geq (0.7)\ (a_e)$ to ensure deflection if far enough from the center of the load plate. Recall

$$a_e = \sqrt{a^2 + \left(D \sqrt[3]{\frac{E_p}{M_R}}\right)^2}$$

$$a_e = \sqrt{5.9^2 + \left(13.0 \sqrt[3]{\frac{77593}{6564}}\right)^2} = 30.2$$

$r \geq (0.7)\ (30.2) = 21.1$ in., OK since r = 24 in.

- SN_{eff} (NDT Method)

  Calculated E_p = 77,593 psi

  $SN_{eff} = (0.0045)\ (13.0)\ (77{,}593)^{1/3} = 2.50$

- $a_{ol} = 0.44$

- $D_{ol} = \frac{SN_f - SN_{eff}}{a_{ol}}$

ESALs (80 kN) Level	D_{ol}
1,000,000	4.0"
2,000,000	5.0"
5,000,000	6.4"

Case 2 (cracked AC, M_R based on deflection @ 24")

- SN_f

ESALs (80 kN) Level	SN_f*
1,000,000	4.28
2,000,000	4.70
5,000,000	5.34

*Assumes p_i = 4.5, p_t = 2.5, R = 50%, M_R = 2169 psi

- Calculation of M_R: assumed, as before, that only the deflection data are available to characterize the subgrade, thus we do not know that the actual subgrade M_r = 7,500 psi (this assumption is quite conservative and will result in thick AC overlays).

$$M_R = \frac{(0.24)\,(9000)}{(24)\,(0.01383)} = 6{,}508 \text{ psi}$$

$$\text{If } C = 3, \text{ then } M_R = \frac{6{,}508}{3} = 2{,}169 \text{ psi}$$

Check $r \geq (0.7)\,(a_e)$

$$a_e = \sqrt{5.9^2 + \left(13.0\sqrt[3]{\frac{45{,}293}{6{,}508}}\right)^2} = 25.5 \text{ in.}$$

$r \geq (0.7)\,(25.5) = 17.9$ in., OK since r = 24.0 in.

- SN_{eff} (NDT Method)

Calculated E_p = 45,293 psi

$$SN_{eff} = (0.0045)\,(13.0)\,(45{,}293)^{1/3} = 2.09$$

- $a_{ol} = 0.44$

- $D_{ol} = \dfrac{SN_f - SN_{eff}}{a_{ol}}$

ESALs (80 kN) Level	D_{ol}
1,000,000	5.0"
2,000,000	5.9"
5,000,000	7.4"

Case 3 (uncracked AC, M_R = 6564 psi))

- SN_f

ESALs (80 kN) Level	SN_f*
1,000,000	2.89
2,000,000	3.21
5,000,000	3.69

*Assumes p_i = 4.5, p_t = 2.5, R = 50%

- a_{ol} = 0.44

- $D_{ol} = \frac{SN_f - SN_{eff}}{a_{ol}}$

ESALs (80 kN) Level	D_{ol}
1,000,000	0.9"
2,000,000	1.6"
5,000,000	2.7"

Case 4 (cracked AC, M_R = 6564 psi))

- SN_f

ESALs (80 kN) Level	SN_f*
1,000,000	2.89
2,000,000	3.21
5,000,000	3.69

*Assumes p_i = 4.5, p_t = 2.5, R = 50%

- a_{ol} = 0.44

- $D_{ol} = \frac{SN_f - SN_{eff}}{a_{ol}}$

ESALs (80 kN) Level	D_{ol}
1,000,000	1.8"
2,000,000	2.6"
5,000,000	3.6"

1.9.7 EXAMPLE SUMMARY

Refer to Table 7.6 for a summary of the various overlay thicknesses for the cases used in this example. Given that the "pavement section" used was purely hypothetical and required numerous assumptions, one should not expect the various overlay design procedures to result in similar solutions; however, there is a modest amount of agreement among the design procedures used.

1.10 GRANULAR OVERLAYS (CUSHION COURSES)

The granular overlay system (often referred to as a "cushion course") is an alternative type of overlay for rehabilitating mostly low volume, rural roads (this does not necessarily imply a low number of ESALs). The overlay consists of a layer of densely compacted, crushed rock (generally WSDOT Crushed Surfacing Top Course or Base Course) overlain by a generally thin surface layer. Granular overlays have been used throughout the world as a pavement rehabilitation treatment (within WSDOT such overlays are most common in Regions 2 and 6). The reasons for their use fall into four primary categories:

- To reduce reflective cracking from a preexisting pavement structure,
- To add extra pavement structure thickness to combat frost related effects,
- To improve the cross-slope, road profile (and ride in general), and
- To strengthen the pavement structure.

One view of why granular overlays structurally perform as well as they do is that they take advantage of the stress stiffening behavior of the granular layer (material). When a crushed rock layer is subjected to a confining pressure, its stiffness increases. Since the old pavement surface and the new surfacing confine the crushed rock layer in a granular overlay, traffic loads can provide high confining stresses which, in effect, increase the stiffness of the crushed rock layer. This increased stiffness implies increased structural capacity for the pavement structure.

For the purpose of design, the following must be considered:

- BSTs are more appropriate surfacings for granular overlays than AC (based on observed performance).

- The crushed rock layer maximum thickness should not exceed 6.0 in. (150 mm) with a minimum thickness of 3.0 in. (75 mm) (based on structural considerations only).

- A design approach is to first estimate the needed AC overlay thickness then convert to an equivalent granular overlay. The crushed rock layer that is properly constructed and well protected from water intrusion has an equivalency to AC of about 2.0. A scheme to calculate the required granular overly depth follows:

$$\text{AC Overlay Depth} = \text{Assumed Surfacing Depth} + \frac{\text{Granular Layer Depth}}{2}$$

where Assumed Surfacing Depth = normal surfacing depth used (a 3 shot Class A BST would be about 1.0 in. (25 mm)), and

Granular Layer Depth = depth of crushed surfacing material.

Table 7.6. Summary of Overlay Thicknesses for Medium Thickness AC (Section B)

ESAL Level	Design Procedures			
	AI - EFF. Tk. @ E_{AC} = 150 ksi (1035 MPa) "cracked"	AI - Eff. Tk. @ E_{AC} = 500 ksi (3450 MPa) "uncracked"	AI - Deflection @ E_{AC} = 150 ksi (1035 MPa) "cracked"	AI-Deflection @ E_{AC} = 500 ksi (3450 MPa) "uncracked"
1,000,000	5.0" (127 mm)	2.5" (64 mm)	4.8" (122 mm)	3.8" (97 mm)
2,000,000	6.2" (157 mm)	3.7" (94 mm)	5.8" (147 mm)	4.8" (122 mm)
5,000,000	8.2" (208 mm)	5.7" (145 mm)	7.5" (190 mm)	6.2" (157 mm)

ESAL Level	Design Procedures			
	WSDOT EVERPAVE @ E_{AC} = 150 ksi "cracked" E_{BS} = 25000 psi	WSDOT EVERPAVE @ E_{AC} = 500 ksi "uncracked" E_{BS} = 25000 psi	WSDOT EVERPAVE @ E_{AC} = 150 ksi E_{BS} = 8000 $(\theta)^{0.375}$	WSDOT EVERPAVE @ E_{AC} = 500 ksi E_{BS} = 8000 $(\theta)^{0.375}$
1,000,000	1.2" (30 mm)	0" (0 mm)	2.2" (56 mm)	0.2" (5 mm)
2,000,000	2.0" (51 mm)	0.5" (13 mm)	3.2" (81 mm)	1.2" (30 mm)
5,000,000	3.5" (89 mm)	1.7" (43 mm)	5.0" (127 mm)	2.8" (71 mm)

ESAL Level	Design Procedures			
	RST Sweden @ E_{AC} = 150 ksi "cracked	RST Sweden @ E_{AC} = 851 ksi "uncracked"	AASHTO "cracked" R = 50% M_R = 2169 psi	AASHTO "uncracked" R = 50% M_R = 2188 psi
1,000,000	4.3" (110 mm)	0.5" (13 mm)	5.0" (127 mm)	4.0" (102 mm)
2,000,000	5.2" (133 mm)	1.6" (41 mm)	5.9" (150 mm)	5.0" (127 mm)
5,000,000	6.5" (165 mm)	3.2" (81 mm)	7.4" (188 mm)	6.4" (163 mm)

Table 7.6 Summary of Overlay Thicknesses for Medium Thickness AC (Section B) (Continued)

ESAL Level	AASHTO "cracked" R = 50% M_R = 6564 psi	AASHTO "uncracked" R = 50% M_R = 6564 psi
1,000,000	1.8" (46 mm)	0.9" (23 mm)
2,000,000	2.6" (66 mm)	1.6" (41 mm)
5,000,000	3.6" (91 mm)	2.7" (69 mm)

For example, if the required AC overlay depth = 3.0 in. (75 mm), then

$$3.0 \text{ in. AC} = 1.0 \text{ in. BST} + \frac{(4.0 \text{ in. Granular})}{2}$$

Thus, the granular overlay would have a 1.0 in. (25 mm) thick surfacing (presumably a 3 shot BST — such as WSDOT Class A BST) and a 4.0 in. thick crushed rock layer all of which is placed on the existing pavement structure.

- The preexisting surface should be left in place if project conditions permit. This enhances the stress stiffening of the crushed rock layer in the granular overlay system. In fact, if the preexisting surface has significant cracking, consideration should be given to application of a heavy fog seal or single shot BST for the purpose of minimizing the upward movement of water.

- A granular overlay must be protected from being saturated thus trapping excess moisture. Adequate drainage is always important but particularly so for this type of overlay.

2. REHABILITATION OF PCC

2.1 INTRODUCTION

Most of the asphalt concrete pavements can be rehabilitated with asphalt concrete overlays; however, rehabilitation of the portland cement concrete pavements, which have a longer service life, is not as straightforward. To provide a more consistent analysis process throughout the state, the following general guidelines have been developed.

2.2 BASIC CONCEPTS

2.2.1 MAINTENANCE

Preventive maintenance, such as joint resealing, minor grinding and profiling, and surface maintenance of AC shoulders, generally yields the greatest benefit when performed early in the life of the pavement, before moisture related and joint related distresses develop to significant levels. (Note: Resealing of joints shall conform to WSDOT Standard Specification 5-05.3(8)B.)

2.2.2 RESTORATION

Restoration activities are warranted when the pavement condition has declined such that the presence of cracking, faulting, and joint spalling are detracting from the pavement's serviceability. Restoration techniques are used to bring the pavement back to a "near new" condition and thus extend its useful life. Restoration techniques for concrete pavements include:

- Full depth repair of joints, cracks, and corner breaks
- Grinding to remove faults, studded tire ruts, and to improve surface friction
- Slab jacking to improve the pavement's profile
- Joint resealing
- Crack sealing
- Subdrainage improvement
- Shoulder improvement
- Retrofit load transfer devices

Concrete pavement restoration (CPR) may involve one of these techniques or a combination of several.

2.2.2.1 Subsealing of PCC Pavements

Subsealing is recognized as one of the basic rehabilitation techniques for plain, jointed PCC pavements. WSDOT has subsealed a number of PCC pavements (mostly on I-5) during the 1980s. The subsealing is accomplished by pumping cement grout into pre-drilled holes. The intent of placing the grout under the slabs is to fill void spaces that

may have occurred (subsealing should not be confused with "slab jacking" which refers to "lifting" a slab to its original position).

2.2.2.1(a) Grout Material

Typically, the grout material is composed of portland cement, pozzolan, and water. The standard proportions are (WSDOT 1994 Standard Specifications Paragraph 5-01.2):

- 1 part (by volume) portland cement (Type I or II)
- 3 parts (by volume) pozzolan
- 2.25 parts (by volume) water.

The pozzolan shall meet ASTM C618 (Class C, F, and N).

2.2.2.1(b) Criteria for Subsealing

The FHWA has recommended deflection testing as one criterion for use in determining the need for subsealing (hence location). A criterion of 0.025 in. (25 mils) has been suggested by the FHWA (*FHWA Pavement Rehabilitation Manual*). The deflections are typically measured with an 18,000 lb axle during the early morning hours (cooler slab temperatures). WSDOT has used deflection criteria ranging from 0.020 in. to 0.035 in. (depends on job specific conditions).

Other criteria can be used such as evidence of pumping, etc.

2.2.2.1(c) Subsealing Effectiveness

The results of subsealing on WSDOT projects have been mixed. Generally, for non-CTB bases, "voids" under PCC slabs are minimal, hence the grout is of little benefit. The results are consistently better on those few projects where PCC overlies CTB.

2.2.2.2 Restoration of Joint Load Transfer

2.2.2.2(a) General

WSDOT completed (September 1992) its first joint restoration project on I-90 east of Snoqualmie Pass. Test sections were developed which combined retrofitting dowel bars to the existing undoweled joints along with grinding to remove the roughness caused by faulting. This project and associated results will be presented in some detail later in subparagraph 2.2.4. First, a brief overview on how Load Transfer Efficiency (LTE) is measured. LTE is a commonly used measure to gauge the load transfer at a joint; hence, the need for restoration. Second, a summary on retrofitting dowel bars provided in the AASHTO Guide [7.15] will follow. Third, a brief overview of results from other dowel retrofit projects in other states is provided.

2.2.2.2(b) Load Transfer Efficiency

When a wheel load is applied at a joint or crack, both the loaded slab and adjacent unloaded slab deflect. The amount the unloaded slab deflects is directly related to joint performance. If a joint is performing perfectly, both the loaded and unloaded slabs deflect equally.

Joint performance can be evaluated by calculating load transfer efficiency (LTE) across a joint or crack using measured deflection data. The concept of joint load transfer efficiency is illustrated in Figure 7.11. Load transfer efficiency can be calculated using the following equation:

$$LTE = (\Delta_u/\Delta_l) * 100$$

LTE = load transfer efficiency, percent

Δ_u = the deflection of the unloaded slab, mils

Δ_l = the loaded slab deflection, mils.

Joint efficiency depends on several factors, including temperature (which affects joint opening), joint spacing, number and magnitude of load applications, foundation support, aggregate particle angularity, and the presence of mechanical load transfer devices. WSDOT typically does not use mechanical load transfer devices, so load transfer was provided solely by aggregate interlock across the joint or crack.

As mentioned, temperature plays a major role in determining joint effectiveness. In general, the lower the temperature, the lower the load transfer efficiency. Load transfer efficiency is reduced because joints open during cooler weather, reducing contact between faces. Joint load transfer efficiency has also been shown, in both laboratory and field studies, to decrease with increasing load applications. However, this impact is lessened for harder and more regular angular aggregates. The aggregate characteristics play a more significant role after many load applications.

To test the approach side of a joint or crack, the FWD loading plate is placed in front of the joint, with the other velocity transducers located across the joint. The leave side of the joint is tested by placing the loading plate at the joint edge on the leave slab with an extra velocity transducer mounted behind the loading plate across the joint. The concept of slab approach and leave sides and of transverse joint testing are illustrated in Figure 7.12.

Joint Load Transfer

- Aggregate Interlock
- Keyways
- Dowel Bars

$$\text{Load Transfer Efficiency (\%)} = \frac{\Delta_u}{\Delta_l} \times 100$$

Where Δ_u = unloaded slab deflection

Δ_l = loaded slab deflection

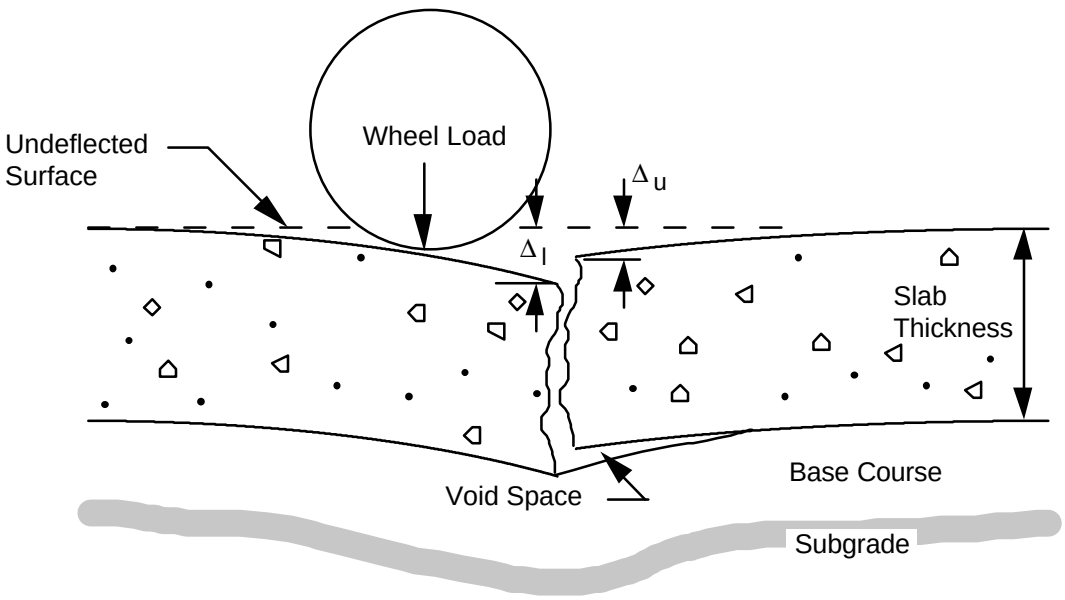


Figure 7.11. The Concept of Joint Load Transfer Efficiency (LTE)

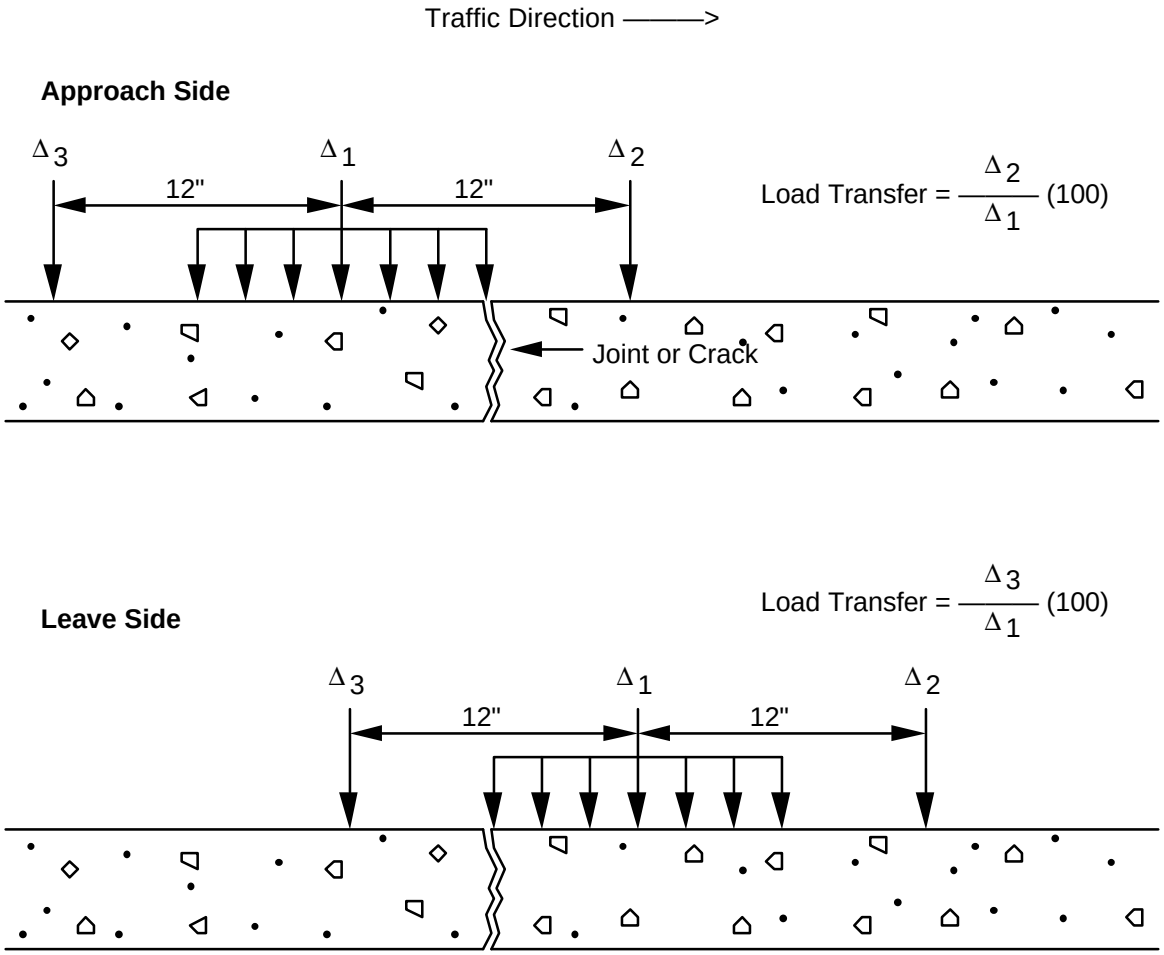


Figure 7.12. Arrangement of Deflection Sensors for Determining Load Transfer Efficiency at Approach and Leave Sides of a Joint or Crack

2.2.2.2(c) AASHTO Guide

The *AASHTO Guide for Design of Pavement Structures* [7.15, 7.27] contains guidelines for use in retrofitting dowel bars to restore load transfer at PCCP joints (Part III of the *AASHTO Guide*). The following items are noted:

- load transfer should be measured (i.e., with the WSDOT FWD) when the slabs are at their lower daily temperature (a time when load transfer is lowest due to slab contraction and upward slab curling),
- load transfer should be measured in the outer lane (which is usually the lowest on a multi-lane highway), and
- "load transfer restoration" should be considered for all transverse joints and cracks that exhibit measured deflection load transfer between 0 and 50 percent.

The *AASHTO Guide* recommends the use of dowel bars to restore load transfer. AASHTO recommends the following:

- Minimum dowel size should be 1 1/4 in. (32 mm) in diameter by 18 in. (457 mm) long.
- To limit faulting to 0.10 in. (2.5 mm) the following dowel designs are recommended:

Number of Dowels in Wheelpath	Dowel Diameter in. (mm)
2	1.625 (41 mm)
3	1.625 (41 mm)
4	1.250 (32 mm)
6	1.250 (32 mm)

2.2.2.2(d) FHWA Pavement Rehabilitation Manual

A draft chapter produced in 1994 for the FHWA Pavement Rehabilitation Manual [7.32] offers the following recommendations relative to retrofitting dowel bars to restore load transfer:

- Trigger values for retrofitting dowel bars
  - Faulting of individual joints or cracks of 3 mm or more
  - Deflection load transfer of 70 percent or less [note AASHTO Guide recommends 50 percent or less, see Paragraph 2.2.2.2(c) above]
  - Differential deflection between approach and leave slabs of over 0.25 mm
  - Cumulative faulting of joints and cracks over 525 mm/km
- Use three to five dowels in each wheelpath, spaced 300 mm apart.

- Locate the outermost dowel a maximum of about 300 mm from the outer lane edge, if the shoulder is asphalt surfaced or a tied PCC shoulder added separately.

- Dowel diameter should be 38 mm and epoxy coated.

2.2.2.2(e) Other State DOT Experience

A report prepared for the FHWA overviewed the experience gained in Florida from a retrofitted dowel bar project (Hall et al. [7.30]) constructed in 1986. The project is located on I-10 near Tallahassee, Florida, and extends for 0.8 km. The original PCC pavement is a 229 mm slab on a cement treated subbase with undowelled joints every 6.1 m. Two types of load transfer devices were used (epoxy coated dowel bars and double-V shear devices). The installed cost of each dowel bar was $62. The pre-retrofit LTE was generally less than 10 percent.

The results can be summarized as follows.

- Deflection

  The LTE increased from 10 percent to a range of 50 to 80 percent following installation of the dowel bars. Two dowel diameters were used 25 mm (1 in.) and 38 mm (1.5 in.). The larger diameter dowels had slightly higher LTEs but corner deflections which were 30 percent lower than joints with the 25 mm (1 in.) dowels.

- Number of Dowels

  Either three or five dowels were used in each wheelpath in the Florida project. Both layouts performed about the same with regard to faulting.

- Cracking

  About five years after construction significant cracking occurred in the vicinity of the dowel bar installations. Based on subsequent discussion with the FHWA, it appears the dowels have suffered extensive corrosion. This corrosion, presumably, contributed to the observed cracking.

2.2.2.2(f) WSDOT I-90 Project

2.2.2.2(f)(i) Introduction

WSDOT has investigated some of the mechanisms causing faulting of the Interstate PCC pavements. Nine slabs have been lifted in various locations across the state (SR-5: Ridgefield vicinity and the Weigh Station at the Pierce/King County Line, SR-90: Easton Hill and Kachess River to Yakima River) and the underlying material investigated. From this analysis it was determined that voids developed only in locations where the underlying base material was CTB. All other locations, where the base course is CSTC, fine-grained material has mitigated to the top of the base course, resulting in a wedge of such material directly beneath the slab, presumably causing a reduction in slab

support. Therefore, it was concluded that PCC slab rehabilitation would require much more than slab subsealing. (Refer to Pierce and Korynta [7.31] for additional details.)

A PCC rehabilitation test section was established on westbound SR-90, milepost 77.35 to milepost 78.11, which is between West Nelson Siding Road and the Little Creek Bridge, west of Cle Elum, Washington. The test section is being used to evaluate the performance of three experimental features and a control: retrofitted dowel bars, a four foot wide tied and doweled concrete shoulder, retrofitted dowel bars and a four foot wide tied and doweled concrete shoulder, and a control section. The test section was also included in the diamond grinding project which was from milepost 69.52 to milepost 102.49 (westbound only). Construction was completed during September 1992. Test section layout, faulting and cracking measurements, and typical FWD testing locations will follow.

The retrofitted dowel bars were placed at the mid-depth of the existing PCC slabs (with a minimum of 0.5 in. undercut beneath the dowel bar). The epoxy coated dowel bars were 1.5 in. in diameter by 18 in. long. The added PCC shoulders were 4 ft wide, tied to the existing 12 ft wide PCC slabs by use of epoxy coated No. 5 tie bars spaced 12 in. apart. The PCC shoulder contraction joints matched the existing main lanes (15 ft). Further, the shoulder contraction joints were doweled with the same size dowel bars as used for the retrofit and were spaced 12 in. apart. Other experimental features include (refer to Figure 7.13):

Test Section Layout

-----> Westbound

Section D	Section C	Section B	Section A	Section D
Control	Concrete Shoulders Only	Retrofit Dowel Bars and Concrete Shoulders	Retrofit Dowel Bars Only	Control
32 Slabs	68 Slabs	69 Slabs	67 Slabs	34 Slabs

MP 77.399 77.308 77.115 76.919 76.728 76.632

Figure 7.13. SR-90 Experimental Layout — Retrofit Dowel Bars and Tied PCC Shoulders

- Section A (Retrofit Dowel Bars Only)
  - Four dowel bars in each wheelpath spaced 12 in. apart. The first dowel in the outer wheelpath was placed 12 in. from the lane/shoulder edge. The first dowel in the inner wheelpath was placed 2 ft from the longitudinal joint with the adjacent lane. The test section length was about 1,000 ft long.
- Section B (Retrofit Dowel Bars and Tied PCC Shoulders)
  - The same number and configuration for the dowel bars was used as described for Section A.
  - The added PCC shoulders were the same thickness as the existing PCCP mainlanes (9 to 10 in. thick).
  - This section was approximately 1,000 ft in length.
- Section C (Tied PCC Shoulders Only)
  - The tied PCC shoulders were the same as described for Section B.
  - The section length was about 1,000 ft long.
- Section D (Control—Existing PCCP)
  - The control section was split into approximately 500 ft long sections at the beginning and end of the experimental features. This existing, undoweled PCC has a contraction joint spacing of 15 ft with slab thicknesses ranging from 9 to 10 in. The existing shoulder is ACP on crushed surfacing.

Figure 7.14 shows additional dowel bar and tied shoulders details—Figure 7.14a shows the transverse joint dowel bar layout and Figure 7.14b the tied PCC shoulder detail.

The existing distress of the PCCP outside lane were a few slabs with single transverse cracks (mid-panel) and contraction joint faulting. The cracking and faulting were relatively uniform between Sections A, B, C, and D. The faulting ranged from a minimum of 1/16 in. to a maximum of 5/8 in. with an overall average of about 5/16 in.

Generally, a fault of 0.20 in. (about 3/16 in.) or more is considered "critical" [7.17].

2.2.2.2(f)(ii) Load Transfer Analysis

The "before" and "after" FWD testing was used to calculate LTEs as shown in Tables 7.7a and 7.7b. The "before" construction testing was done during July 1992 and "after," or post construction, testing during September 1992. On the FWD test days, the deflection measurements were obtained between the hours of 4:00 a.m. and 9:30 a.m.

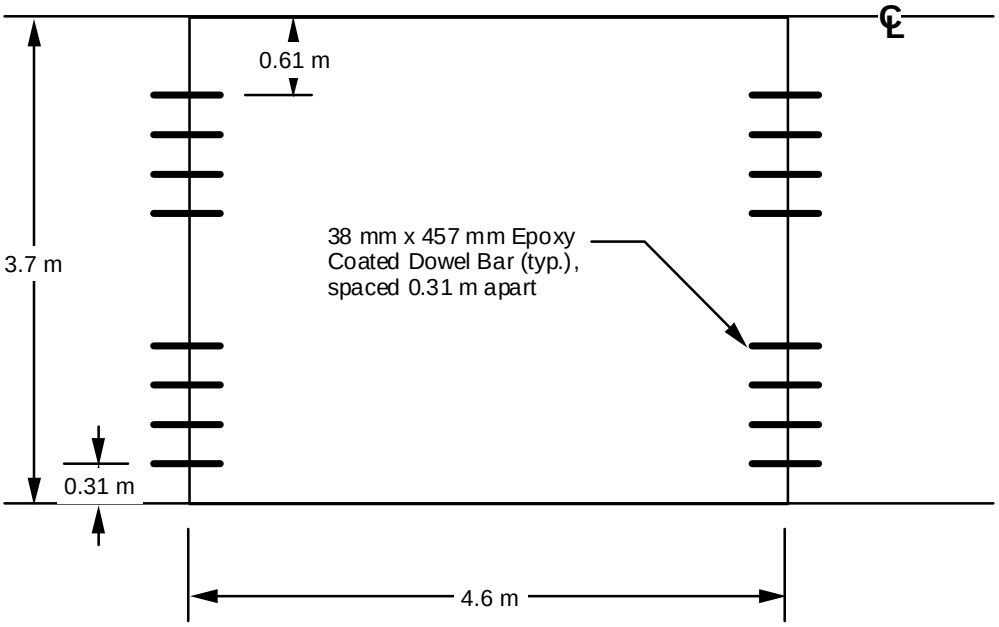


Figure 7.14a. Dowel Bar, I-90, MP 76.919 - 76.728, Westbound

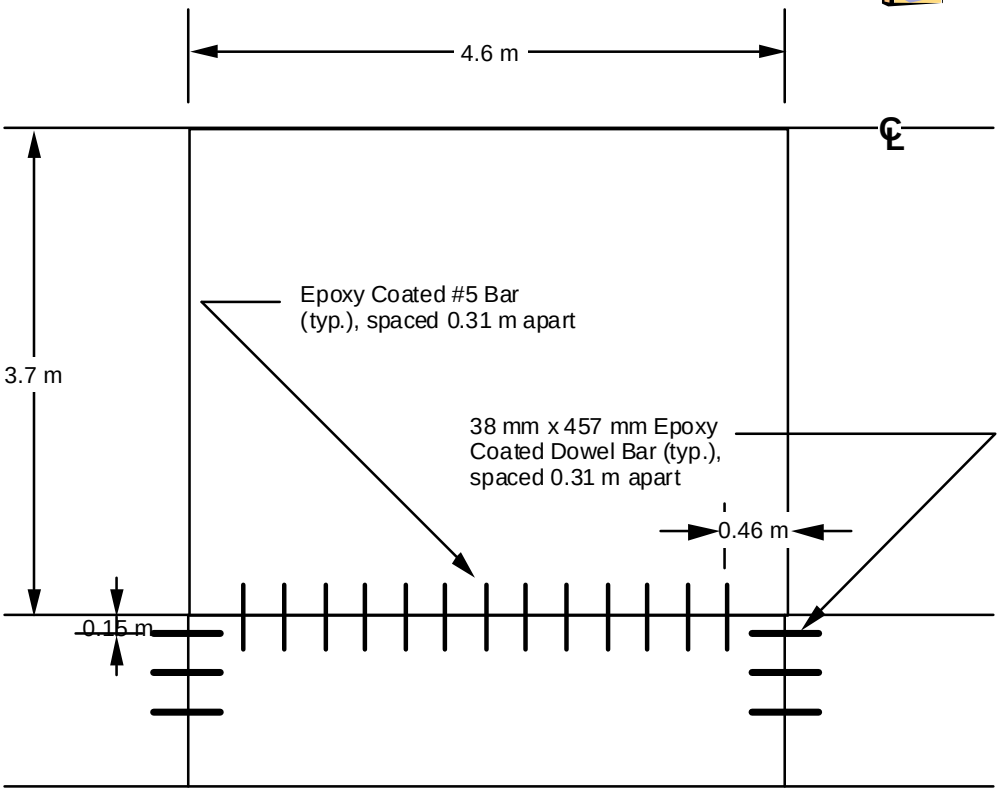


Figure 7.14b. Tied Shoulder Layout — I-90, MP 77.308 - 77.115, Westbound

Table 7.7a. Load Transfer Efficiencies for I-90 Test Site (MP 77.35 to 78.11) - Rows 1, 2, 3

Section	Experimental Feature	Row 1				Row 2				Row 3			
		Before 7/92	After 9/92	6 mo. 3/93	12 mo. 7/93	Before 7/92	After 9/92	6 mo. 3/93	12 mo. 7/93	Before 7/92	After 9/92	6 mo. 3/93	12 mo. 7/93
A	Dowel Bars Only												
	Average	33	81	82	85	37	89	92	93	29	89	92	93
	Std Dev	15	6	5	5	18	4	3	3	10	4	3	3
B	Dowel Bars and Tied PCC Shoulders												
	Average	41	82	88	86	38	86	90	92	39	86	91	92
	Std Dev	23	6	4	7	25	5	4	4	20	5	3	3
C	Tied PCC Shoulders Only												
	Average	27	47	72	74	22	36	63	69	41	48	81	82
	Std Dev	13	12	17	17	11	13	19	21	24	19	9	15
D	Control (Do Nothing but Grind)												
	Average	52	31	50	42	45	32	59	50	58	41	79	77
	Std Dev	24	15	15	17	14	15	16	16	24	19	10	18

Table 7.7b. Load Transfer Efficiencies for I-90 Test Site (MP 77.35 to 78.11) – Six years of Performance – Row 1 Only

Test Date	Section							
	A		B		C		D	
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
July 1992	31	12	40	23	27	13	53	25
Sept 1992	81	6	81	6	45	12	30	12
March 1993	82	5	88	4	74	14	50	13
June 1993	86	5	87	6	75	16	40	15
May 1994	85	6	83	7	40	15	31	11
Sept 1994	79	8	85	8	40	14	31	10
March 1995	85	4	86	7	66	16	58	17
April 1996	82	6	83	8	44	12	60	16
April 1997	88	6	84	19	46	15	57	23
April 1998	77	5	78	7	40	11	59	12

A: Dowel bars only
B: Dowel bars and tied PCC shoulders
C: Tied PCC shoulders only
D: Control (do nothing but grind)

Table 7.7c. Average Fault Measurments (mm) for I-90 Test Site (MP 77.35 to MP 78.11)

Test Date	Section							
	A		B		C		D	
	Average	Std Dev	Average	Std Dev	Average	Std Dev	Average	Std Dev
July 1992	10.2	2.5	8.6	2.3	6.6	1.5	7.4	2.3
March 1993	0.3	0.8	0.3	0.8	0.5	0.8	1.5	0.8
March 1995	0.0	0.0	0.0	0.3	0.3	0.5	2.5	0.8
April 1998	0.3	0.8	0.3	0.5	1.5	1.5	2.3	1.8

to preclude large thermal gradients in the slabs (the more critical condition at transverse joints (larger deflections) occurs when the slabs are warped upward which is due to a lower surface slab temperature and a higher bottom slab temperature). The limiting (maximum) thermal gradient was set at 30°F between the top and bottom of the slab (or about 3°F per inch of slab).

All FWD testing was done in the outside lane. SR 90 has two westbound lanes at this location. As noted in Figure 7.15, Row 1 was located on the outside edge of the PCC slabs, Row 2 the outside wheelpath and Row 3 the inside wheelpath.

The results are encouraging. The retrofitted dowel bars in Sections A and B generally increased the LTEs from a range of about 30 to 40 percent (low LTEs) to 80 to 90 percent (good LTEs). The tied PCC shoulders without retrofitted dowel bars (Section C) increased the LTEs from the 20 to 40 percent range to about the 35 to 50 percent range (not a substantial increase in LTE). A review of Table 7.7b shows that six years after construction, the retrofitted dowel bar sections (A and B), continue to show significant load transfer differences over the non-doweled sections (C and D). This is supported by faulting measurements shown in Table 7.7c. Figure 7.16 shows transverse joint fault measurements (before construction and six months later).

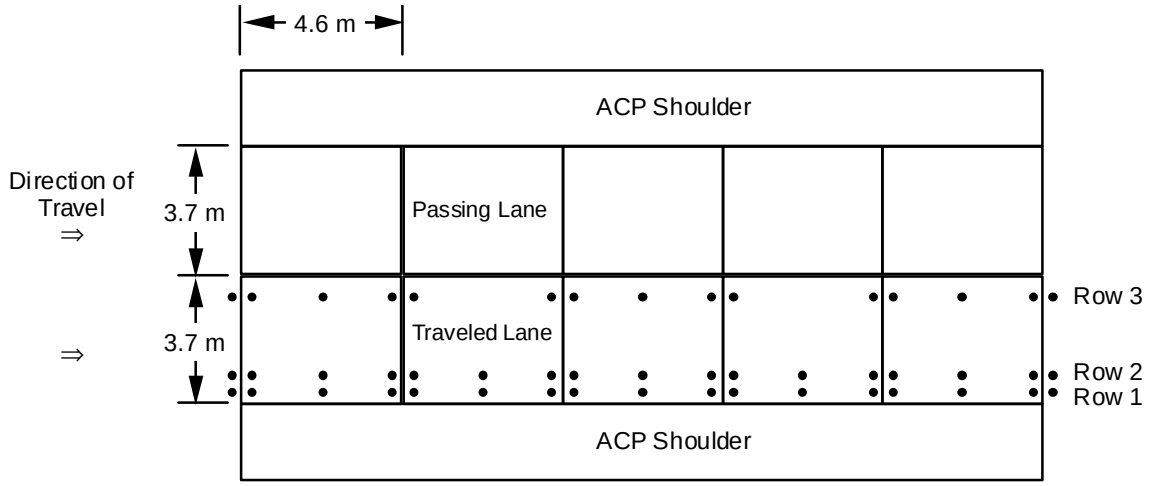
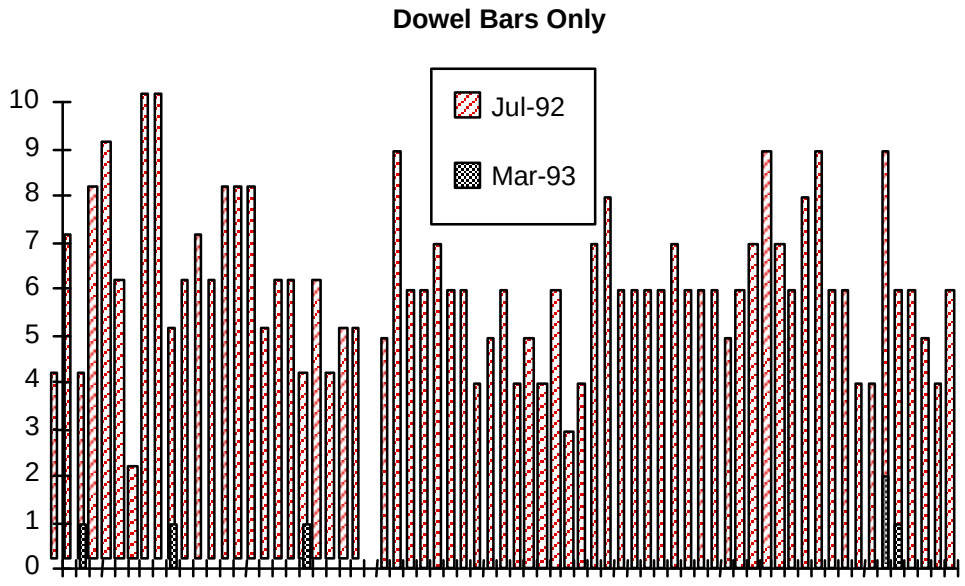


Figure 7.15. Typical FWD Testing Locations for I-90 PCCP Rehabilitation Test Section, Vicinity of MP 77 (Westbound)



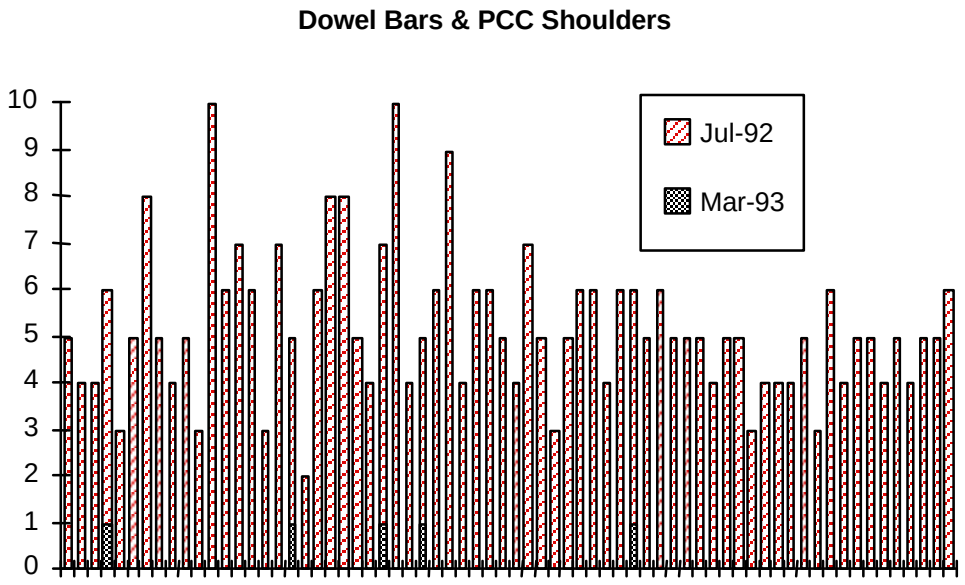


Figure 7.16. Fault Measurements for I-90 PCCP Rehabilitation Test Section

An additional measure of joint performance is the differential deflection between the approach and leave slabs. This was measured for the test sections by use of the WSDOT FWD and taking the difference in Δ_1 values with the load plate on the approach slab and then the leave slab (essentially as shown in the upper and lower portions of Figure 7.12). The FHWA "trigger value" for retrofitting slabs with dowel bars is 10 mils (0.25 mm) as described in Paragraph 2.2.2.2(d). The measured mean differential deflections for Sections A through D follow:

Test Date	Mean Differential Deflection (mils) by Section			
	A	B	C	D
July 1992	2.6	4.0	5.4	2.1
Sept 1992	0.7	0.4	0.6	1.8
March 1993	0.5	0.3	0.7	1.5
June 1993	0.6	0.4	0.6	2.6
May 1994	0.6	0.3	2.9	4.6
Sept 1994	1.0	0.8	4.0	5.1

Thus, at no time have the differential deflections exceeded the FHWA criterion. It is of interest how these deflections were decreased for doweled sections (A and B).

New retrofitted dowel bars appear to be an effective PCC rehabilitation treatment. The addition of tied PCC shoulders is of marginal benefit as characterized by the initial FWD testing. Long-term performance may indicate a different conclusion (both about tied PCC shoulders and retrofitted dowel bars). Only time, traffic and more FWD measurements will verify or modify these initial observations.

2.2.3 RESURFACING

Several of the more appropriate structural overlays that can be applied to concrete pavements are:

- Conventional thick AC overlay (with or without reflection cracking reduction treatments)
- Thick AC overlay over cracked and seated PCC slabs
- Unbonded PCC overlay

Other factors which must be considered and evaluated during the rehabilitation selection process include:

Clearances (vertical and horizontal)	Slab removal and regrading may be necessary to maintain appropriate clearances for an overlay option. Slab removal is expensive and changes the construction work sequence.
Drainage (surface and subsurface)	Both surface and subsurface drainage situations must be considered. Improvements or replacement of the subdrainage system may be needed, and if so, slab removal will be the only solution. In addition, any improvements to the roadbed soil will also require slab removal. Effective surface drainage can be insured through proper joint maintenance, which can also do much to avoid joint and slab problems.
Alignment (possible shifts)	Horizontal shift in the alignment can cause existing crown and joints to fall other than on the lane lines. There may be a need for slab removal.
Adding Lanes (tying shoulders — doweling)	These considerations may suggest slab replacement if the existing is only 8 to 8.5 in. thick. Truck lanes and increased traffic may raise equivalent axle load estimates. Construction timing of projects involving adding lanes should be planned in conjunction with any possible future restoration.
Structural Analysis (foundation considerations)	Foundation/soils reports will need to be accurate and complete. These reports should include, at a minimum: a pavement analysis study which incorporates the use of the Falling Weight Deflectometer (FWD), the determination of joint and/or crack load transfer, and whether or not slab tipping/rocking is present. In addition, the foundation/soils reports must consider rehabilitation strategies that will extend over a performance period of 30 or more years.
Traffic (Can you realistically work on this roadway in the future?)	With the future growth projections on all state routes, it will be increasingly difficult to conduct slab replacement in the future. Construction of AC overlays will need to occur during non-peak hours (night paving). Another question to address is whether a relatively low maintenance, long performance pavement (such as PCC) is best to place (or reconstruct) now rather than later on those highways which are unique with respect to high traffic growth (current or potential growth) and would incur large traffic delays.

2.2.4 RECONSTRUCTION (INCLUDING RECYCLING)

A pavement that is allowed to deteriorate eventually reaches a state of such advanced deterioration that even thick asphalt concrete overlays cannot compete cost effectively with reconstruction.

Reconstruction may require a longer lane closure duration than resurfacing since time for pavement breakup and removal operations are included. The difference might not be significant, however, when compared to a resurfacing option which requires extensive pre-overlay repair. Other concerns include condition of the base, subbase, and subgrade. If these layers can be left in place, surface removal and reconstruction may be conducted reasonably quick. However, if these layers are in poor condition (i.e., saturated or contaminated) and must be replaced or reworked, excessive costs and delays may result. An unbonded PCC overlay might be preferable to reconstuction under such conditions when drainage and engineering needs can be adequately addressed.

A variety of concerns not directly related to the condition of the pavement may come into play when considering reconstruction. These include improving geometric conditions, changing the roadway alignment, adding traffic lanes, constructing a pavement with a better design (perhaps including a drainage layer), reducing maintenance needs and even consideration for future total facility improvements. On some high volume routes, reconstruction may be the only rehabilitation alternative which can provide the performance life required of the system, due to the high impact of traffic volumes and the difficulty in returning to these sites after 10 to 15 years for additional rehabilitation. When high volume, high risk routes are reconstructed, route continuity and extended service life in the 30-50 year range may be most appropriate.

2.3 PCC REHABILITATION ALTERNATIVES (PERFORMANCE LIFE)

The more common structural resurfacing or reconstruction rehabilitation alternatives are usually: remove and replace with new PCCP, remove and replace with new ACP, or overlay with either ACP or PCCP. All details regarding the design of the alternatives shall be coordinated with the District Materials Engineer and the Pavement Design Engineer.

2.4 OTHER CONSIDERATIONS

With approximately half of the Interstate lane miles paved with PCC, the restoration, resurfacing, and reconstruction decisions WSDOT will face in the future will be significant. The WSDOT staff will have to keep abreast of the latest technology. To do this it will be necessary to maintain a keen awareness of new developments in PCC pavement rehabilitation strategies, their successes, and importantly, their failures. The only way that WSDOT can hope to be cost effective in maintaining PCC pavement serviceability is through employing the best methods. Good communication with other states, the national research organizations, and with major educational institutions will play a major role. Interaction with the industry will also be an important component in

this process. The WSDOT/industry interface must be cultivated to produce not only the most effective fixes, but to receive the most competitive bids. Feedback is essential.

Other issues to consider include the following:

- Costs involved with recycling the PCC material versus the cost of disposal
- Environmental costs (i.e., noise, air quality)
- Is night work appropriate? Are there plants in urban areas that can produce at night?
- Accident potential
- Improved ride (lower user costs)
- Impact to maintenance activities
- Mode shifts
- HOV usage
- Availability of alternate routes (diversions)
- Trip avoidance

3. NONDESTRUCTIVE TESTING INTERPRETATION TECHNIQUES

3.1 INTRODUCTION

This portion of SECTION 7.0 will be oriented toward techniques which can be used to interpret NDT data from the WSDOT FWD. Clearly, there are all kinds of NDT data which can be collected on or about pavements but concentration is placed on measured surface deflections.

Much of the material in Paragraph 3 was taken directly from the notes "Pavement Moduli Backcalculation Shortcourse" [7.19].

3.1.1 DEFLECTION BASIN PARAMETERS

Over the years numerous techniques have been developed to analyze deflection data from various kinds of pavement deflection equipment. A fairly complete summary of deflection basin parameters was provided by Horak at the Sixth International Conference Structural Design of Asphalt Pavement [7.20] and is shown in Table 7.8.

Table 7.8. Summary of Deflection Basin Parameters [modified from Ref. 7.20]

Parameter	Formula	Measuring device
Maximum deflection	D_0	Benkelman Beam, Lacroux deflectometer, FWD
Radius of curvature	$R = \frac{r^2}{2\,D_0(D_0/D_r - 1)}$ r = 5"	Curvaturemeter
Spreadability	$S = \frac{[(D_0 + D_1 + D_2 + D_3)/5]100}{D_0}$ $D_1 \ldots D_3$ spaced 12" apart	Dynaflect
Area	$A = 6[1 + 2\,(D_1/D_0) + 2\,(D_2/D_0) + D_3/D_0)$; 0, 1, 2, 3 ft	FWD
Shape factors	$F_1 = (D_0 - D_2) / D_1$ $F_2 = (D_1 - D_3) / D_2$	FWD
Surface curvature index	$SCI = D_0 - D_r$, where r = 12" or r = 20"	Benkelman Beam Road Rater FWD
Base curvature index	$BCI = D_{24"} - D_{36"}$	Road Rater
Base damage index	$BDI = D_{12"} - D_{24"}$	Road Rater
Deflection ratio	$Q_r = D_r/D_0$, where $D_r \quad D_0/2$	FWD
Bending index	BI = D/a, where a = Deflection basin	Benkelman Beam
Slope of deflection	$SD = \tan^{-1} (D_0 - D_r)/r$ where r = 24 in.	Benkelman Beam

All of these parameters tend to focus on four major areas:

(a) Plate or center load deflection which represents the total defection of the pavement. This was obviously the first deflection parameter which came with the Benkelman Beam. It has been used for many years as the primary input for several overlay design procedures.

(b) The slope or deflection differences close to the load such as Radius of Curvature (R), Shape Factor (F_1), and Surface Curvature Index (SCI). These parameters tend to reflect the relative stiffness of the bound or upper regions of the pavement section.

(c) The slope or deflection differences in the middle of the basin about 11.8 in. (300 mm) to 35.4 in. (900 mm) from the center of the load. These parameters tend to reflect the relative stiffness of the base or lower regions of the pavement section.

(d) The deflections toward the end of the basin. Deflections in this region relate quite well to the stiffness of the subgrade below the pavement surfacing.

The subsequent parameters to be presented were developed to provide a means of obtaining the resilient modulus values of the surfacing layers more easily or quickly than full backcalculation. In general, the success of these regression equations to predict the resilient modulus of the surfacing layers has been limited. There is a clear consensus; however, that the deflections measured beyond the primary effects of the load stress bulb relate quite well to the resilient modulus of the subgrade, (E_{SG}).

3.1.2 WSDOT EQUATIONS

Several researchers have developed regression equations to predict E_{SG} from plate load and deflections measured at distances from about 24 in. (600 mm) to 48 in. (1200 mm) from the center of the plate load. Newcomb developed such regression equations to predict E_{SG} as part of an overall effort to develop a mechanistic empirical overlay design procedure for WSDOT [7.21]. For <u>two layer</u> cases, the <u>subgrade modulus</u> can be estimated from:

$$E_{SG} = -466 + 0.00762\ (P/D_3), \qquad \text{(Eq. 7.3)}$$

$$E_{SG} = -198 + 0.00577\ (P/D_4), \qquad \text{(Eq. 7.4)}$$

$$E_{SG} = -371 + 0.00671\ (2P/(D_3 + D_4)), \qquad \text{(Eq. 7.5)}$$

and for three layer cases

$$E_{SG} = -530 + 0.00877\ (P/D_3), \qquad \text{(Eq. 7.6)}$$

$$E_{SG} = -111 + 0.00577\ (P/D_4), \qquad \text{(Eq. 7.7)}$$

$$E_{SG} = -346 + 0.00676\ (2P/(D_3 + D_4)) \qquad \text{(Eq. 7.8)}$$

The $R^2 \approx 99\%$ for all equations and the sample sizes were 180 (two layer case) and 1,620 (three layer case). Figures 7.17 and 7.18 are used to illustrate typical results from Equations 7.6 and 7.7.

These equations (and those to follow in Paragraph 3.1.2) were developed from generated data (ELSYM5) with the following levels of input data:

Two Layer Cases

Load, P, lb (kN)	Surface Thickness, h_{AC}, in. (mm)	Surface Modulus, E_{AC}, psi (MPa)	Subgrade Modulus, E_{SG}, psi (MPa)
5,000 (22)	2 (50)	2,000,000 (13800)	50,000 (345)
10,000 (44)	6 (150)	500,000 (3450)	30,000 (207)
15,000 (67)	12 (300)	100,000 (690)	10,000 (69)
	18 (450)		5,000 (35)
			2,500 (17)

Three Layer Cases

Load, P, lb (kN)	Surface Thickness, h_{AC}, in. (mm)	Base Thickness, h_B, in (mm)	Surface Modulus, E_{AC}, psi (MPa)	Base Modulus, E_B, psi (MPa)	Subgrade Modulus, E_{SG}, psi (MPa)
5,000 (22)	2 (50)	4 (100)	2,000,000 (13800)	100,000 (690)	50,000 (345)
10,000 (44)	6 (150)	10 (250)	500,000 (3450)	50,000 (345)	30,000 (207)
15,000 (67)	12 (300)	18 (450)	100,000 (690)	30,000 (207)	10,000 (69)
				10,000 (69)	5,000 (35)
					2,500 (17)

(assumed that load applied on a 11.8 in. (300 mm) diameter load plate)

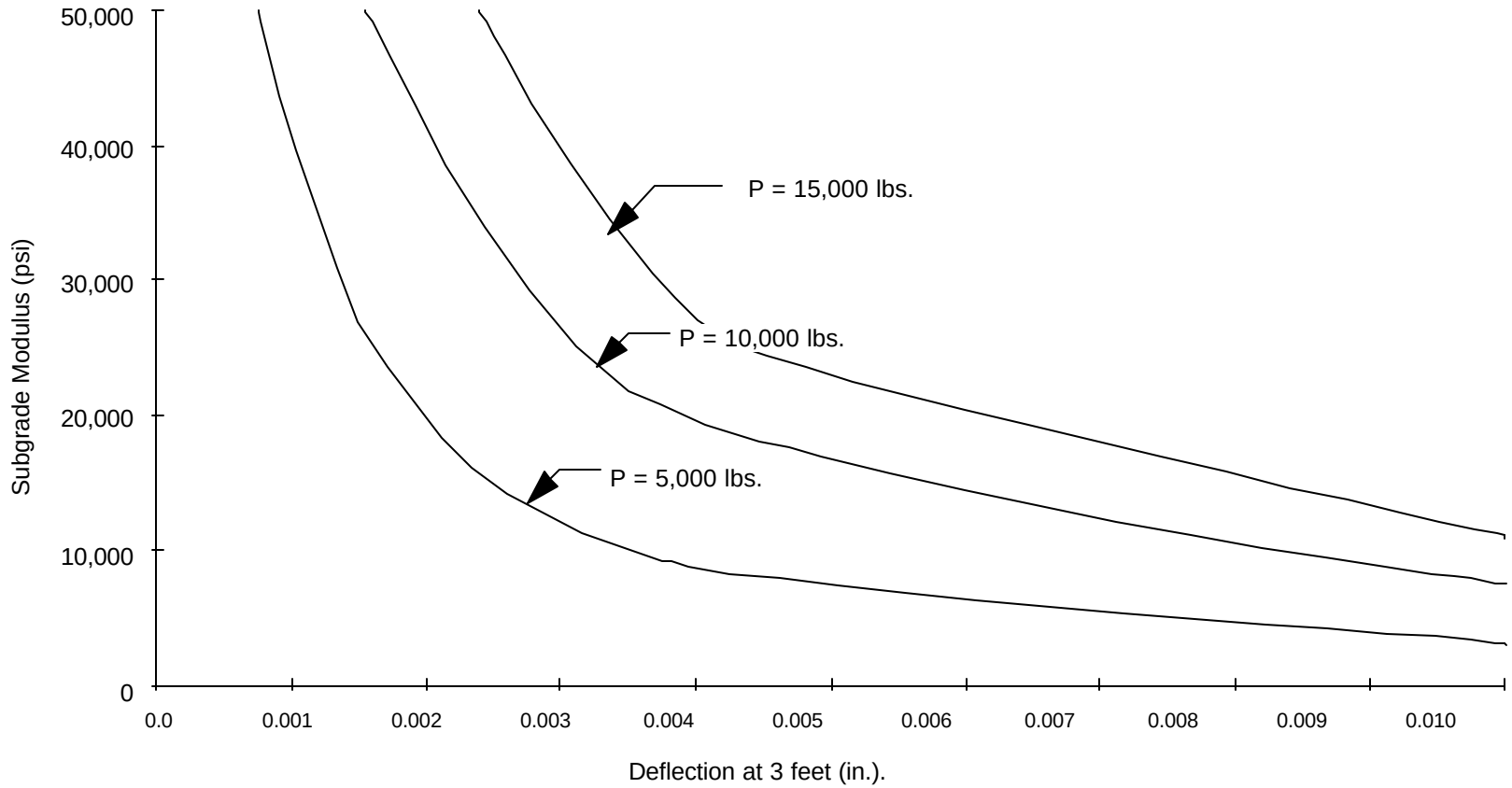


Figure 7.17. Deflection vs. Subgrade Modulus for Equation 7.6 (Three Layer Case – Deflection Measured at 3 ft) [from Newcomb 7.21]

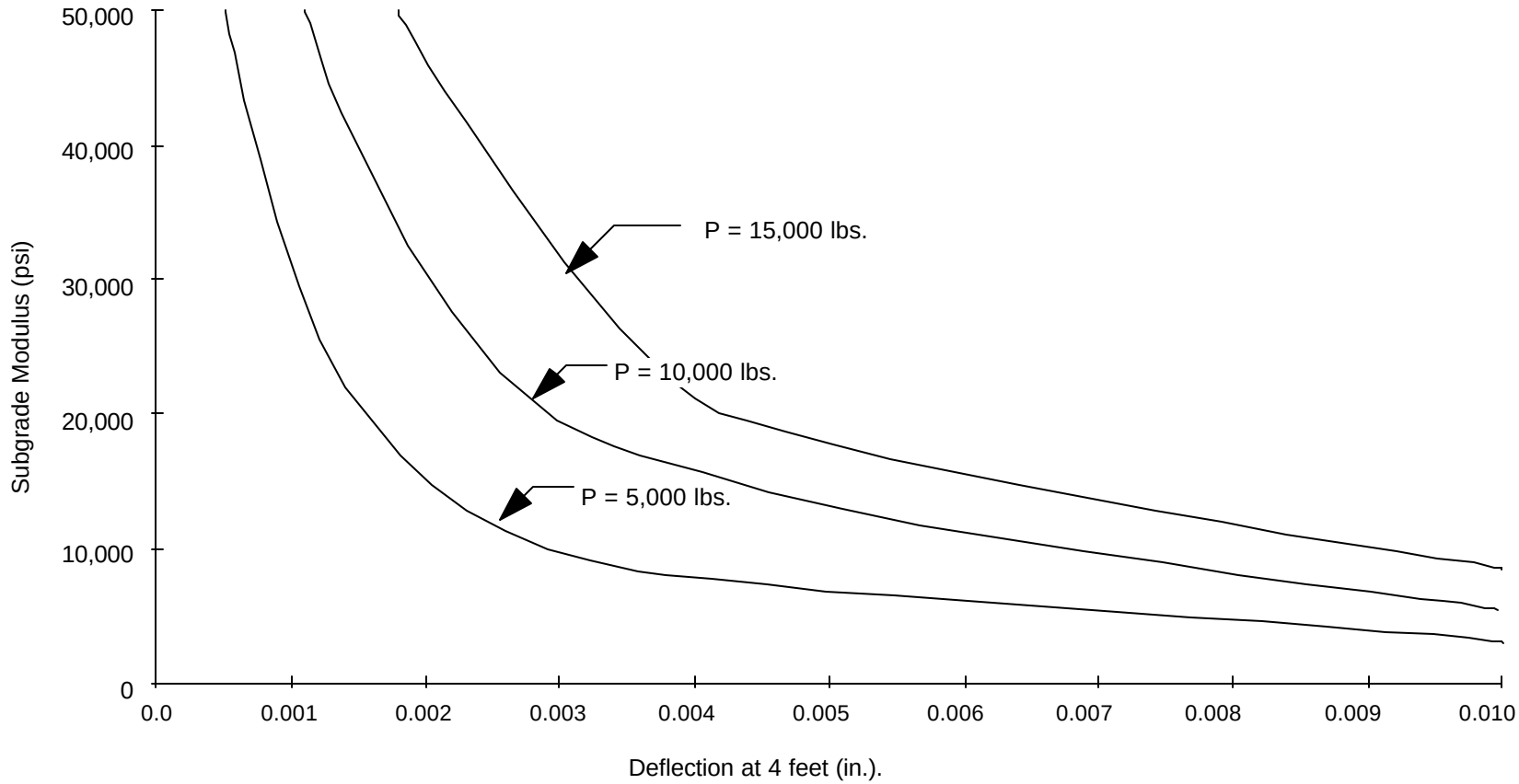


Figure 7.18. Deflection vs. Subgrade Modulus for Equation 7.7 (Three Layer Case – Deflection Measured at 4 ft) [from Newcomb 7.21]

From this generated data (no rigid base), regression equations were also developed for estimating the surface modulus (AC) for a two layer case (for example a "full-depth" pavement):

$$\log E_{AC} = -0.53740 - 0.95144 \log_{10} E_{SG} \qquad \text{(Eq. 7.9)}$$

$$-1.21181\sqrt[3]{h_{AC}} + 1.78046 \log_{10} (PA_1/D_0^2)$$

where $R^2 = 0.83$

For a three layer case, equations were developed for both E_{AC} and E_B as follows:

If both E_{AC} and E_B unknown:

$$\log E_{AC} = -4.13464 + 0.25726 (5.9/h_{AC}) \qquad \text{(Eq. 7.10)}$$

$$+ 0.92874 \sqrt{5.9/h_B} - 0.69727 \sqrt{h_{AC}/h_B} - 0.96687 \log_{10} E_{SG}$$

$$+ 1.88298 \log_{10} (PA_1/D_0^2)$$

where $R^2 = 0.78$.

$$\log E_B = 0.50634 + 0.03474 (5.9/h_{AC}) \qquad \text{(Eq. 7.11)}$$

$$+ 0.12541 \sqrt{5.9/h_B} - 0.09416 \sqrt{h_{AC}/h_B}$$

$$+ 0.51386 \log E_{SG} + 0.25424 \log_{10} (PA_1/D_0^2)$$

where $R^2 = 0.70$.

The following variables were used in the equations shown in Paragraph 3.1.2:

P	=	applied load (lbs) on a 11.8 in. (300 mm) plate,
h_{AC}	=	surface course thickness (in.),
h_B	=	base course thickness (in.),
E_{AC}	=	surface course modulus (psi),
E_B	=	base course modulus (psi),
E_{SG}	=	subgrade modulus (psi),
D_0	=	deflection under center of applied load (in.),
$D_{0.67}$	=	deflection at 8 in. (0.67 ft) from center of applied load (in.),
D_1	=	deflection at 1 ft from center of applied load (in.),

D_2 = deflection at 2 ft from center of applied load (in.),

D_3 = deflection at 3 ft from center of applied load (in.),

D_4 = deflection at 4 ft from center of applied load (in.), and

A_1 = approximate area under deflection basin out to 3 ft
= $2\,[2\,(D_0 + D_{0.67}) + (D_{0.67} + D_1) + 3(D_1 + D_2) + 3(D_2 + D_3)]$
= $4D_0 + 6D_{0.67} + 8D_1 + 12D_2 + 6D_3$

3.1.3 AASHTO EQUATIONS

Witczak presented a similar regression equation in Part III of the 1986 AASHTO Guide for Design of Pavement Structures [7.15]. That equation is in the form:

$$E_{SG} = (P)(S_f)/(D_r)(r) \qquad \text{(Eq. 7.12)}$$

where P = plate load (lb),

S_f = subgrade modulus prediction factor,

= 0.2686 for $\mu = 0.50$

= 0.2792 for $\mu = 0.45$

= 0.2892 for $\mu = 0.40$

= 0.2874 for $\mu = 0.35$

= 0.2969 for $\mu = 0.30$

D_r = pavement surface deflection measured at r distance, from the load, and

r = distance from the load to D_r (in.).

Using an S_f value of 0.2892 where the Poisson's ratio is 0.40, the equation reduces to the following equations for deflections measured at 2 ft (610 mm), 3 ft (914 mm), and 4 ft. (1,219 mm).

$$E_{SG} = 0.01205(P/D_2) \qquad \text{(Eq. 7.13)}$$

$$E_{SG} = 0.00803(P/D_3) \qquad \text{(Eq. 7.14)}$$

$$E_{SG} = 0.00603(P/D_4) \qquad \text{(Eq. 7.15)}$$

In the AASHTO Guide, detailed procedures are provided to insure that the deflections used to determine E_{SG} are outside the pressure bulb from the test load. To most accurately represent the subgrade stiffness; however, the deflections closest to the load without being directly effected by the pressure bulb should be used. Stiff underlying layers will have the greatest effect on deflections furthermost from the test load. For example in cases where total surfacing depths are around 11.8 in. (300 mm), deflections taken around 23.6 in. (600 mm) should be used to determine E_{SG}.

In a recent NCHRP study [7.16] which will be used to revise Part III of the AASHTO Pavement Guide [7.15], it is recommended that the following equation be used to solve for subgrade modulus:

$$M_R = P(1 - \mu^2)/(\pi)(D_r)(r) \qquad \text{(Eq. 7.16)}$$

where M_R = <u>backcalculated</u> subgrade resilient modulus (psi),

P = applied load (lbs),

D_r = pavement surface deflection a distance r from the center of the load plate (inches), and

r = distance from center of load plate to D_r (inches).

Using a Poisson's ratio of 0.40, Equation 7.16 reduces to

$$M_R = 0.01114\ (P/D_2) \qquad \text{(Eq. 7.17)}$$

$$M_R = 0.00743\ (P/D_3) \qquad \text{Eq. 7.18)}$$

$$M_R = 0.00557\ (P/D_4) \qquad \text{Eq. 7.19)}$$

for sensor spacings of 2 ft (610 mm), 3 ft (914 mm), and 4 ft (1219 mm). If a Poisson's ratio of 0.45 is used instead for the same sensor spacings, the equations become:

$$M_R = 0.01058(P/D_2) \qquad \text{(Eq. 7.20)}$$

$$M_R = 0.00705\ (P/D_3) \qquad \text{(Eq. 7.21)}$$

$$M_R = 0.00529\ (P/D_4) \qquad \text{(Eq. 7.22)}$$

Darter et al. [7.16] recommends that the deflection used for subgrade modulus determination should be taken at a distance at least 0.7 times r/a_e where r is the radial distance to the deflection sensor and a_e is the radial dimension of the applied stress bulb at the subgrade "surface." The a_e dimension can be determined from the following:

$$a_e = \sqrt{a^2 + \left(D\sqrt[3]{\frac{E_P}{M_R}}\right)^2}$$

where a_e = radius of stress bulb at the subgrade-pavement interface,

a = NDT load plate radius (inches),

D = total thickness of pavement layers (inches)

E_P = effective pavement modulus (psi), and

M_R = backcalculated subgrade resilient modulus.

For "thin" pavements (such as Section A, Figure 7.21), $a_e \simeq$ 15 in.; "medium" to "thick" pavements (Sections B and C), $a_e \simeq$ 26 to 33 in. Thus, the minimum r is usually 24 to 36 in. (recall $r \geq 0.7\ (a_e)$).

3.1.4 SOUTH AFRICAN EQUATIONS

A similar relation between deflections taken at 2000 mm (78.7 in.) was shown by Horak [7.20]:

$$\log_{10} E_{SG} = 9{,}727 - 0.989 \log_{10} \delta_{2000}$$

where E_{SG} = subgrade elastic modulus (Pa), (same as E_{SG}), and

δ_{2000} = deflection at a distance of 2000 mm from the (point) of loading (µm).

3.1.5 ADDITIONAL MODULUS ESTIMATES

3.1.5.1 Subgrade Soils

(a) Table 7.9 is used to show typical subgrade moduli as listed by Chou et al. [7.22].

(b) CBR Correlation

A classic correlation that has been widely used is

$$E_{SG}\ (\text{psi}) = 1{,}500\ (\text{CBR})$$

or $$E_{SG}\ (\text{MPa}) = 10\ (\text{CBR})$$

where CBR = California Bearing Ratio

Note: Limited to fine-grained soils with a CBR of 10 or <u>less</u>.

Table 7.9. Typical Values of Subgrade Moduli [after Ref. 7.22]

	Climate Condition			
			Wet - Freeze	
	Dry	Wet — No Freeze	Unfrozen	Frozen
Material	psi (MPa)	psi (MPa)	psi (MPa)	psi (MPa)
Clay	15,000 (103)	6,000 (41)	6,000 (41)	50,000 (345)
Silt	15,000 (103)	10,000 (69)	5,000 (34)	50,000 (345)
Silty or Clayey Sand	20,000 (138)	10,000 (69)	5,000 (34)	50,000 (345)
Sand	25,000 (172)	25,000 (172)	25,000 (172)	50,000 (345)
Silty or Clayey Gravel	40,000 (276)	30,000 (207)	20,000 (138)	50,000 (345)
Gravel	50,000 (345)	50,000 (345)	40,000 (276)	50,000 (345)

3.1.5.2 Granular Materials Shell Method [7.22, 7.23]

(a) Shell Method [7.22, 7.23]

The granular base modulus is a function of the subgrade modulus and base course thickness.

$$E_B = 0.2\,(25.4h_2)^{0.45}\,E_{SG}$$

where E_B =base modulus (psi),
h_2 =base thickness (in.)
E_{SG} =subgrade modulus (psi)

<u>Example</u>

If E_{SG} =15,000 psi and the base thickness = 10 in., then

$$E_B = 0.2\,(25.4\,(10))^{0.45}\,(15{,}000)$$
$$\simeq 36{,}000 \text{ psi } (250 \text{ MPa})$$

3.1.5.3 Stabilized Base Materials

From Chou et al. [7.22]:

- Asphalt stabilized materials: use information available for asphalt concrete.
- Lime-stabilized materials: a modulus range of 100,000 to 1,000,000 psi (690 to 6,895 MPa) is suggested.
- Cement stabilized materials: a range of 300,000 to 4,000,000 psi (2,070 to 27,600 MPa) is suggested.

3.1.5.4 Asphalt Concrete

From Chou et al. [7.22]:

- Absolute <u>minimum</u> and <u>maximum</u> values for AC are 50,000 to 2,000,000 psi (345 to 13,790 MPa).
- To estimate the lowest and highest AC moduli, use expected high and low temperatures along with the Asphalt Institute equation [4.6] which is of the form:

$$|E^*| = f(P_{200}, f, V_v, \eta_{70\,F}, T, P_{ac})$$

where
$|E^*|$ = dynamic modulus (stiffness of asphalt concrete),
P_{200} = percent aggregate passing No. 200 sieve,
f = frequency of loading,
V_v = percent air voids,
$\eta_{70\,F}$ = original absolute viscosity used in mix at 70 F (21.1 F)
T = temperature, and
P_{ac} = asphalt content, by weight of mix.

Chou et al. [7.22] use default values of

P_{200} = 6%
f = 25 H_z
V_v = 7%
$\eta_{70\,F}$ = 10^6 poises
P_{ac} = 5%

3.1.6 PCC JOINTS—LOAD TRANSFER EFFICIENCY

This was treated in adequate detail in subparagraph 2.2.2.2(b) (SECTION 7.0) and, as such, will not be repeated here.

3.2 INDICES FOR PROJECT ANALYSIS

3.2.1 INTRODUCTION

Over the years WSDOT has found that the use of selected indices and algorithms provide a "good picture" of the relative conditions found throughout a project. This picture is very useful in performing backcalculation and may at times be used by themselves on projects with large variations in surfacing layers.

WSDOT is currently using deflections measured at the center of the test load combined with Area values and E_{SG} computed from deflections measured at 24 in. (610 mm) presented in a linear plot to provide a visual picture of the conditions found along the length of any project (as illustrated in Figure 7.19).

3.2.2 AREA PARAMETER

The Area value represents the normalized area of a slice taken through any deflection basin between the center of the test load and 3 ft (914 mm). By normalized, it is meant that the area of the slice is divided by the deflection measured at the center of the test load, D_0. Thus the Area value is the length of one side of a rectangle where the other side of the rectangle is D_0. The actual area of the rectangle is equal to the area of the slice of the deflection basin between 0 and 3 ft (914 mm).

The original Area equation is:

$$A = 6(D_0 + 2D_1 + 2D_2 + D_3)/D_0$$

where D_0 = surface deflection at center of test load,
D_1 = surface deflection at 1 ft (305 mm),
D_2 = surface deflection at 2 ft (610 mm), and
D_3 = surface deflection at 3 ft (914 mm).

The approximate metric equivalent of this equation is:

$$A = 150(D_0 + 2D_{300} + 2D_{600} + D_{900})/D_0$$

where D_0 = deflection at center of test load,
D_{300} = deflection at 300 mm,
D_{600} = deflection at 600 mm, and
D_{900} = deflection at 900 mm.

Figure 7.20 shows the development of the normalized area for the Area value using the Trapezoidal Rule to estimate area under a curve.

WSDOT Non-destructive Pavement Testing
SR 510; MP 0.0 - 7.60; EB; Case #1a

Note: Summary values are normalized to a 9,000 pound load and adjusted for pavement thickness and temperature. Modulus determination is based on the deflection at the 4th sensor (2 feet from load)

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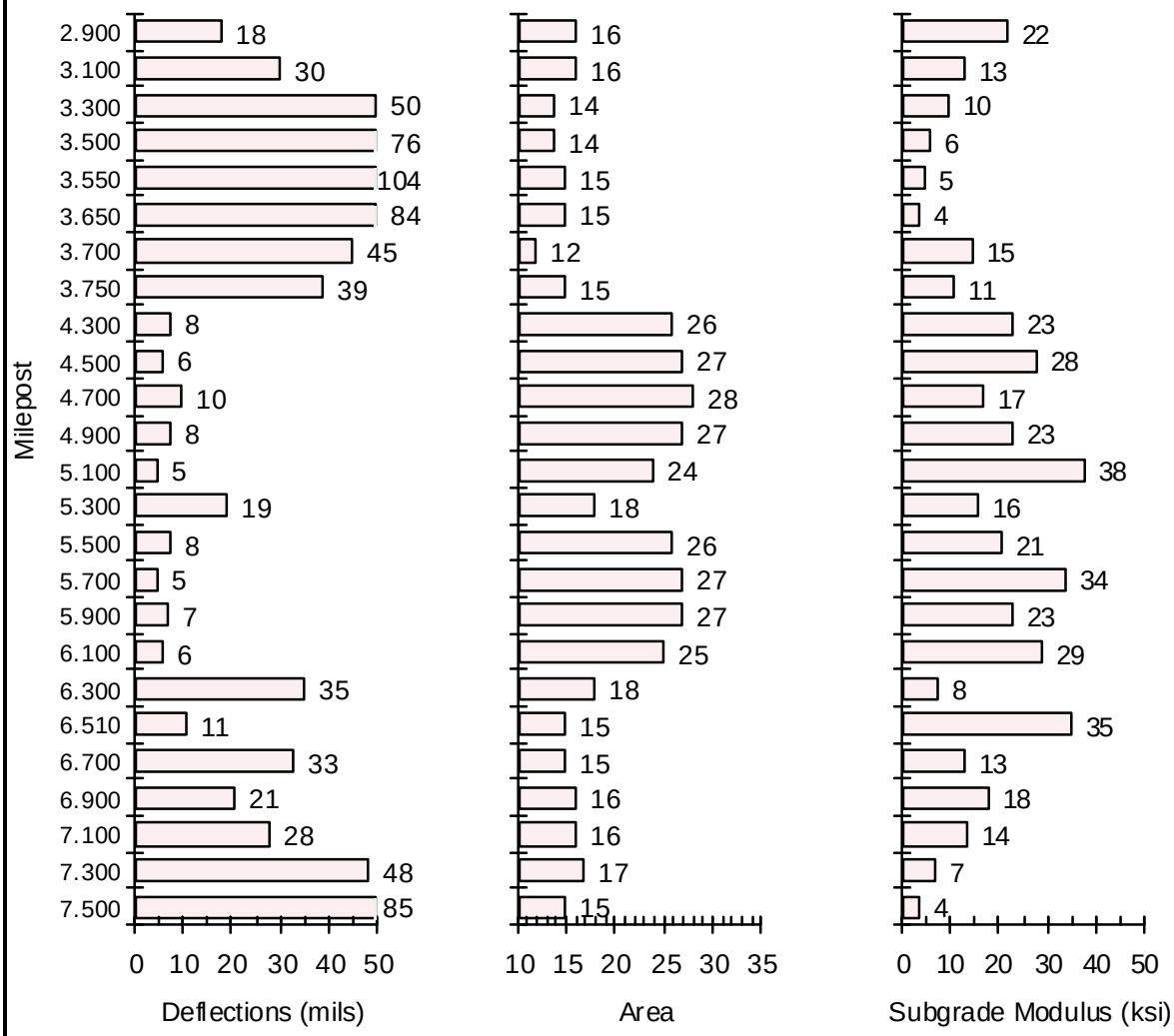

Figure 7.19. Illustration of Basic NDT Parameters as Used by WSDOT [SR 510 MP 2.9 to MP 7.5]

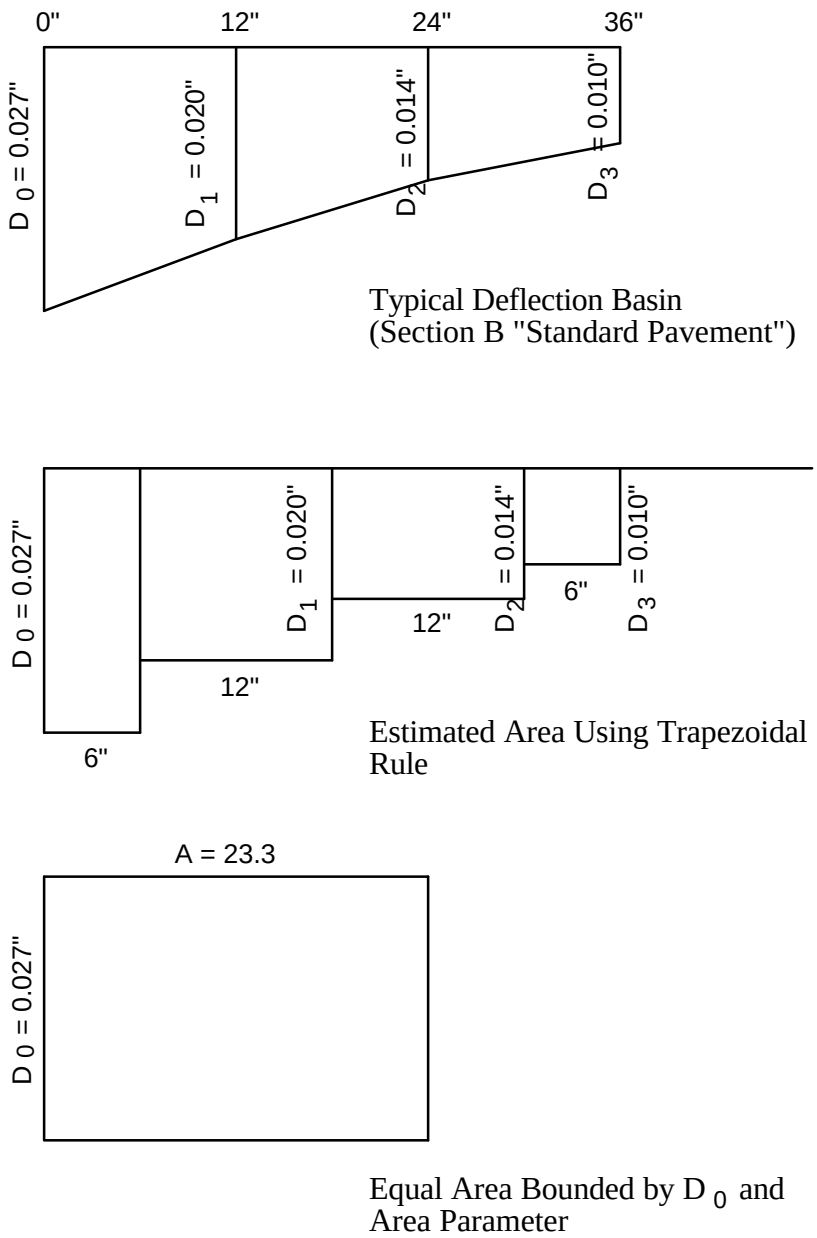

Figure 7.20. Computing an Area Parameter

The basic Trapezoidal Rule is:

$$K = h\left(\frac{1}{2}y_0 + y_1 + y_2 + \frac{1}{2}y_3\right)$$

where K = any planar area,
y_0 = initial chord,
y_1, y_2 = immediate chords,
y_3 = last chord, and
h = common distance between chords.

Thus, to estimate the area of a deflected basin using D_0, D_1, D_2, and D_3, and h = 6 in., then:

$$K = 6\,(D_0 + 2D_1 + 2D_2 + D_3)$$

Further, normalize by dividing by D_0:

$$\text{Area} = \frac{6}{D_0}(D_0 + 2D_1 + 2D_2 + D_3) \quad \text{or} \quad \text{Area} = 6\left(1 + \frac{2D_1}{D_0} + \frac{2D_2}{D_0} + \frac{D_3}{D_0}\right)$$

Thus, since we normalized by D_0, the Area Parameter's unit of measure is inches (or mm) not in^2 or mm^2 as one might expect.

The <u>maximum</u> value for Area is 36.0 (915 mm) and occurs when all four deflection measurements are equal (not likely to actually occur) as follows:

If, $D_0 = D_1 = D_2 = D_3$

Then, Area = 6(1 + 2 + 2 + 1) = 36.0 in.

For all four deflection measurements to be equal (or nearly equal) would indicate an <u>extremely stiff</u> pavement system (like portland cement concrete slabs or thick, full-depth asphalt concrete.)

The <u>minimum</u> Area value should be no less than 11.1 in. (280 mm).

This value can be calculated for a one-layer system which is analogous to testing (or deflecting) the top of the subgrade (i.e., no pavement structure). Using appropriate equations, the ratios of

$$\frac{D_1}{D_0}, \frac{D_2}{D_0}, \frac{D_3}{D_0}$$

always result in 0.26, 0.125, and 0.083, respectively. Putting these ratios in the Area equation results in

$$\text{Area} = 6(1+ 2(0.26) + 2(0.125) + 0.083) = 11.1 \text{ in.}$$

Further, this value of Area suggests that the elastic moduli of any pavement system would all be equal (e.g., $E_1 = E_2 = E_3 = \ldots$). This is highly unlikely for actual, inservice pavement structures. Low area values suggest that the pavement structure is not much different from the underlying subgrade material (this is not always a bad thing if the subgrade is extremely stiff — which doesn't occur very often).

Typical Area values were computed for pavement Sections A, B, and C (refer to Figure 7.21) and are shown in Table 7.10 (along with the calculated surface deflections (D_0, D_1, D_2, D_3)). Table 7.11 provides a general guide in the use of Area values obtained from FWD pavement surface deflections.

Pavement	Area		Pavement	Area	
	in.	(mm)		in.	(mm)
• PCCP • "Sound" PCC*	24-33 29-32	(610-840) (740-810)	• BST flexible pavement (relatively thin structure)	15-17	(380-430)
• Thick ACP (0.35 ft ACP)	21-30	(530-760)	• Weak BST	12-15	(300-380)
• Thin ACP (< 0.35 ft ACP)	16-21	(410-530)			

* from 1993 AASHTO Guide, p. III-117

3.3 FUNDAMENTALS OF BACKCALCULATION

3.3.1 INTRODUCTION

A method which has received recent, extensive development work is the "backcalculation" method. This is essentially a mechanistic evaluation (usually elastic analysis — refer to SECTION 4.0 of this Guide) of pavement surface deflection basins generated by various pavement deflection devices. Backcalculation is one where measured and calculated surface deflection basins are matched (to within some tolerable error) and the associated layer moduli required to achieve that match are determined. The backcalculation process is usually iterative and normally done with software that can run on microcomputers. An illustration of the backcalculation process is shown in Figures 7.22 and 7.23.

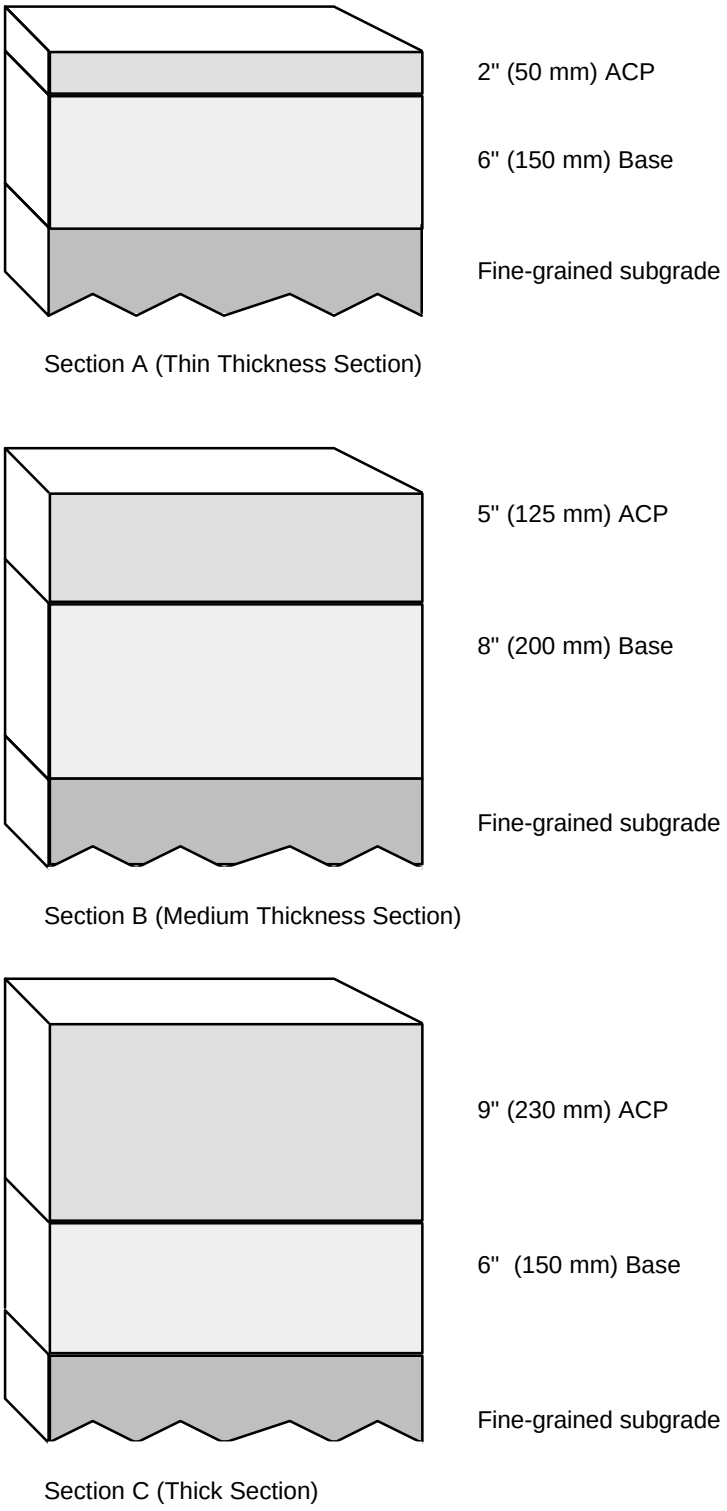

Figure 7.21. "Typical" Pavement Sections

Table 7.10. Estimates of Area from Pavement Sections — Cases A, B, and C

Pavement Cases	Pavement Surface Deflections, inches				Area	
	D_0	D_1	D_2	D_3	in.	(mm)
Standard Pavement						
Section A (thin)	0.048	0.026	0.014	0.009	17.1	(434)
Section B (med)	0.027	0.020	0.014	0.010	23.3	(592)
Section C (thick)	0.018	0.015	0.012	0.009	27.0	(686)
Stabilize Subgrade						
Section A (thin)	0.036	0.020	0.013	0.009	18.5	(470)
Section B (med)	0.023	0.017	0.012	0.009	23.5	(597)
Section C (thick)	0.016	0.013	0.011	0.009	27.4	(696)
Asphalt Treated Base						
Section A (thin)	0.021	0.018	0.013	0.010	26.6	(676)
Section B (med)	0.014	0.012	0.010	0.009	28.7	(729)
Section C (thick)	0.012	0.011	0.009	0.008	30.0	(762)
Moisture Sensitivity						
Section A (thin)	0.053	0.026	0.014	0.009	16.1	(409)
Section B (med)	0.033	0.022	0.014	0.009	20.7	(526)
Section C (thick)	0.024	0.018	0.013	0.010	24.0	(610)

Table 7.11. Trends of D_0 and Area Values

FWD Based Parameter		Generalized Conclusions*
Area	Maximum Surface Deflection (D_0)	
Low	Low	Weak structure, strong subgrade
Low	High	Weak structure, weak subgrade
High	Low	Strong structure, strong subgrade
High	High	Strong structure, weak subgrade

*Naturally, exceptions can occur

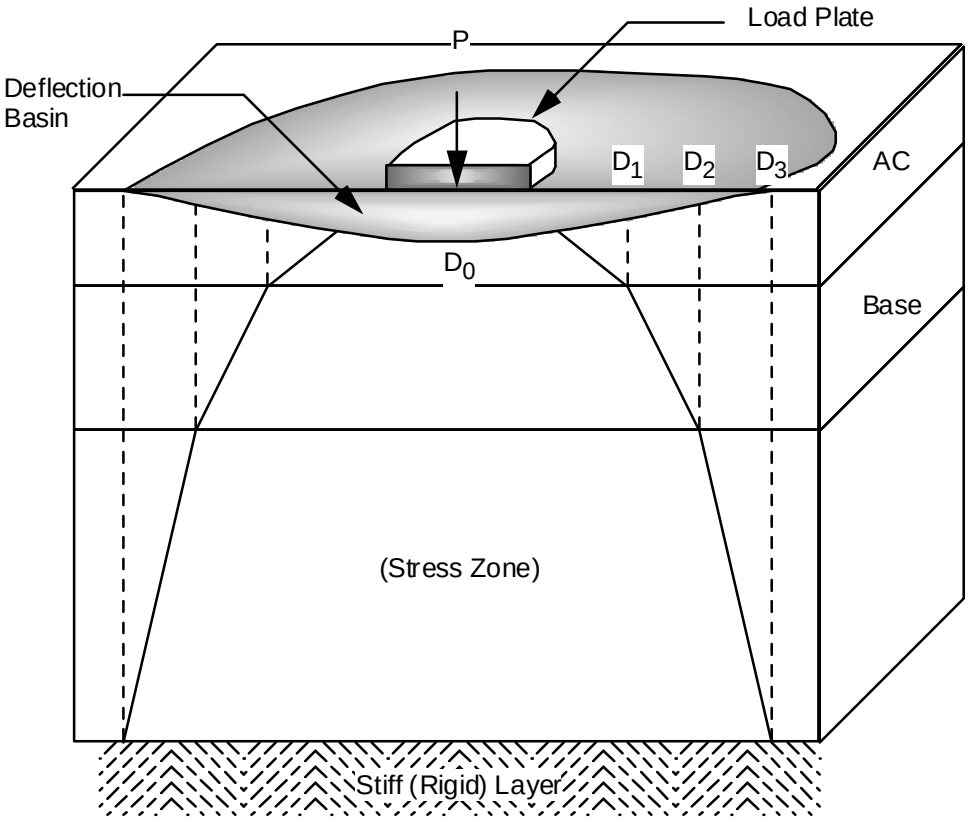

Figure 7.22. Illustration of Backcalculation to Estimate Layer Moduli

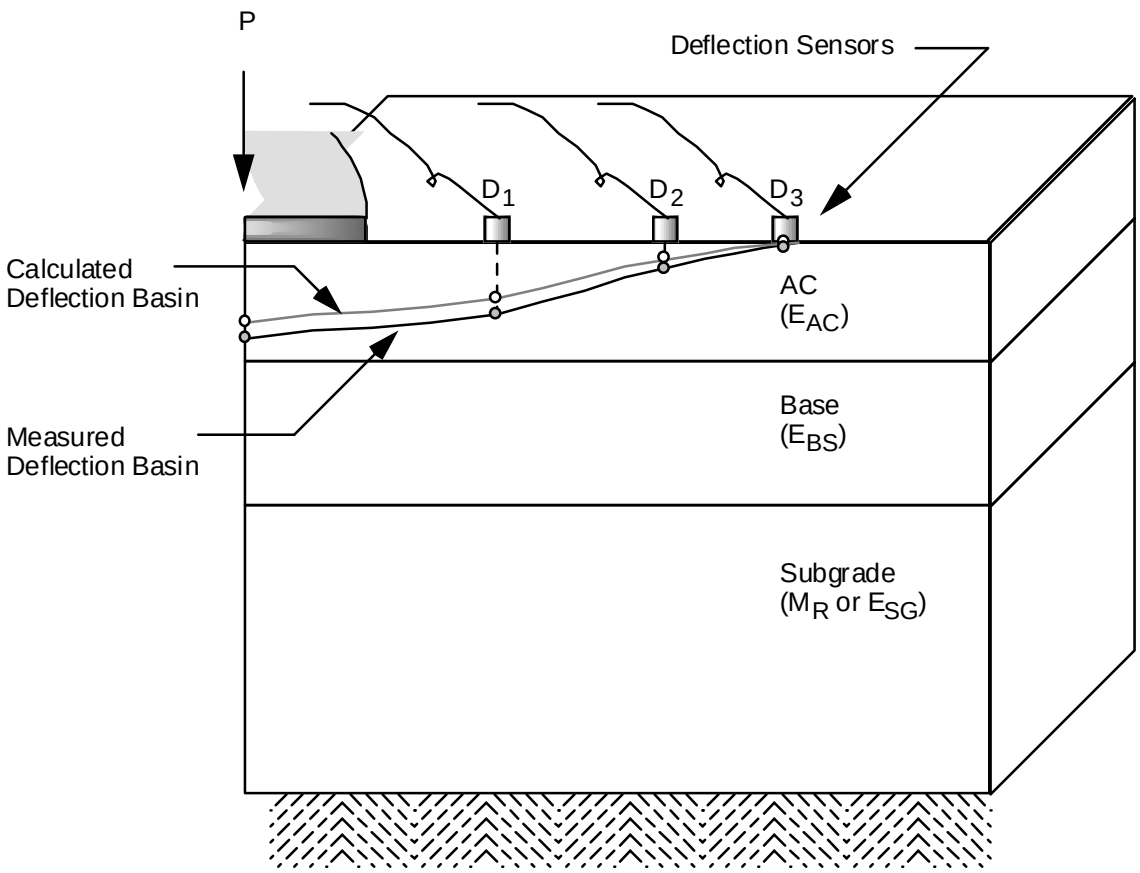

Figure 7.23. The Backcalculation Process - Matching Measured and Calculated Deflection Basins

3.3.2 TYPICAL FLOWCHART

A basic flowchart which represents the fundamental elements in all known backcalculation programs is shown as Figure 7.24. This flowchart was patterned after one shown by Lytton [7.24]. Briefly, these elements include:

(a) Measured deflections

Includes the measured pavement surface deflections and associated distances from the load.

(b) Layer thicknesses and loads

Includes all layer thicknesses and load levels for a specific test location.

(c) Seed moduli

The seed moduli are the initial moduli used in the computer program to calculate surface deflections. These moduli are usually estimated from user experience or various equations (as illustrated in this SECTION).

(d) Deflection calculation

Layered elastic computer programs such as CHEVRON, BISAR, or ELSYM5 are generally used to calculate a deflection basin.

(e) Error check

This element simply compares the measured and calculated basins. There are various error measures which can be used to make such comparisons (more on this in a subsequent paragraph in this section).

(f) Search for new moduli

Various methods have been employed within the various backcalculation programs to converge on a set of layer moduli which produces an acceptable error between the measured and calculated deflection basins.

(g) Controls on the range of moduli

In some of the backcalculation programs, a range (minimum and maximum) of moduli are selected or calculated to prevent program convergence to unreasonable moduli levels (could be too high or low).

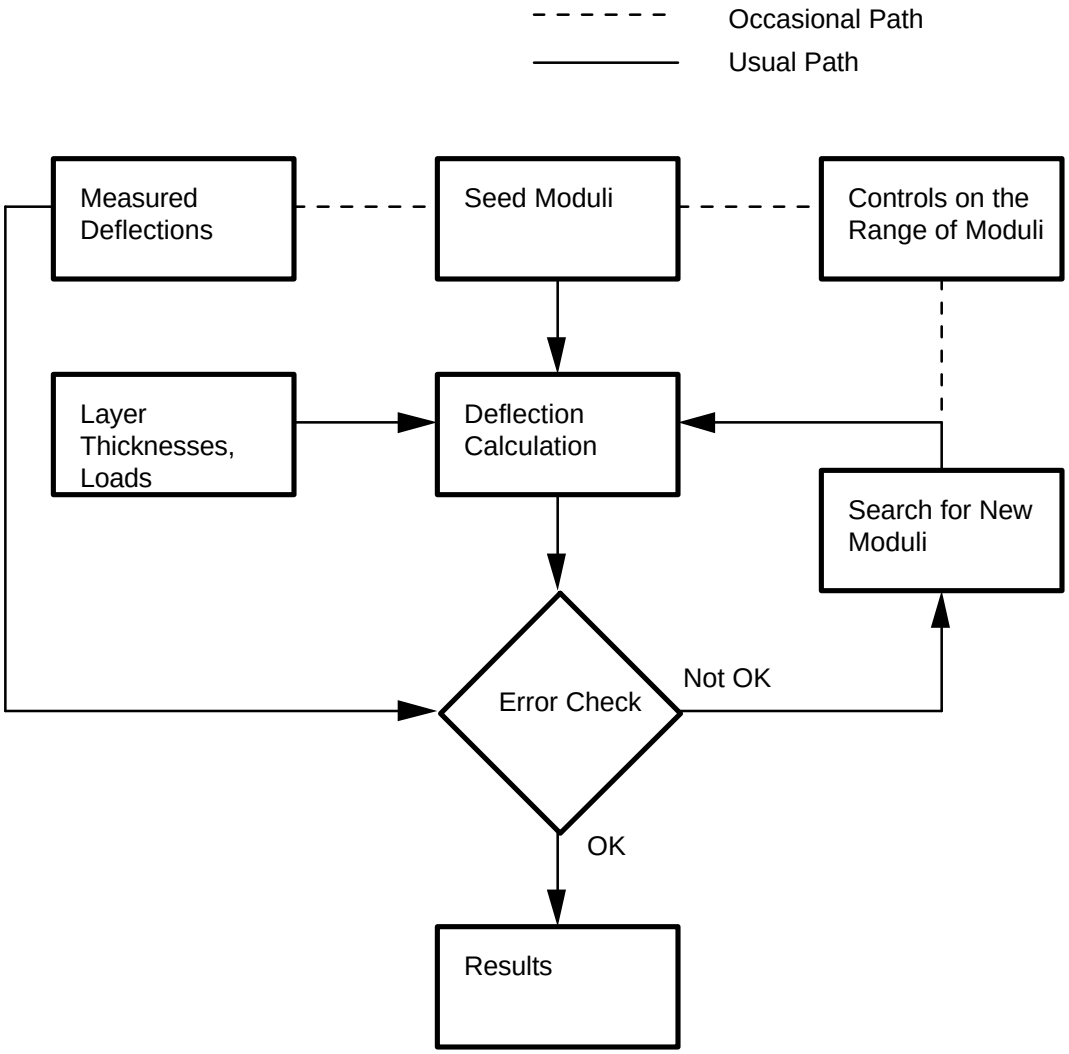

Figure 7.24. Common Elements of Backcalculation Programs [modified after Lytton (5.1)]

3.3.3 MEASURE OF CONVERGENCE

3.3.3.1 Measure of Deflection Basin Convergence

In backcalculating layer moduli, the measure of how well the calculated deflection basin matches (or converges to) the measured deflection basin was previously described as the "error check." This is also referred to as the "goodness of fit" or "convergence error."

The primary measure of convergence will be Root Mean Square (RMS) or Root Mean Square Error (RMSE) and is used in the EVERCALC program. Both terms are taken to be the same.

$$\text{RMS (\%)} = \left(\sqrt{\frac{1}{n_d}\sum_{i=1}^{n}\left(\frac{d_{ci}-d_{mi}}{d_{mi}}\right)^2}\right)(100)$$

where RMS = root mean square error,
d_{ci} = calculated pavement surface deflection at sensor i,
d_{mi} = measured pavement surface deflection at sensor i, and
n_d = number of deflection sensors used in the backcalculation process.

3.3.3.2 Example RMS calculation

n_d	Deflections (mils)	
	Measured	Calculated
1 (0")	5.07	4.90
2 (8")	4.32	3.94
3 (12")	3.67	3.50
4 (18")	2.99	3.06
5 (24")	2.40	2.62
6 (36")	1.69	1.86
7 (60")	1.01	0.95

$$\text{RMS (\%)} = \left[\frac{1}{7}\left(\frac{4.90 - 5.07}{1.01}\right)^2 + \ldots + \left(\frac{0.95 - 1.01}{1.01}\right)^2\right]$$

$$= 6.9\ \%\ \text{(very high)}$$

Generally an adequate range of RMS is 1 to 2 percent.

3.3.4 DEPTH OF STIFF LAYER

Recent literature provides at least two approaches for estimating the depth to stiff layer [Rohde and Scullion (7.25), Hossain and Zaniewski (7.26)]. The approach used by

Rohde and Scullion [7.25] will be summarized below. There are two reasons for this selection: (1) initial verification of the validity of the approach is documented, and (2) the approach is used in MODULUS 4.0 — a backcalculation program widely used in the U.S.

(a) Basic Assumptions and Description

A fundamental assumption is that the measured pavement surface deflection is a result of deformation of the various materials in the applied stress zone; therefore, the measured surface deflection at any distance from the load plate is the direct result of the deflection below a specific depth in the pavement structure (which is determined by the stress zone). This is to say that only that portion of the pavement structure which is stressed contributes to the measured surface deflections. Further, no surface deflection will occur beyond the offset (measured from the load plate) which corresponds to the intercept of the applied stress zone and the stiff layer (the stiff layer modulus being 100 times larger than the subgrade modulus). Thus, the method for estimating the depth to stiff layer assumes that the depth at which zero deflection occurs (presumably due to a stiff layer) is related to the offset at which a zero surface deflection occurs. This is illustrated in Figure 7.25 where the surface deflection D_c is zero.

An estimate of the depth at which zero deflection occurs can be obtained from a plot of measured surface deflections and the inverse of the corresponding offsets $\left(\frac{1}{r}\right)$. This is illustrated in Figure 7.26. The middle portion of the plot is linear with either end curved due to nonlinearities associated with the upper layers and the subgrade. The zero surface deflection is estimated by extending the linear portion of the D vs. $\frac{1}{r}$ plot to a D = 0, the $\frac{1}{r}$ intercept being designated as r_0. Due to various pavement section-specific factors, the depth to stiff layer cannot be directly estimated from r_0 — additional factors must be considered. To do this, regression equations were developed based on BISAR computer program generated data for the following factors and associated values:

Load = P = 9000 lbs (40 kN) (only load level considered)

Moduli ratios: $\frac{E_1}{E_{SG}} = 10, 30, 100$

$\frac{E_2}{E_{SG}} = 0.3, 1.0, 3.0, 10.0$

$\frac{E_{rigid}}{E_{SG}} = 100$

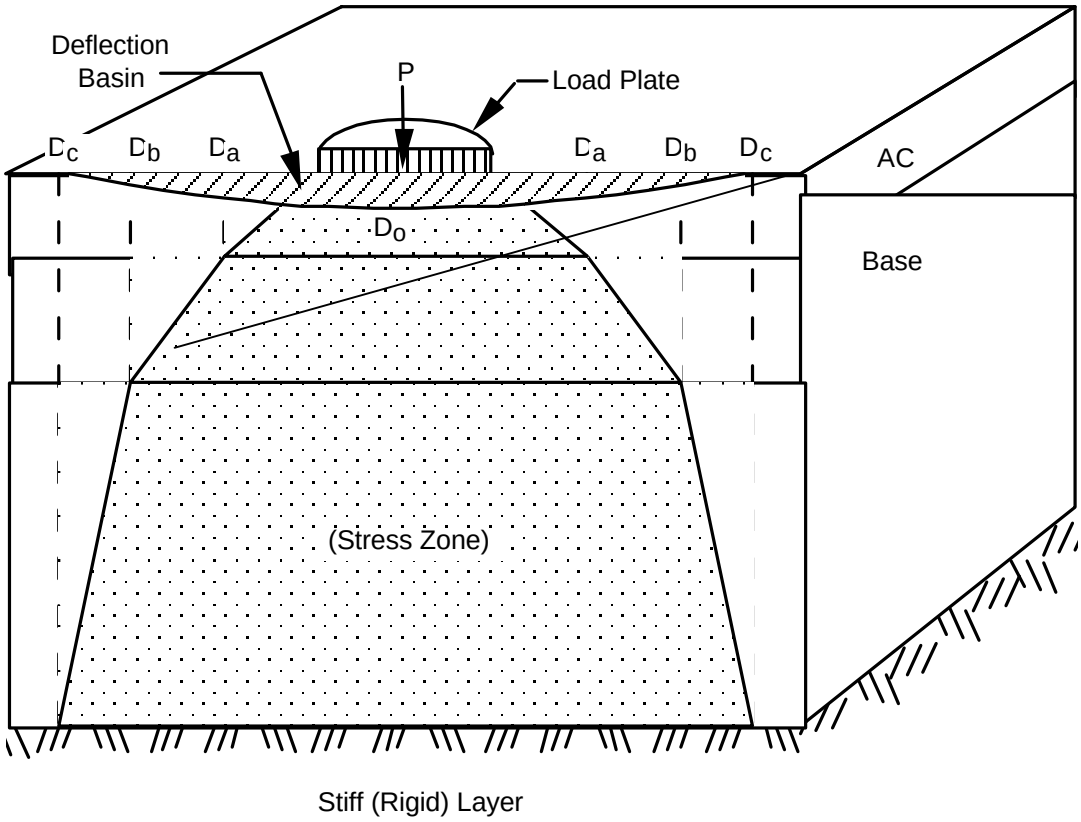

Figure 7.25. Illustration of Zero Deflection Due to a Stiff Layer

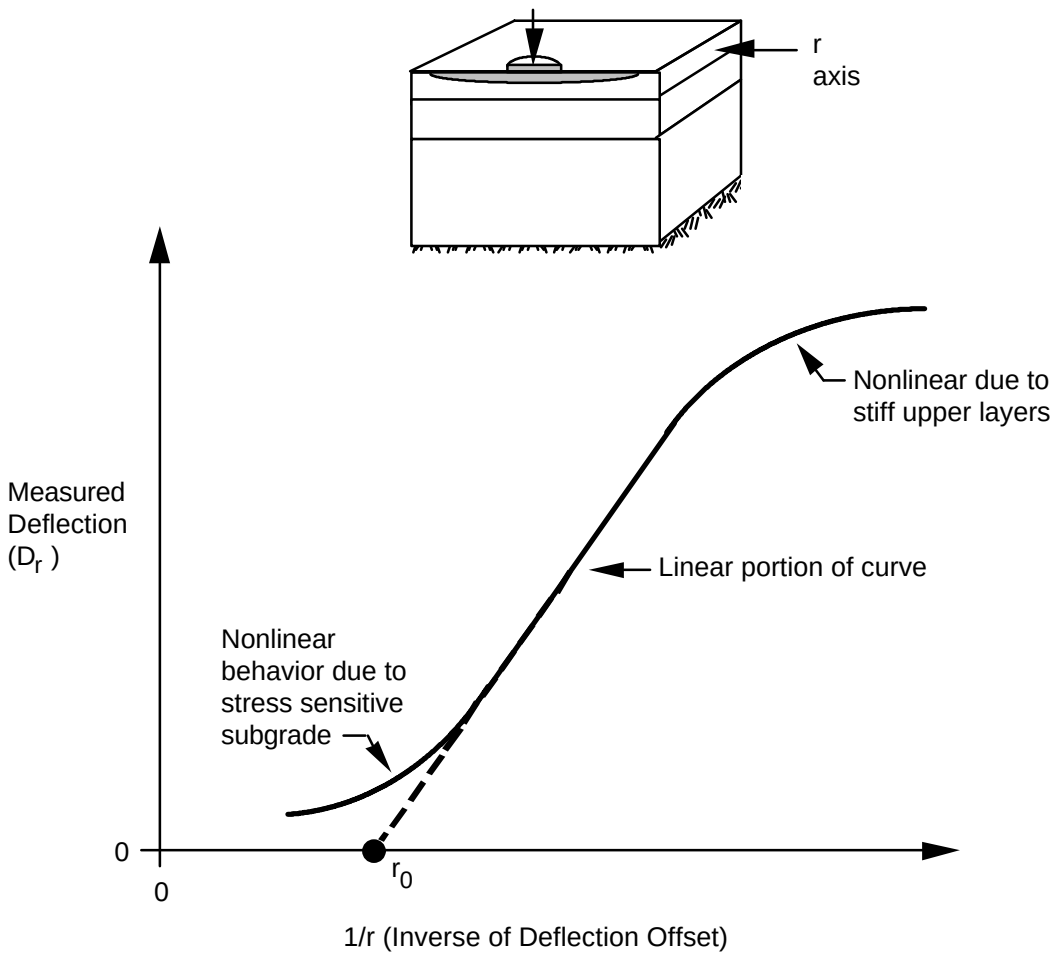

Figure 7.26. Plot of Inverse of Deflection Offset vs. Measured Deflection [17]

Thickness levels: T_1 = 1, 3, 5, and 10 in. (25, 75, 125, 250 mm)

T_2 = 6, 10, and 15 in. (150, 250, 375 mm)

B = 5, 10, 15, 20, 25, 30, and 50 ft (1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 15.0 m)

where E_i = elastic modulus of layer i,

T_i = thickness of layer i,

B = depth of the rigid (stiff) layer measured from the pavement surface (ft).

The resulting regression equations follow in (b).

(b) Regression Equations

Four separate equations were developed for various AC layer thicknesses. The dependent variable is $\frac{1}{B}$ and the independent variables are r_0 (and powers of r_0) and various deflection basin shape factors such as SCI, BCI, and BDI (discussed previously in subparagraph 3.1.1, SECTION 7.0).

(i) AC less than 2 in. (50 mm) thick

$$\frac{1}{B} = 0.0362 - 0.3242\,(r_0) + 10.2717\,(r_0^2) - 23.6609\,(r_0^3) - 0.0037\,(BCI)$$

$$R^2 = 0.98$$

(ii) AC 2 to 4 in. (50 to 100 mm) thick

$$\frac{1}{B} = 0.0065 + 0.1652\,(r_0) + 5.4290\,(r_0^2) - 11.0026\,(r_0^3) - 0.0004\,(BDI)$$

$$R^2 = 0.98$$

(iii) AC 4 to 6 in. (100 to 150 mm) thick

$$\frac{1}{B} = 0.0413 + 0.9929\,(r_0) - 0.0012\,(SCI) + 0.0063\,(BDI) - 0.0778\,(BCI)$$

$$R^2 = 0.94$$

(iv) AC greater than 6 in. (150 mm) thick

$$\frac{1}{B} = 0.0409 + 0.5669\,(r_0) + 3.0137\,(r_0^2) + 0.0033\,(BDI) - 0.0665 \log (BCI)$$

$$R^2 = 0.97$$

where r_0 = $\frac{1}{r}$ intercept (extrapolation of the steepest section of the D vs. $\frac{1}{r}$ plot) in units of $\frac{1}{ft}$,

SCI = $D_0 - D_{12''}$ ($D_0 - D_{305\ mm}$), Surface Curvature Index,

BDI = $D_{12''} - D_{24''}$ ($D_{305} - D_{610\ mm}$), Base Damage Index,

BCI = $D_{24''} - D_{36''}$ ($D_{610} - D_{914\ mm}$) Base Curvature Index,

D_i = surface deflections (mils) normalized to a 9,000 lb (40 kN) load at an offset i in feet.

(c) Example

Use some typical deflection data to estimate B (depth to stiff layer). The drillers log suggests a stiff layer <u>might</u> be encountered at a depth of 198 in. (5.0 m) (deflection data originally from SHRP).

(i) First, calculate normalized deflections (9,000 lb (40 kN) basis).

'

	Load Level	Deflections (mils)						
		D_0	D_8	D_{12}	D_{18}	D_{24}	D_{36}	D_{60}
	9,512 lb	5.07	4.32	3.67	2.99	2.40	1.69	1.01
(normalized)	9,000 lb	4.76	4.04	3.44	2.80	2.26	1.59	0.95
	6,534 lb	3.28	2.69	2.33	1.88	1.56	1.09	0.68

(ii) Second, estimate r_0. Plot D_r vs. $\frac{1}{r}$ (refer to Figure 7.27) :

D_r (mils)	r	$\frac{1}{r}$ $\left(\frac{1}{ft}\right)$
4.76	0"	—
4.04	8"	1.50
3.44	12"	1.00
2.80	18"	0.67
2.26	24"	0.50
1.59	36"	0.33
0.95	60"	0.20

where all D_r normalized to 9,000 lb (40 kN)

(iii) Third, use regression equation in (b)(iv) (for AC = 7.65 in. (194 mm)) to calculate B:

$$\frac{1}{B} = 0.0409 + 0.5669\,(r_0) + 3.0137\,(r_0^2) + 0.0033\,(BDI) - 0.0665 \log (BCI)$$

where $r_0 = \frac{1}{r}$ intercept (refer to Figure 7.27)

$\cong$ 0 (used steepest part of deflection basin which is for Sensors at 36 and 60 inches)

$$BDI = D_{12''} - D_{24''} = 3.44 - 2.26 = 1.18 \text{ mils}$$

$$BCI = D_{24''} - D_{36''} = 2.26 - 1.59 = 0.67 \text{ mils}$$

$$\frac{1}{B} = 0.0409 + 0.5669\,(0) + 3.0137\,(0^2) + 0.0033\,(1.18) - 0.0665 \log (0.67)$$

$$= 0.0564$$

$$\frac{1}{B} = \frac{1}{0.0564} = 17.7 \text{ feet (213 inches or 5.4 m)}$$

This value agrees fairly well with "expected" stiff layer conditions at 16.5 feet (5.0 m) as indicated by the drillers log even though the $\frac{1}{r}$ value equaled zero.

3.3.5 EXAMPLE OF DEPTH TO STIFF LAYER ESTIMATES — ROAD Z-675 (SWEDEN)

(a) Overview

This road located in south central Sweden is used to illustrate calculated and measured depths to stiff layers (the stiff layer apparently being rock for the specific road).

(b) Measurement of Measured Depth

The depth to stiff layer was measured using borings (steel drill) and a mechanical hammer. The hammer was used to drive the drill to "refusal." Thus, the measured depths could be to bedrock, a large stone, or hard till (glacially deposited material). Further, the measured depths were obtained independently of the FWD deflection data (time difference of several years).

(c) Deflection Measurements

A KUAB 50 FWD (refer to SECTION 3.0, Paragraph 5.2.3.3) was used to obtain the deflection basins. All basins were obtained within ± 5 m (16 ft) of a specific borehole. The deflection sensor locations were set at 0, 200, 300, 450, 600, 900, and 1200 mm (0, 7.9, 11.8, 17.7, 23.6, 35.4, and 47.2 in.) from the center of the load plate.

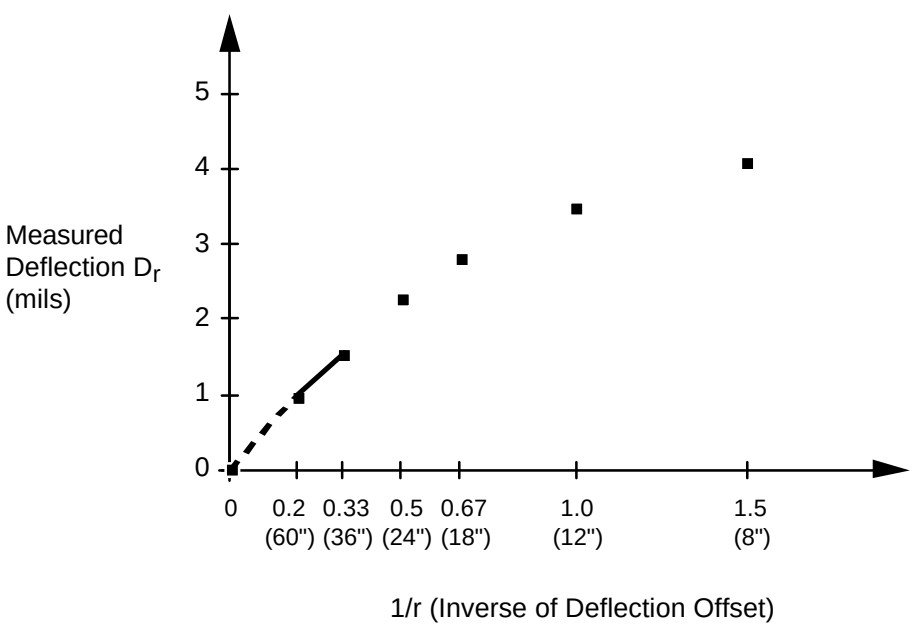

Figure 7.27. Plot of Inverse of Deflection Offset vs. Measured Deflection

(d) Calculations

The equations described in Section 3.3.4 were used to calculate the depth to stiff layer. Since the process requires a 9000 lb (40 kN) load and 1 ft (305 mm) deflection sensor spacings, the measured deflections were adjusted linearly according to the ratio of the actual load to a 9000 lb (40 kN) load.

(e) Results

The results of this comparison are shown in Figure 7.28. Given all the uncertainties concerning the measured depths, the agreement is quite good.

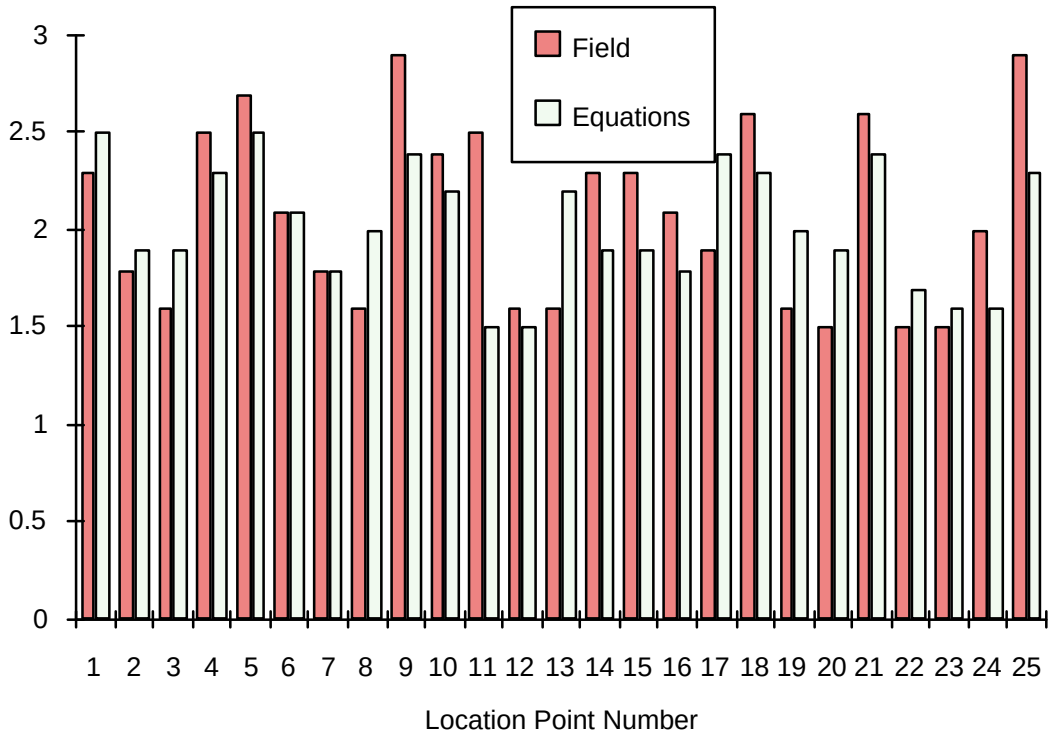

Figure 7.28. Plot of Measured and Calculated Depths to Stiff Layer (Road Z-675, Sweden)

3.3.6 BACKCALCULATION PROGRAMS

As of November 1990, 16 different backcalculation computer programs had been identified by a SHRP contractor (reported in Ref. 7.19). The primary program used by WSDOT is EVERCALC. This program has been revised to keep up with knowledge of how to best estimate layer moduli. The detailed documentation of this software will be provided in the WSDOT Pavement Guide, Volume 2.

3.3.7 VERIFICATION OF BACKCALCULATION RESULTS

There have been and, undoubtedly, will be a number of attempts to verify that backcalculated layer moduli are "reasonable." One way to do this is to compare measured, in-situ strains to calculated (theoretical) strains based on layered elastic analyses.

Two recent studies have done this. The first study summarized (Winters [7.28]) was conducted at the PACCAR Technical Center located at Mt. Vernon, Washington. Horizontal strains were measured on asphalt concrete instrumented cores (inserted into the original asphalt concrete surface material). The AC layer was approximately 5.5 in. thick over a 13.0 in. thick base course. The WSDOT FWD was used to apply a known load to induce the strain response (measured strain). The calculated strains were determined by use of the EVERCALC program (which used FWD deflection basins to backcalculate layer moduli which, in turn, were used to calculate the corresponding strains.)

Figures 7.29 and 7.30 show the measured and calculated strains in the two horizontal directions at the bottom of the AC layer (longitudinal and transverse). This specific testing was conducted during February 1993. In general, the agreement is good.

The second study was reported by Lenngren [7.29]. That work was conducted for RST Sweden and used backcalculated layer moduli (from a modified version of the EVERCALC program) to estimate strains at the bottom of two thicknesses of AC (3.1 and 5.9 in. thick). The pavement structures were located in Finland. Tables 7.12 and 7.13 are used as summaries. The backcalculated strains are generally within ± 5 percent of the measured strains.

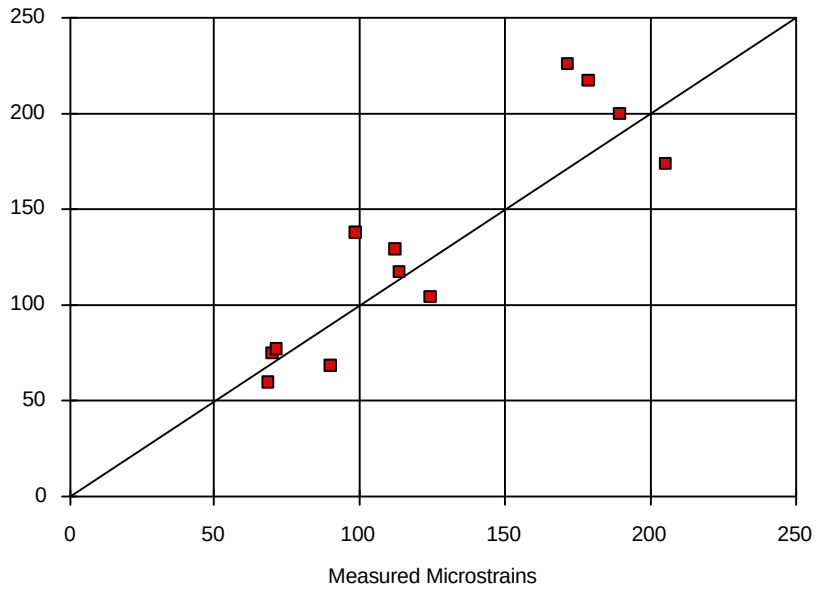

Figure 7.29. Measured vs. Calculated Strain for Axial Core Bottom Longitudinal Gauges – February 1993 FWD Testing (from Winters [7.29])

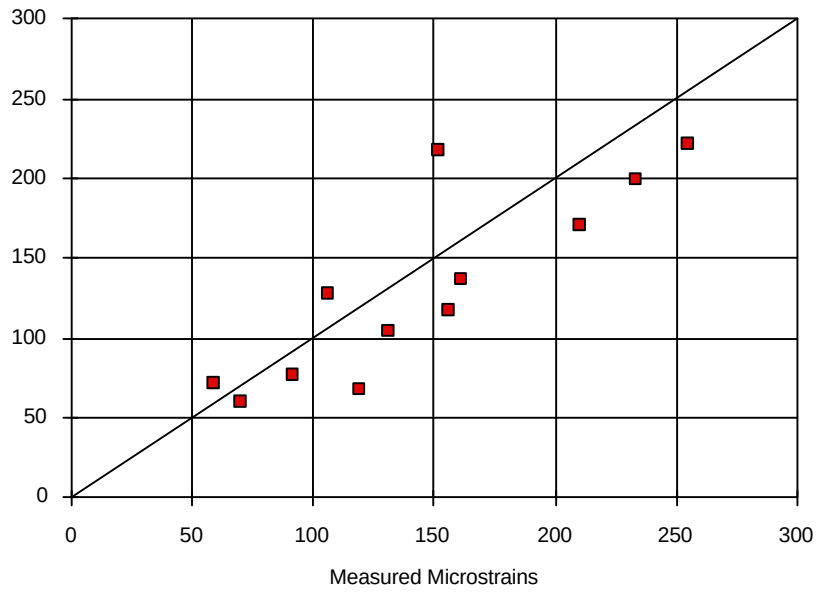

Figure 7.30. Measured vs. Calculated Strain for Axial Core Bottom Transverse Gauges – February 1993 FWD Testing (from Winters [7.29])

Table 7.12 Backcalculated and Measured Tensile Strains — 3.1 in. (80 mm) AC Section [after Lenngren (7.29)]

Time of Day (a.m. or p.m.)	Tensile Strain Bottom of AC (x 10^{-6})		
	Backcalculated *	Measured	% Difference
a.m.	119	123	-3
a.m.	119	122	-2
a.m.	74	64.9	+14
a.m.	60	64.7	-8
p.m.	284	292	-3
p.m.	284	283	~0
p.m.	167	159	+5
p.m.	167	158	+6
p.m.	87	84.8	+2
p.m.	81	84.2	-4

*Backcalculation process used sensors @ D_0, D_{300}, D_{600}, D_{900}, and D_{1200}

Table 7.13 Backcalculated and Measured Tensile Strains — 5.9 in. (150 mm) AC Section [after Lenngren (7.29)]

Time of Day (a.m. or p.m.)	Tensile Strain Bottom of AC (x 10^{-6})		
	Backcalculated *	Measured	% Difference
a.m.	66	69.5	-6
a.m.	71	69.0	+3
a.m.	68	68.7	-1
a.m.	38	34.7	+9
a.m.	127	130	-2
a.m.	119	130	-8
p.m.	178	185	-4
p.m.	182	183	-1
p.m.	104	95.9	+8
p.m.	51	48.0	+6
p.m.	56	48.5	+14

*Backcalculation process used sensors @ D_0, D_{300}, D_{600}, D_{900}, and D_{1200}

SECTION 7.0
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SECTION 7.0, APPENDIX 7.1

SEASONAL TEMPERATURES FOR WASHINGTON STATE

1. INTRODUCTION

The EVERPAVE program requires seasonal mean air temperatures (winter, spring, summer, fall). A summary of NOAA temperature data was prepared at the WSDOT Materials Laboratory and is included in this appendix.

2. MEAN SEASONAL TEMPERATURES

The NOAA temperature data is summarized in Table 1 (monthly, yearly means) and Table 2 (seasonal means) by NOAA designated "divisions." The means are based on monthly means measured from 1957 through 1989.

3. EXCEPTIONS

The data contained in Tables 1 and 2 are very general. Specific project locations may have unique temperature conditions due to local microclimate effects, etc. Local temperature effects should be used if known.

4. NOAA DIVISIONS

"Typical" weather stations used in developing Tables 1 and 2 include:

NOAA Division 1:
Western Olympic Peninsula — Coastal Area

- Neah Bay
- Forks
- Clearwater
- Quinault Ranger Station
- Cushman Dam
- Aberdeen
- Montesano
- Westport
- Raymond
- Long Beach

NOAA Division 2:
Northeast Olympia Peninsula — San Juan Islands

- Port Angeles
- Sequim
- Port Townsend
- Anacortes
- Mount Vernon

NOAA Division 3:
Puget Sound Lowlands

- Blaine
- Bellingham
- Sedro Wooley
- Burlington
- Monroe
- Seattle
- Sea-Tac Airport
- Kent
- Tacoma
- Puyallup
- Olympia
- Centralia
- Bremerton

NOAA Division 4:
Eastern Olympic-Western Cascade Foothills

- Quilcene
- Shelton
- Elma
- Toledo
- Longview
- Vancouver
- Mud Mountain Dam
- Snoqualmie Falls
- Startup
- Arlington
- Concrete

NOAA Division 5:
Western Cascade Mountains

- Ross Dam
- Marblemount Ranger Station
- Stevens Pass
- Snoqualmie Pass
- Stampede Pass
- Paradise
- Cougar

NOAA Division 6:
East Slope of Cascades

- Mazama
- Winthrop
- Stehekin
- Lake Wenatchee
- Plain
- Leavenworth
- Peshastin
- Easton
- Cle Elum
- Naches
- Mount Adams Ranger Station
- Glenwood
- Status Pass

NOAA Division 7: Okanogan-Big Bend

- Tonasket
- Conconully
- Omak
- Methow
- Nespelem
- Chief Joseph Dam
- Grand Coulee Dam
- Chelan
- Mansfield
- Waterville
- Hartline
- Wilbur
- Davenport
- Odessa

NOAA Division 8: Central Basin

- Wenatchee
- Ephrata
- Quincy
- Moses Lake
- Ritzville
- Othello
- Ellensburg
- Yakima
- Wapato
- Sunnyside
- Dallesport
- Goldendale
- Prosser
- Richland/Kennewick/Pasco
- Connell
- Walla Walla

NOAA Division 9:
Northeastern

- Wauconda
- Republic
- Northport
- Colville
- Chewelah
- Newport
- Spokane

NOAA Division 10:
Palouse-Blue Mountain

- Rosalia
- Tekoa
- St. John
- Colfax
- Pullman
- Pomeroy
- Dayton
- Asotin

Table 1. Monthly and Yearly Mean Temperatures (Years 1957-1989)

Region (NOAA Division)	Monthly Means (°F)												Yearly Mean (°F)
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
1 West Olympic-Coastal	39.6	42.5	44.2	47.4	52.5	56.7	59.9	60.5	58.0	51.5	44.7	40.8	49.9
2 Northeast Olympic-San Juans	39.2	42.2	44.6	48.3	53.6	57.9	61.0	61.3	57.7	50.8	44.1	40.6	50.1
3 Puget Sound Lowlands	38.6	42.4	45.0	49.2	55.2	60.2	63.9	64.0	59.4	51.8	44.2	39.9	51.1
4 East Olympic-West Cascades Foothills	—	40.7	43.6	47.8	54.0	59.2	63.3	63.4	58.7	50.8	42.7	38.2	49.9
5 Western Cascade Mountains	29.5	33.2	36.3	41.0	48.2	54.5	60.0	60.2	54.7	46.4	36.4	31.4	44.3
6 East Slope of Cascades	25.6	31.4	37.3	44.2	52.3	59.4	65.0	64.6	56.5	46.2	34.7	27.4	45.4
7 Okanogan-Big Bend	24.7	31.8	39.8	47.7	56.2	63.6	69.6	68.9	59.9	48.1	35.6	27.3	47.8
8 Central Basin	29.9	36.9	44.0	50.6	58.8	66.3	72.2	71.2	62.6	51.5	39.6	31.7	51.3
9 Northeastern	24.1	30.6	37.5	45.3	53.8	60.8	66.7	65.9	56.9	45.6	33.6	26.3	45.6
10 Palouse-Blue Mountains	29.8	36.3	41.5	47.6	55.2	62.4	68.8	68.3	60.0	49.7	38.7	31.9	49.2

Table 2. Seasonal Mean Temperatures (Years 1957-1989)

Region (NOAA Division)	Seasonal Mean Temperature (°F)			
	Spring • March • April • May	Summer • June • July • August	Fall • September • October • November	Winter • December • January • February
1 West Olympic-Coastal	48.0	59.0	51.4	41.0
2 Northeast Olympic-San Juans	48.8	60.1	50.9	40.7
3 Puget Sound Lowlands	49.8	62.7	51.8	40.3
4 East Olympic-West Cascades Foothills	48.5	62.0	50.7	38.6
5 Western Cascade Mountains	41.8	58.2	45.8	31.4
6 East Slope of Cascades	44.6	63.0	45.8	28.1
7 Okanogan-Big Bend	47.9	67.4	47.9	27.9
8 Central Basin	51.1	69.9	51.2	32.8
9 Northeastern	45.5	64.5	45.4	27.0
10 Palouse-Blue Mountains	48.1	66.5	49.4	32.7

SECTION 8.0

LIFE CYCLE COST ANALYSES

This SECTION overviews life cycle costing of pavements, the required inputs, and WSDOT project-specific examples. U.S. customary units are primarily used in this SECTION.

1. INTRODUCTION

The discussion of life cycle cost analyses will be mostly in terms of PCCP rehabilitation. This is due to the greater complexity associated with decision-making on these kinds of projects and illustrate the use of user costs. The principles involved apply to any type of pavement design activity.

2. ECONOMIC EVALUATION

2.1 INTRODUCTION

All PCCP rehabilitation or reconstruction projects, by the very nature of their complexity and higher costs, require a detailed economic evaluation. The Federal-Aid Highway Program Manual [8.1] states in Volume 6, Chapter 2, Section 4, Subsection 1, Paragraph 4(d):

> "Each SHA shall have a pavement rehabilitation selection process that is acceptable to FHWA and that includes identification of candidate solutions and methodology for structural design. For pavements approaching terminal serviceability and exhibiting significant structural deficiencies, the process shall include procedures for making an engineering and economic analysis of alternative rehabilitation strategies. These alternative rehabilitation strategies should include both reconstruction and rehabilitation alternatives."

Further, for all major rehabilitation projects, one of the necessary steps is Project Analysis as described in Paragraph 4(d)(2)(a):

> "Perform an engineering and economic analysis on candidate strategies. ...The economic analysis should consider service life, initial cost, maintenance costs, and future rehabilitation requirements, including maintenance of traffic costs."

Further, in the following subparagraph 4(d)(2)(b):

> "Select the best rehabilitation alternative. Although the economic analysis results are important in selecting the preferred alternatives, budget constraints and engineering judgment should also be considered in selecting the best alternative for a particular project."

The economic evaluation process most appropriate for the purpose of comparing various rehabilitation alternatives is that of life cycle cost (LCC) analysis. A LCC analysis is one that considers the cost of construction, rehabilitation, and maintenance of a facility over an analysis period that usually encompasses the service life of all alternatives.

The *AASHTO Guide for Design of Pavement Structures* [8.8] in Part I, Chapter 3, Paragraph 3.2, states:

> "It is essential in economic evaluation that all costs occurring during the life of the facility be included. When making economic comparisons this has not always been carefully practiced or even understood by pavement designers because comparisons were often made over a fixed period. Thus, designers assumed that first-cost comparisons were adequate for economic studies. This is not true, and, in order to emphasize the need for a complete cost analysis, the term "life-cycle costs" was coined about 1970 for use with pavements."

With the above insight, when considering the LCC analysis of various pavement rehabilitation or reconstruction options, the analysis compares the present worth of all costs associated with each alternative over a period of 30 or more years. The costs normally associated with pavement reconstruction are:

- Initial Construction Costs,
- Maintenance Costs,
- Rehabilitation Costs,
- Salvage Value, and
- User Delay Costs.

These costs are usually summarized over time by discounting all costs that occur at different times using the present worth method to account for the time value of money. The comparative costs may be shown as either total present worth or an annualized cost.

LCC analysis, the availability of funds, project specific and environmental conditions or constraints, project constructability and the ability of each alternative to serve the anticipated traffic volumes should all be used in the decision process for selecting the most appropriate alternative.

2.2 TYPES OF COSTS

2.2.1 INITIAL CONSTRUCTION COSTS

The more common structural resurfacing or reconstruction rehabilitation alternatives are usually: remove and replace with new PCCP, remove and replace with new ACP, or overlay with either ACP or PCCP. All details regarding the design of the alternatives shall be coordinated with the Region Materials Engineer and the Pavement Design Engineer. Typically new PCCP construction will consist of 10 to 12 in. (250 to 300 mm) of PCCP over asphalt treated base (ATB) or asphalt permeable base (APB). This alterative will provide the longest service life and will set the analysis period of about 30 to 40 years. Typically the PCCP surface will require grinding and joint resealing at 20 years. New ACP will usually consist of 8 to 10 in. (200 to 250 mm) of ACP over CSTC or ATB, depending on design needs. This option will typically need resurfacing every 10 to 15 years depending upon the traffic level and environment. The overlay options will follow about the same patterns.

The construction costs should reflect all unique costs associated with each alternative. For instance, each alternative has different roadway sections and material quantities, this should be accounted for in the analysis. Items that may be common to the rehabilitation, such as bridge and embankment widening, guard rail replacement, etc., need not be included in the analysis, due to repetition. Each overlay option will also require some grade adjustment of adjacent ramps, guard rails, barriers, etc. These added costs, which are unique to each alternative, should be included in the analysis.

2.2.2 MAINTENANCE COSTS

Maintenance costs are those costs associated with maintaining the pavement surface at some acceptable level. Since WSDOT programs the rehabilitation of its pavements quite early in the pavement deterioration cycle, there are no consistent maintenance cost differences associated with the various rehabilitation alternatives. When maintenance costs are available for the alternatives considered, they should be incorporated into the life cycle cost analysis.

2.2.3 REHABILITATION COSTS

These costs represent the costs associated with each rehabilitation alterative, such as grinding and joint resealing of the PCCP at 20 years, or overlaying the ACP at 10 to 15 years. They are computed in a manner consistent with the initial construction costs.

2.2.4 SALVAGE VALUE

The salvage value is the relative value of the various alternatives at the end of the analysis period. If an alternative has reached its full life cycle at the end of the analysis period, it is generally considered to have no remaining salvage value. If it has not completed a life cycle, it is given a salvage value which is usually determined by

multiplying the last construction or resurfacing cost by the ratio of the remaining expected life cycle to the total expected life.

2.2.5 USER DELAY COSTS

User delay costs are those costs that are accrued by the user of the facility during the construction, maintenance and/or rehabilitation of a roadway section. These costs are in the form of delay due to speed changes, speed reductions, and idling time.

3. LCC — THE DETAILS

3.1 INTRODUCTION

This document will discuss the LCC analysis for the rehabilitation of PCC pavements, which follows the basic procedure used for new construction, but contains more details and options.

Life cycle costs refer to all costs that are involved with the construction, maintenance, rehabilitation and associated user impacts of a pavement over a given analysis period. Life cycle cost analysis is an economic comparison of all feasible construction or rehabilitation alternatives, evaluated over the same analysis period.

> "A feasible alternative is one that fits with the required constraints (e.g., geometric, construction time, traffic flow conditions, clearances, right-of-way, maximum funds available, etc.)" [8.2].

3.2 PRESENT WORTH METHOD

The present worth method is an economic method that involves the conversion of all of the present and future expenses to a base of today's costs [8.3]. The totals of the present worth costs are then compared one with another. The general form of the present worth equation is as follows:

$$PW = F\left[\frac{1}{(1+i)^n}\right]$$

where PW = Present worth,
F = The future sum of money at the end of n years,
n = Number of years, and
i = Discount rate

The factor $\frac{1}{(1+i)^n}$ is also known as the Single Payment Present Worth Factor (SPW).

Example

Initial Construction Cost	=	$4,000,000
Future Rehabilitation Costs (15th and 30th year)	=	$500,000
Salvage Value	=	$200,000
Analysis Period	=	40 years
Discount Rate	=	4 percent

$$PW = \$4{,}000{,}000 + \frac{\$500{,}000}{(1.04)^{15}} + \frac{\$500{,}000}{(1.04)^{30}} - \frac{\$200{,}000}{(1.04)^{40}}$$

$$PW = \$4{,}000{,}000 + \frac{\$500{,}000}{(1.801)} + \frac{\$500{,}000}{(3.243)} - \frac{\$200{,}000}{(4.801)}$$

$$PW = \$4{,}000{,}000 + \$27$$

7,624 + $154,178 - $41,658 = $4,390,144

3.3 ANNUALIZED METHOD

The annualized method is an economic procedure that requires converting all of the present and future expenditures to a uniform annual cost [8.3]. This method reduces each alternative to a common base of a uniform annual cost. The costs are equated into uniform annual costs through the use of an appropriate discount rate [8.4]. Recurring costs, such as annual maintenance, are already expressed as annual costs. A given future expenditure, such as a pavement overlay, must first be converted to its present worth before calculating its annualized cost. The general form of the Annualized cost equation is as follows:

$$A = PW\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right]$$

where A = Annual cost
PW = Present worth
n = Number of years
i = Discount rate

The factor $\frac{i(1+i)^n}{(1+i)^n - 1}$ is also known as the Uniform Capital Recover Factor (UCR).

Example

Using the PW value that was determined in the previous example, calculate the annual costs:

$$A = \$4{,}390{,}144 \times \frac{0.04(1.04)^{40}}{(1.04)^{40} - 1}$$

$$A = \$4{,}390{,}144 \times 0.0505 = \$221{,}805$$

A 1988 survey of SHAs [8.10] revealed that 25 use Present Worth and 10 use Equivalent Uniform Annual LCC calculation methods. Some of the SHAs use both.

3.4 DISCOUNT RATE

"In a life cycle cost analysis, a discount rate is needed to compare costs occurring at different points in time. The discount rate reduces the impact of future costs on the analysis, reflecting the fact that money has a time value." [8.5] The discount rate is defined as the difference between the market interest rate and inflation, using constant dollars.

"The discount rate can affect the outcome of a life cycle cost analysis in that certain alternatives may be favored by higher or lower discount rates. High discount rates favor alternatives that stretch out costs over a period of time, since the future costs are discounted in relation to the initial cost. A low discount rate favors *high initial cost alternatives* since future costs are added in at almost face value. In the case of a discount rate equal to 0, all costs are treated equally regardless of when they occur. Where alternative strategies have similar maintenance, rehabilitation and operating costs, the discount rate will have a minor effect on the analysis and initial costs will have a larger effect" [8.5].

What is the appropriate discount rate to use in the economic analysis of pavements? The following points are offered for use in adopting a particular value [8.5]:

(a) "The difference between interest rates and inflation rates does not remain constant over time. Therefore, it is not possible to identify a unique discount rate which will always be correct. It is clear that the selection of an appropriate rate should not be based on unusual economic conditions which may occur for a relatively short period of time."

(b) "Since we cannot accurately forecast discount rates for long periods of time, a conservative approach would be to adopt a value somewhere between the high and the low range. A reasonable value might be in the range of 3 to 5 percent. It is perhaps on this basis that a discount rate of 4 percent is commonly used in pavement life cycle cost analyses. Such a range recognizes that discount rates of 7 or 8 percent have been relatively rare in this country and have lasted for only a short period of time. Additionally, we have had high discount rates for almost a

decade. It is probably unrealistic to assume that they will return in the short run to a range of 1 to 2 percent."

(c) "Once a discount rate has been selected, Agencies may wish to conduct a sensitivity analysis by calculating Present Worths or Equivalent Uniform Annualized Costs using several discount rates. If one alternative is favored over a range of discount rates, the Agency can have confidence that the analysis has truly identified the least cost alternative. It is important, however, to emphasize that the sensitivity analysis should not be used for changing discount rates on a project by project basis." However, they can help in evaluating the alternatives under consideration.

A survey of SHAs reported in 1988 [8.10] a range of discount rates. These varied from 0 percent (Montana, Ohio, South Dakota) to a high of 9 percent (Texas). The overall average discount rate for the 24 reporting SHAs was about 4.1 percent (standard deviation = 1.9 percent).

3.5 ANALYSIS PERIOD

The analysis period is the time period used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated [8.6].

> "In the past, pavements were typically designed and analyzed for a 20 year performance period, since the original Interstate Highway Act in 1956 required that traffic be considered through 1976. It is now recommended that consideration be given to longer analysis periods, since these may be better suited for the evaluation of alternative long-term strategies based on life cycle costs. Consideration should be given to extending the analysis period to include one rehabilitation."

Following are general guidelines noted in the 1993 *AASHTO Guide for the Design of Pavement Structures*, Paragraph 2.1.1, p. II-7 [8.8]:

Highway Conditions	Analysis Period (years)
High Volume Urban	30 to 50
High Volume Rural	20 to 50
Low Volume Paved	15 to 25
Low Volume Aggregate Surface	10 to 20

A 40 year analysis period is recommended (refer to Volume 1, Paragraphs 2.1 and 2.3.1).

3.6 COST FACTORS

Cost factors are those costs associated with the LCC analysis for a facility or in comparing alternative designs that cover the full cycle from initial design through the end of the analysis period. The possible costs associated with a pavement are [8.6]:

3.6.1 DESIGN COSTS

The expected costs for designing a new or rehabilitated pavement including materials, site investigations, traffic analysis, pavement design, and plans and specifications. Design costs would only need to be included if the cost of designing one alternative is different from those of another [8.6].

Design costs are estimated to be a percentage of the total initial construction costs.

3.6.2 CONSTRUCTION COSTS

The costs for building a section of pavement in accordance with the plans and specifications. These costs can be estimated from previous bids, previous projects, historical cost data, etc. The construction costs used in the analysis should be the most current and accurate data available [8.6]. The current version of the WSDOT Unit Bid Analysis is an excellent source of cost information.

Construction costs are the summation of all items involved in the construction of the project. For example, the items involved with preparation, grading, drainage, surfacing, AC and PCC pavement, traffic etc.

3.6.3 MAINTENANCE COSTS

Those costs associated with maintaining a pavement at or above some predetermined performance level. This includes both corrective and preventive maintenance but does not include rehabilitation. Maintenance costs are one of the most difficult areas to deal with in a LCC analysis. There are certain inherent problems in obtaining accurate and reliable maintenance costs. Maintenance costs are generally not gathered with the precision required for a LCC analysis [8.6].

WSDOT attempts to program the rehabilitation of its pavements quite early in the pavement deterioration cycle, hence, there are no consistent maintenance cost differences associated with the various rehabilitation alternatives identified in the WSDOT maintenance data base. When maintenance costs are available for the alternatives considered, they should be incorporated into the LCC analysis.

3.6.4 REHABILITATION COSTS

These costs cover the types of activities performed as part of rehabilitating or restoring the pavement. These represent periodic costs at future dates used to restore the pavement to an acceptable performance level. One major problem with future rehabilitation is the inability to accurately predict at what time in the future rehabilitation might be required. Good historical pavement performance data is helpful in estimating the best possible timing for rehabilitation [8.6]. The WSDOT Pavement Management System can be of significant assistance in developing representative pavement performance estimates.

Some of the common rehabilitation alternatives which can be considered for WSDOT Interstate type pavements are listed below along with typical performance lives.

(a) Remove and Replace with New PCCP

Typically, new PCCP construction will consist of 10 to 12 in. (250 to 300 mm) of PCCP over asphalt concrete base (ACB) or asphalt treated permeable base (ATPB). When dowel bars are to be used in transverse joints, dowel bars should be placed on chairs, and the chairs firmly secured to the layer below. Due to the difficulty in assuring that the joints will be sawed at the appropriate dowel locations, skewed joints should not be used. The existing WSDOT random joint spacing (9, 10, 14, 13 ft. (2.7, 3.0, 4.3, 4.0 m)) should be continued when dowels are not used. The "cyclic hop" that is created with a uniform 15 ft. (4.6 m) joint spacing is potentially damaging to trucks. The current random spacing of joints helps to minimize this "cyclic hop."

(b) <u>Remove and Replace with New ACP</u>

Typically, new ACP (Class A and ACB) will be similar to PCC slab thicknesses for similar traffic levels. The ACP is placed on CSBC and/or ATPB. This alternative should have an initial service life of about 15 years due to aging of the asphalt in the surface course. It will be followed by AC overlays and service lives of 10 to 15 years depending upon the rate of aging, wearing, rutting, fatigue cracking, etc. of the surface course. The actual service lives will depend upon the traffic levels, environment, and construction quality.

(c) ACP Overlay

Typically, an ACP overlay of PCCP will require 0.35 to 0.50 ft. (107 to 152 mm) of ACP. This alternative, depending upon traffic level and environment, will need resurfacing every 10 to 15 years. An ACP overlay of a thick section of ACP generally will require a 0.15 ft. (46 mm) overlay or recycle 0.15 ft. (46 mm) with a Class D ACP, at 10 to 15 year cycles. Very heavy traffic or serious environmental conditions, such as mountain passes or long grades, will cause lower service lives, such as 10 to 12 years. More benign service conditions will provide for a longer service life.

(d) Other Rehabilitation/Repair Considerations

With the development of new materials and/or new techniques, other rehabilitation alternatives should be studied, and with repeatable success, incorporated into the selection process. These may include, but not limited to: grinding, retrofitting dowel bars, PCCP unbonded and bonded overlays, tied PCC shoulders, etc.

When developing "feasible" rehabilitation alternatives it is important to ensure "that the existing deterioration is repaired and then prevented from recurring, if at all possible" [8.2, 8.7]. All details regarding the design of the alternatives shall be coordinated with the Region Materials Engineer and the Pavement Design Engineer.

Rehabilitation costs are based on current material costs. Note that future costs (projects such as AC overlays) will be paid for with dollars that will be worth less (due to inflation) and such future projects will cost more (also due to inflation) [after Blank and Tarquin (8.9)]. To some extent, these are counter balancing effects and reinforce the view that current costs can be used for future costs. If cost increases are anticipated due to other causes (such as a diminishing aggregate supply in a specific area or region), then these increases should be included in the future costs.

3.6.5 USER COSTS

These costs are those accumulated by the user of the facility as related to pavement type, condition, maintenance activities, or rehabilitation work. These costs include vehicle operating costs (tires, gas, oil, etc.) and user delay costs (costs associated with slow downs due to construction and maintenance activities and denial-of-use) [8.6].

It is difficult to determine whether or not one rehabilitation alternative results in a higher vehicle operating cost than another. Therefore, the user costs associated with each of the rehabilitation alternatives will be determined using only costs associated with user delay. This will be based on the construction periods and the traffic volumes that are affected by each of the rehabilitation alternatives.

Several studies have been performed that associate a cost with the amount of time the user is delayed through a construction project. The method used is not as important as using the same method for each of the different alternatives.

A 1988 SHA survey [8.10] revealed at that time that only nine states included some element of user costs in their LCC analyses. Eight of the nine SHAs used time delay in their estimated user costs (the primary consideration).

The California Department of Transportation (CALTRANS) in a 1986 study determined that the average value of time is equal to $6.25 per vehicle-hour of delay. Vehicle delay is determined by the amount of time delayed due to speed changes within the construction zone. This method is used in the case study enclosed.

$$UC = (\$6.25/\text{veh-hr})\left(\frac{L}{RS} - \frac{L}{IS}\right)(ADT)(PT)(CP)$$

where UC = User cost

L = Project length

RS = Reduced speed through construction zone

IS = Initial speed prior to construction zone

ADT = Average daily traffic in current year (only that portion of the ADT affected by the project)

PT = Percent of the traffic that will be affected due to construction project. A traffic study will need to be performed, such that the percentage of traffic that uses the facility during the construction period can be determined.

CP = Construction period

Example

L = 5 miles

RS = 20 mph

IS = 55 mph

ADT = 20,000 vehicles per day

PT = 75 percent

CP = 20 days

UC = ($6.25)(5/20 - 5/55)(20,000)(0.75)(20) = $298,295
(note: units cancel to total dollars)

3.6.6 SALVAGE VALUE

This is the value of the pavement at the end of the analysis period. The salvage value can be either positive or negative depending on whether the material has some economic value or the cost of demolition or removal exceeds any value [8.6]. If an alternative has reached its full life cycle at the end of the analysis period, it is generally considered to have no remaining salvage value. If it has not completed a life cycle, it is given a salvage value which is usually determined by multiplying the last construction or resurfacing cost by the ratio of the remaining expected life cycle to the total expected life.

$$\text{Salvage Value} = (CC)\left(\frac{ERL}{TEL}\right)$$

where CC = Last construction or rehabilitation cost

ERL = Expected remaining life

TEL = Total expected life

Example

One of the rehabilitation alternatives for an existing PCC pavement is an AC overlay. The performance life of the AC overlay is expected to be 15 years. If the analysis period is selected as 40 years, then a total of three AC overlays will need to be performed over the entire analysis period. The first overlay will be in the current year, the second in year 15, and the third in year 30. Since the third overlay has an expected performance life of 15 years, this implies that the overlay will go beyond the analysis period by 5 years. Assuming a rehabilitation cost of $1,500,000:

CC = $1,500,000
ERL = 5 years
TEL = 15 years
Salvage Value = $1,500,000 x 5/15 = $500,000

3.7 COST CALCULATIONS

Once the rehabilitation alternatives have been selected, the next step in the LCC process is to determine project limits, lane and shoulder widths, construction durations, and a listing of required construction bid items and their corresponding costs that are unique to each design alternative. These costs will also include that of traffic control.

3.8 STANDARD FORM

The following case study has been developed using the SMART Spreadsheet computer program, and is available to all WSDOT offices (contact WSDOT Materials Laboratory for details).

3.9 LCC — A CASE STUDY

These case studies are detailed so that realistic examples are shown. The types of pavement layers used (such as ATB) may not conform with current WSDOT recommendations.

3.9.1 SUMMARY OF PCCP REHABILITATION COSTS

This location is for I-90 from the Spokane city limit (west) to the Spokane Viaduct, MP 275.50 to 280.16 (eastbound lanes). Five alternatives were considered and are presented.

Alternate Number	Present Worth			Annualized Costs		
	Construction Cost	User Costs		Excluding User Costs	Including User Costs	
		Day	Night		Day	Night
1	5,553,000	1,696,000	1,372,000	281,000	366,000	350,000
2a	4,406,000	1,465,000	1,324,000	223,000	297,000	289,000
2b	5,179,000	1,561,000	1,344,000	262,000	341,000	330,000
3a	1,721,000	430,000	90,000	87,000	109,000	91,000
3b	2,554,000	570,000	119,000	129,000	158,000	135,000

Analysis Period = 40 years
Discount Rate = 4 percent

Alternate 1 — Remove and replace with new 11" PCCP, and 0.35 ft. ATB.

Alternate 2A — Remove and replace with new 14" ACP, 0.20 ft. overlay in 15th and 30th year.

Alternate 2B — Remove and replace with new 14" ACP, 0.20 ft. overlay in 10th, 20th, and 30th year.

Alternate 3A — 0.40 ft. overlay, shoulder to shoulder; mill 0.15 ft. then overlay 0.15 ft., shoulder to shoulder, in 15th year; mill 0.30 ft., then overlay 0.30 ft., shoulder to shoulder in 30th year.

Alternate 3B — 0.40 ft. overlay, shoulder to shoulder; mill 0.15 ft. then overlay 0.15 ft., shoulder to shoulder, in 10th year; mill 0.30 ft., then overlay 0.30 ft., shoulder to shoulder in 20th and 30th year.

3.9.2 BASIC DATA FOR CONSTRUCTION COST CALCULATIONS

Begin	End	Miles	Feet	Lane Width	Shldr Width
275.50	275.55	0.05	264	24	20
275.55	279.49	3.94	20,803	24	14
279.49	280.16	0.67	3,538	36	12

Total number of miles	4.66
Total number of lane-miles	9.99
Total number of shoulder lane-miles	5.35

	sq. ft.	sq. yd.
Lane Area	632,966	70,330
Shoulder Area	338,976	37,664
Total Area	971,942	107,994

3.9.3 BASIC DATA FOR USER COST CALCULATIONS

Determination of Future Traffic Levels

34 percent ADT increase in 10 years
56 percent ADT increase in 15 years
81 percent ADT increase in 20 years
143 percent ADT increase in 30 years

1990 ADT = 16,370 (only the eastbound portion of the ADT)

ADT(2000) = (16,370)(1 + 0.34) = 21,936
ADT(2005) = (16,370)(1 + 0.56) = 25,537
ADT(2010) = (16,370)(1 + 0.81) = 29,630
ADT(2020) = (16,370)(1 + 1.43) = 39,779

3.9.4 PROJECT ENGINEERING AND CONSTRUCTION COSTS IN 1990 DOLLARS — ALTERNATE 1

(a) Summary — Alternate 1

Description: Remove all existing pavement. Place 0.35 ft. ATB and pave full width with 11 inches PCCP (transverse joints with dowel bars).

Profile grind, clean and reseal all transverse and longitudinal joints in 20th year.

Description: 11" PCCP DWL. JTD — Profile Grind

Year	1990	2000	2010	2020	2030	Total
P.E.*	690,000		75,000			765,000
Construction	4,602,000		497,000			5,099,000
Subtotal	5,292,000		571,000			5,864,000
PW No User Cost	5,292,000		261,000			5,553,000
PW User Cost						
Day Grinding	1,287,000		409,000			1,696,000
Night Grinding	1,287,000		85,000			1,372,000
Total PW						
Day Grinding	6,579,000		670,000			7,249,000
Night Grinding	6,579,000		346,000			6,925,000

Annualized Costs: Excluding User Costs = $281,000
Including User Costs = $366,000 Day Work
Including User Costs = $350,000 Night Work

Note: All estimates include traffic control
Discount rate = 4 percent

*P.E. = 15% added for Project Engineer office activities (project construction)

(b) Details (assumes construction period of 90 days for removal and replacement with PCCP)

Reconstruction

(137 lb/sy = 137 lb/sy of 0.10 ft. compacted depth)

Remove PCCP	70,330 sy x $5.00	$351,648
ATB	107,994 sy x 3.5* x 137 lb/sy / 2000 lb/ton x $20.00	$517,829
PCCP	107,994 sy x $18.00/sy	$1,943,885
Subtotal		$2,813,362

*(i.e., a depth of 0.35 ft. of ATB)

Traffic

Labor	5 percent of construction cost	$140,668
Signs	Estimate $35,000 for Class A and B construction signs	$35,000
Temp. Barrier	30,880 lf x $11.00	$339,768
Reset Barrier	30,880 lf x 4 moves x $1.00	$123,552
Impact Atten	3 each x $12,000	$36,000
Reset Atten	4 moves x $2000	$8000
Subtotal		$682,988

Subtotal (Construction and Traffic)		$3,496,350
Mobilization	(6 percent)	$209,781
Tax	(7.5 percent)	$277,960
Engineering	(10 percent)	$398,409
Contingencies	(5 percent)	$219,125
TOTAL		$4,601,625

Future Profile and Seal (Construction period = 45 days)

Seal	9.99 ln-mi x $8000 + 5.35 shldr ln-mi x $8000	$122,720
Profile	107,994 sy x $2.00	$215,987
Total		$338,707

Traffic

Labor	5 percent of construction cost	$16,935
Arrow	1 each x 45 days x 8 hrs/day x $5.00	$1,800
Maint Traffic	Estimate $20,000	$20,000
Subtotal		$38,735

Subtotal (Construction and Traffic)		$377,443
Mobilization	(6 percent)	$22,647
Tax	(7.5 percent)	$30,007
Engineering	(10 percent)	$43,010
Contingencies	(5 percent)	$23,655
TOTAL		$496,761

User Cost Calculations

Initial Construction — 1990
$6.25 x (4.66/20 - 4.66/50) x 16,370 x 1.00 x 90 days $1,287,296

Profile and Grind — 2010
Day Grinding
$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.77 x 45 days $897,052
Night Grinding
$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.16 x 45 days $186,400

3.9.5 PROJECT ENGINEERING AND CONSTRUCTION COSTS IN 1990 DOLLARS — ALTERNATE 2A

(a) Summary — Alternate 2A

Description: Remove all existing pavement. Place 0.35 ft. ATB, 0.45 ft. ACB and 0.35 ft. ACP throughout the entire project limits, shoulder to shoulder.

Recycle 0.20 ft. and overlay, shoulder to shoulder, with 0.20 ft. ACP in 15th and 30th years.

Description:	New 14" ACP		Recycle 0.20 ft.	Recycle 0.20 ft.		
Year	1990	2000	2005	2020	2030	Total
P.E.*	451,000		156,000	156,000		763,000
Construction	3,005,000		1,040,000	1,040,000		5,085,000
Subtotal	3,456,000		1,196,000	1,196,000		5,848,000
PW No User Cost	3,456,000		664,000	369,000	-83,000	4,406,000
PW User Cost						
Day Paving	1,287,000		95,000	83,000		1,465,000
Night Grinding	1,287,000		20,000	17,000		1,324,000
Total PW						
Day Paving	4,743,000		759,000	452,000	-83,000	5,871,000
Night Paving	4,743,000		684,000	386,000	-83,000	5,730,000

Annualized Costs: Excluding User Costs = $223,000
Including User Costs = $297,000 Day Paving
Including User Costs = $289,000 Night Paving

Note: All estimates include traffic control
Discount rate = 4 percent

Costs in year 2030 are salvage values

*P.E. = 15% added for Project Engineer office activities (project construction)

(b) Details (see Alternate 2B, Paragraph 3.9.6)

3.9.6 PROJECT ENGINEERING AND CONSTRUCTION COSTS IN 1990 DOLLARS — ALTERNATE 2B

(a) Summary — Alternate 2B

Description: Remove all existing pavement. Place 0.35 ft. ATB, 0.45 ft. ACB and 0.35 ft. ACP throughout the entire project limits, shoulder to shoulder.

Recycle 0.20 ft. and overlay, shoulder to shoulder, with 0.20 ft. ACP in 10th, 20th, and 30th years.

Description:	New 14" ACP	Recycle 0.20 ft.	Recycle 0.20 ft.	Recycle 0.20 ft.		
Year	1990	2000	2010	2020	2030	Total
P.E.*	451,000	156,000	156,000	156,000		919,000
Construction	3,005,000	1,040,000	1,040,000	1,040,000		6,125,000
Subtotal	3,456,000	1,196,000	1,196,000	1,196,000		7,044,000
PW No User Cost	3,456,000	808,000	546,000	369,000		5,179,000
PW User Cost						
Day Paving	1,287,000	100,000	91,000	83,000		1,561,000
Night Paving	1,287,000	21,000	19,000	17,000		1,344,000
Total PW						
Day Paving	4,743,000	908,000	637,000	452,000		6,740,000
Night Paving	4,743,000	829,000	565,000	386,000		6,523,000

Annualized Costs: Excluding User Costs = $262,000
Including User Costs = $341,000 Day Paving
Including User Costs = $330,000 Night Paving

Note: All estimates include traffic control
Discount rate = 4 percent

*P.E. = 15% added for Project Engineer office activities (project construction)

(b) <u>Details</u> (assumes construction period of 90 days for removal and replacement with ACP)

<u>Reconstruction</u>

(0.05 = 0.05 gal/sy; 240 = 240 gal/ton)
(137 lb/sy = 137 lb/sy of 0.10 ft. compacted depth)

Remove PCCP	70,330 sy x $5.00	$351,648
ATB & ACB	(70,330 sy x (3.5 + 4.5) x 137 lb/sy/2000 lb/ton) x $20.00	$770,812
ACP	107,994 sy x 3.5 x 137 lb/sy/2000 lb/ton x $20.00	$517,829
Tack	0.05 x (70,330 sy x 3 + 107,994 sy x 2)/240 x $200	$17,791
Total		$1,658,080
<u>Traffic</u>		
Labor	5 percent of construction cost	$82,904
Signs	Estimate $35,000 for Class A and B construction signs	$35,000
Temp Barrier	30,880 lf x $11.00	$339,768
Reset Barrier	30,880 lf x 4 moves x $1.00	$123,552
Impact Atten	3 each x $12,000	$36,000
Reset Atten	4 moves x $2000	$8000
Total		$625,224
Subtotal (Construction and Traffic)		$2,283,304
Mobilization	(6 percent)	$136,998
Tax	(7.5 percent)	$181,523
Engineering	(10 percent)	$260,183
Contingencies	(5 percent)	$143,100
TOTAL		$3,005,108
<u>0.20' AC Overlay (Construction period = 10 days)</u>		
Milling	107,994 sy x $4.00	$431,974
ACP	107,994 sy x 2 x 137 lb/sy/2000 lb/ton x $20.00	$295,902
Tack	0.05 x 107,994 sy / 240 x $200	$4,500
Total		$732,377
<u>Traffic</u>		
Labor	4 persons x 10 days x 14 hrs/day x $22.00	$12,320
Arrow	1 each x 10 days x 8 hrs/day x $5.00	$400
Maint. Traffic	Estimate $20,000	$20,000
Signs	Estimate $25,000 for Class A and B construction signs	$25,000
Total		$57,720
Subtotal (Construction and Traffic)		$790,097
Mobilization	(6 percent)	$47,406
Tax	(7.5 percent)	$62,813
Engineering	(10 percent)	$90,032
Contingencies	(5 percent)	$49,517
TOTAL		$1,039,864

User Cost Calculations

Initial Construction — 1990

$6.25 x (4.66/20 - 4.66/50) x 16,370 x 1.00 x 90 days	$1,287,296

0.20 ft. Overlay — 2000 (Alternate 2B)

Day Paving	
$6.25 x (4.66/20 - 4.66/50) x 21,936 x 0.77 x 10 days	$147,581
Night Paving	
$6.25 x (4.66/20 - 4.66/50) x 21,936 x 0.16 x 10 days	$30,666

0.20 ft. Overlay — 2005 (Alternate 2A)

Day Paving	
$6.25 x (4.66/20 - 4.66/50) x 25,537 x 0.77 x 10 days	$171,811
Night Paving	
$6.25 x (4.66/20 - 4.66/50) x 25,537 x 0.16 x 10 days	$35,701

0.20 ft. Overlay — 2010 (Alternate 2B)

Day Paving	
$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.77 x 10 days	$199,345
Night Paving	
$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.16 x 10 days	$41,422

0.20 ft. Overlay — 2020 (Alternate 2A and 2B)

Day Paving	
$6.25 x (4.66/20 - 4.66/50) x 39,779 x 0.77 x 10 days	$267,629
Night Paving	
$6.25 x (4.66/20 - 4.66/50) x 39,779 x 0.16 x 10 days	$55,611

3.9.7 PROJECT ENGINEERING AND CONSTRUCTION COSTS IN 1990 DOLLARS — ALTERNATE 3A

(a) Summary — Alternate 3A

Description: Overlay, shoulder to shoulder, with 0.40 ft. ACP.

Mill and replace 0.15 ft. ACP in right lane, then overlay with 0.15 ft., shoulder to shoulder in 15th year.

Mill and replace 0.30 ft. ACP, all lanes, then overlay with 0.30 ft. ACP, shoulder to shoulder, in 30th year.

Description:	0.40 ft. Overlay		0.15 ft. Overlay	0.30 ft. Overlay		
Year	1990	2000	2005	2020	2030	Total
P.E.*	133,000		85,000	187,000		405,000
Construction	884,000		565,000	1,249,000		2,698,000
Subtotal	1,017,000		650,000	1,436,000		3,103,000
PW No User Cost	1,017,000		361,000	443,000	-100,000	1,721,000
PW User Cost						
Day Paving	220,000		86,000	124,000		430,000
Night Paving	46,000		18,000	26,000		90,000
Total PW						
Day Paving	1,237,000		447,000	567,000	-100,000	2,151,000
Night Paving	1,063,000		379,000	469,000	-100,000	1,811,000

Annualized Costs: Excluding User Costs = $87,000
Including User Costs = $109,000 Day Paving
Including User Costs = $91,000 Night Paving

Note: All estimates include traffic control
Discount rate = 4 percent
Costs in year 2030 are salvage values

*P.E. = 15% added for Project Engineer office activities (project construction)

(b) <u>Details</u> (see alternate 3B, Paragraph 3.9.8)

3.9.8 PROJECT ENGINEERING AND CONSTRUCTION COSTS IN 1990 DOLLARS — ALTERNATE 3B

(a) <u>Summary — Alternate 3B</u>

Description: Overlay, shoulder to shoulder, with 0.40 ft. ACP.

Mill and replace 0.15 ft. ACP in right lane, then overlay with 0.15 ft., shoulder to shoulder in 10th year.

Mill and replace 0.30 ft. ACP, all lanes, then overlay with 0.30 ft. ACP, shoulder to shoulder, in 20th and 30th years.

Description:	0.40 ft. Overlay	0.15 ft. Overlay	0.30 ft. Overlay	0.30 ft. Overlay		
Year	1990	2000	2010	2020	2030	Total
P.E.*	133,000	85,000	187,000	187,000		592,000
Construction	884,000	565,000	1,249,000	1,249,000		3,947,000
Subtotal	1,017,000	650,000	1,436,000	1,436,000		4,539,000
PW No User Cost	1,017,000	439,000	655,000	443,000		2,554,000
PW User Cost						
Day Paving	220,000	90,000	136,000	124,000		570,000
Night Grinding	46,000	19,000	28,000	26,000		119,000
Total PW						
Day Paving	1,237,000	529,000	791,000	567,000		3,124,000
Night Paving	1,063,000	458,000	683,000	469,000		2,673,000

Annualized Costs: Excluding User Costs = $129,000
Including User Costs = $158,000 Day Paving
Including User Costs = $135,000 Night Paving

Note: All estimates include traffic control
Discount rate = 4 percent

*P.E. = 15% added for Project Engineer office activities (project construction)

(b) <u>Details</u> (assumes construction period of 20 days for ACP Overlay)

<u>Original AC Overlay</u>

(0.05 = 0.05 gal/sy; 240 = 240 gal/ton)
(137 lb/sy = 137 lb/sy of 0.10 ft. compacted depth)

ACP	107,994 sy x 4 x 137 lb/sy / 2000 lb/ton x $20.00	$591,805
Tack	0.05 x 107,994 sy / 240 x 2 coats x $200	$8,999
Total		$600,804
<u>Traffic</u>		
Labor	4 persons x 20 days x 14 hrs/day x $22.00	$24,640
Arrow	1 each x 20 days x 14 hrs/day x $5.00	$1,400
Maint. Traffic	Estimate $20,000	$20,000
Signs	Estimate $25,000 for Class A and B construction signs	$25,000
Total		$71,040
Subtotal (Construction and Traffic)		$671,844
Mobilization	(6 percent)	$40,311
Tax	(7.5 percent)	$53,412

Engineering	(10 percent)	$76,557
Contingencies	(5 percent)	$42,106
TOTAL		$884,230

0.15' AC Overlay — Mill 0.15' RL, 0.15' @ year 15
(Construction period = 9 days)

Milling	7,040 sy x 4.66 miles x $4.00	$131,226
ACP	(107,994 + 7,040 sy) x 1.5 x 137 lb/sy / 2000 lb/ton x $20.00	$236,394
Tack	0.05 x (107,994 + 7,040 sy) / 240 x $200	$4,793
Total		$372,413

Traffic

Labor	4 persons x 9 days x 14 hrs/day x $22.00	$11,088
Arrow	1 each x 9 days x 14 hrs/day x $5.00	$630
Maint. Traffic	Estimate $20,000	$20,000
Signs	Estimate $25,000 for Class A and B construction signs	$25,000
Total		56,718

Subtotal (Construction and Traffic)		$429,131
Mobilization	(6 percent)	$25,748
Tax	(7.5 percent)	$34,116
Engineering	(10 percent)	$48,899
Contingencies	(5 percent)	$26,895
TOTAL		$564,789

0.30' AC Overlay — Mill 0.30', shoulder to shoulder
(Construction period = 15 days)

Milling	107,994 sy x $4.00	$431,974
ACP	107,994 sy x 3.0 x 137 lb/sy / 2000 lb/ton x $20.00	$443,854
Tack	0.05 x 107,994 sy / 240 x 2 coats x $200	$8,999
Total		$884,828

Traffic

Labor	4 persons x 15 days x 14 hrs/day x $22.00	$18,480
Arrow	1 each x 15 days x 14 hrs/day x $5.00	$1,050
Maint. Traffic	Estimate $20,000	$20,000
Signs	Estimate $25,000 for Class A and B construction signs	$25,000
Total		$64,530

Subtotal (Construction and Traffic)		$949,358
Mobilization	(6 percent)	$56,961
Tax	(7.5 percent)	$75,474
Engineering	(10 percent)	$108,179
Contingencies	(5 percent)	$59,499
TOTAL		$1,249,471

User Cost Calculations

Initial Construction — 1990

Day Paving

$6.25 x (4.66/20 - 4.66/50) x 16,370 x 0.77 x 20 days $220,271

Night Paving

$6.25 x (4.66/20 - 4.66/50) x 16,370 x 0.16 x 20 days $45,771

0.15 ft. Overlay — 2000 (Alternate 3B)

Day Paving

$6.25 x (4.66/20 - 4.66/50) x 21,936 x 0.77 x 9 days $132,823

Night Paving

$6.25 x (4.66/20 - 4.66/50) x 21,936 x 0.16 x 9 days $27,600

0.15 ft. Overlay — 2005 (Alternate 3A)

Day Paving

$6.25 x (4.66/20 - 4.66/50) x 25,537 x 0.77 x 9 days $154,630

Night Paving

$6.25 x (4.66/20 - 4.66/50) x 25,537 x 0.16 x 9 days $32,131

0.30 ft. Overlay — 2010 (Alternate 3B)

Day Paving

$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.77 x 15 days $299,017

Night Paving

$6.25 x (4.66/20 - 4.66/50) x 29,630 x 0.16 x 15 days $62,133

0.30 ft. Overlay — 2020 (Alternate 3A and 3B)

Day Paving

$6.25 x (4.66/20 - 4.66/50) x 39,779 x 0.77 x 15 days $401,443

Night Paving

$6.25 x (4.66/20 - 4.66/50) x 39,779 x 0.16 x 15 days $83,417

4. TYPICAL WSDOT PAVING COSTS

A few, selected project costs were obtained from the WSDOT "Summary of Costs and Resources Used" for the time period of June 1, 1992, through May 31, 1993, and bid tabulation sheets (1994). These costs are from actual project bids. For the purposes of this summary, only the low bid numbers were used. In the tabular information which follows, the following items are noted:

- WSDOT Standard Item Number,
- Region for which cost data obtained,
- Contract quantity (ton, CY, etc.),
- Cost per unit quantity, and
- Percent item represents of the total project cost.

Additionally, comments are made which refer to the cost per unit quantity listed. For example, for some items, the lowest and highest low bids for projects within a given region are noted, followed by a region average (total item cost for all contracts divided by the total item quantity for all contracts). Thus, the region average cost per unit quantity is heavily weighted by larger quantity projects.

The specific items summarized represent only a small portion of paving related items but include

Standard Item Number	Description
5602	• Cement Concrete Pavement, 14 day, 0.75' Section, (SY)
5614	• Cement Concrete Pavement, 14 day, 0.83' Section, (SY)
5711	• Planning Bituminous Pavement, (SY)
5764	• Asphalt Concrete Pavement, Class A, (ton)
5765	• Asphalt Concrete Pavement, Class B, (ton)
5775	• Asphalt Concrete Pavement, Class D, (ton)
5780	• Asphalt Concrete Pavement, Class E, (ton)
5790	• Asphalt Concrete Pavement, Class G, (ton)
5510	• Asphalt Treated Base, (ton)
5100	• Crushed Surfacing Base Course, (ton)
5120	• Crushed Surfacing Top Course, (ton)

One noticeable difference for bulk materials with large quantities of aggregate (specifically ACP—Class A and B, CSBC, and CSTC) are costs for westside and eastside regions. For example, westside ACP—Class A is about 37 percent more expensive ($29.48/ton vs. $21.45/ton), ACP—Class B is about 16 percent more expensive ($28.05/ton vs. $24.27/ton), CSBC is about 50 percent more ($10.74/ton vs. $7.16/ton), and CSTC 39 percent more ($11.64/ton vs. $8.37/ton). Naturally, there are a number of reasons for such differences such as haul distances, pit sources, traffic congestion, etc.

WSDOT Standard Item No.	Comments	Region	Contract Quantity	Cost/Unit	Percent of Project Cost
5602 (PCCP, 14 day, 9" thick slab)	• All contracts (total of 2)	Northwest	166 s.y. 4,698 s.y.	$37.75/s.y. $25.00/s.y.	0.3% 0.5%
5614 (PCCP, 14 day, 10" thick)	• One contract	South Central	118,460 s.y.	$12.50/s.y.	36.8%
5614 (PCCP, 14 day, 11" thick)	• One contract	Eastern	240,010 s.y.	$19.30/s.y.	30.0%
5711 (Planing Bit. Pavement) Note: Region — Projects Northwest: 29 North Central: 6 Olympic: 22 Southwest: 12 South Central: 3 Eastern: 5	• Lowest Project • Highest Project	Northwest Northwest	20,547 s.y. 234 s.y.	$1.25/s.y. $17.85/s.y	4.4% 1.8%
	• Lowest Project • Highest Project	North Central North Central	359,336 s.y. 440 s.y.	$0.75/s.y. $9.00/s.y.	7.0% 0.3%
	• Lowest Project • Highest Project	Olympic Olympic	36,130 s.y. 410 s.y.	$1.57/s.y. $18.00/s.y.	2.4% 2.3%
	• Lowest Project • Highest Project	Southwest Southwest	181,560 s.y. 356 s.y.	$0.50/s.y. $12.00/s.y.	7.3% 2.5%
	• Lowest Project • Highest Project	South Central South Central	81,950 s.y. 1,180 s.y.	$1.20/s.y. $5.00/s.y.	7.2% 3.1%
	• Lowest Project • Highest Project	Eastern Eastern	15,000 s.y. 2,110 s.y.	$1.50/s.y. $3.50/s.y.	0.8% 0.6%
5764 (ACP, Class A)	• Lowest Project • Highest Project • Region Average	Northwest Northwest Northwest	14,750 ton 40 ton —	$25.50/ton $148.75/ton $29.58/ton	7.1% 2.6% —
	• Lowest Project • Highest Project • Region Average	North Central North Central North Central	50,503 ton 3,060 ton —	$17.00/ton $44.00/ton $21.06/ton	3.7% 7.4% —
	• Lowest Project • Highest Project • Region Average	Olympic Olympic Olympic	35,730 ton 45 ton —	$21.50/ton $100.00/ton $28.70/ton	44.3% 1.4% —
	• Lowest Project • Highest Project • Region Average	Southwest Southwest Southwest	13,780 ton 130 ton —	$24.35/ton $185.00/s.y. $30.15/ton	24.0% 3.7% —
	• Lowest Project • Highest Project • Region Average	South Central South Central South Central	24,480 ton 315 ton —	$22.00/ton $67.95/ton $24.46/ton	20.7% 6.2% —
	• Lowest Project • Highest Project • Region Average	Eastern Eastern Eastern	31,100 ton 16,871 ton —	$17.50/ton $21.00/ton $18.83/ton	10.1% 19.3% —

WSDOT Standard Item No.	Comments	Region	Contract Quantity	Cost/Unit	Percent of Project Cost
5765 (ACP, Class B)	• Region Average	Northwest	—	$27.39/ton	—
	• Region Average	North Central	—	$22.94/ton	—
	• Region Average	Olympic	—	$28.71/ton	—
	• Region Average	Southwest		—	—
	• Region Average	South Central	—	$24.05/ton	—
	• Region Average	Eastern	—	$25.81/ton	—
5775 (ACP, Class D)	• Lowest Project • Highest Project • Region Average	South Central South Central South Central	4470 ton 29 ton —	$28.00/ton $170.00/ton $30.27/ton	5.3% 1.3% —
5780 (ACP, Class E)	• Region Average	Northwest	—	$27.91/ton	—
	• Region Average	Olympic	—	$24.69/ton	—
	• Region Average	Southwest	—	$28.94/ton	—
	• Region Average	Eastern	—	$23.99/ton	—
5790 (ACP, Class G)	• Lowest Project • Highest Project • Region Average	Northwest Northwest Northwest	5545 ton 23 ton —	$29.50/ton $300.00/ton $41.55/ton	27.9% 3.9% —
	• Lowest Project • Highest Project • Region Average	Olympic Olympic Olympic	2350 ton 100 ton	$24.50/ton $80.00/ton $31.54/ton	3.5% 0.8% —
5510 (ATB)	• Region Average	Northwest	—	$23.58/ton	—
	• Region Average	Olympic	—	$19.79/ton	—

WSDOT Standard Item No.	Comments	Region	Contract Quantity	Cost/Unit	Percent of Project Cost
5100 (CSBC)	• Lowest Project • Highest Project • Region Average	Northwest Northwest Northwest	1251 ton 4 ton —	$8.25/ton $35.70/ton $9.97/ton	1.7% 0.1% —
	• Region Average	North Central	—	$6.00/ton	—
	• Region Average	Olympic	—	$13.71/ton	—
	• Region Average	Southwest	—	$8.55/ton	—
	• Lowest Project • Highest Project • Region Average	South Central South Central South Central	10,930 ton 418 ton —	$5.45/ton $30.00/ton $7.19/ton	20.1% 1.0% —
	• Region Average	Eastern	—	$8.29/ton	—
5120 (CSTC)	• Region Average	Northwest	—	$11.53/ton	—
	• Region Average	North Central	—	$9.15/ton	—
	• Region Average	Olympic	—	$11.86/ton	—
	• Region Average	Southwest	—	$11.52/ton	—
	• Region Average	South Central	—	$8.42/ton	—
	• Region Average	Eastern	—	$7.55/ton	—

SECTION 8.0

REFERENCES

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8.5 Kleskovic, Peter Z., "A Discussion of Discount Rates for Economic Analysis of Pavements." Draft Report, FHWA Pavement Division, February 1990.

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8.8 American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures*, AASHTO, Washington, D.C., 1993.

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8.10 Federal Highway Administration, "Summary of the AASHTO Questionnaire on the *Guide for Design of Pavement Structures, 1986*," Office of Highway Operations, Federal Highway Administration, Washington, D.C., April 1, 1988.

SECTION 9.0

SUBSURFACE PAVEMENT DRAINAGE

This SECTION is used to describe the basic elements of subsurface pavement drainage. Specific reference is made to normally specified WSDOT materials.

1. NEED FOR SUBSURFACE DRAINAGE

A variety of reasons can necessitate the need for subsurface drainage such as

- high water table or groundwater flows (particularly in side hill cuts), or
- enhance overall pavement performance by eliminating (reducing) water related deterioration within the pavement structure or subgrade.

The emphasis in this section is placed on the latter reason for new or reconstructed pavement structures. For additional information on subgrade drains used to intercept high water table or groundwater flows refer to WSDOT Standard Plan B-10.

The FHWA is strongly supporting the use of subsurface drainage where needed. This support can be illustrated by a quote from former FHWA Administrator Tom Larson in a letter to the FHWA Regional and Federal Land Administrators, February 6, 1992:

> "The developments in technology for permeable bases and longitudinal edgedrains make the provision of positive drainage of the pavement section possible and affordable. Accordingly, to be acceptable to the Federal Highway Administration, each State's pavement design procedure must include a drainage analysis for each new and reconstructed pavement section. Where the drainage analysis or past performance indicates the potential for reduced service life due to saturated structural layers or pumping, the design must include positive measures to minimize that potential."

2. BASIC ELEMENTS

A drainable pavement structure will have three basic elements

- permeable base to provide for rapid removal of water which enters the pavement structure,
- a longitudinal collector system to convey accumulated water from the permeable base,
- a "filter" layer to prevent the migration of fines (generally minus No. 200) into the permeable base from the subgrade, subbase or shoulder base material. (A "filter" layer can be a graded aggregate layer(s), a geotextile or an underlying stabilized base (such as ATB)).

Each of these three basic elements will be discussed in the following paragraphs.

Three FHWA references will be used repeatedly throughout this SECTION. Reference 9.1 (draft FHWA Technical Advisory) was replaced by Reference 9.8 (technical guide paper). Reference 9.6 (FHWA training notebook) provided much useful information.

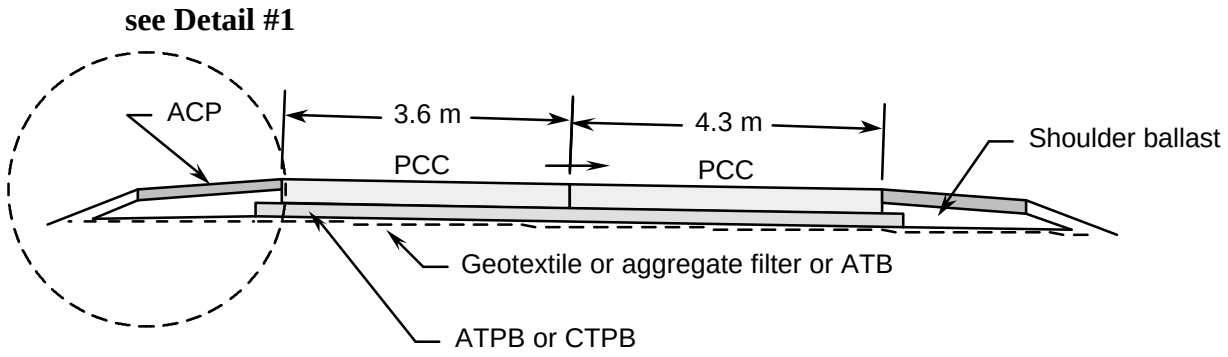
3. PERMEABLE BASE

3.1 GENERAL

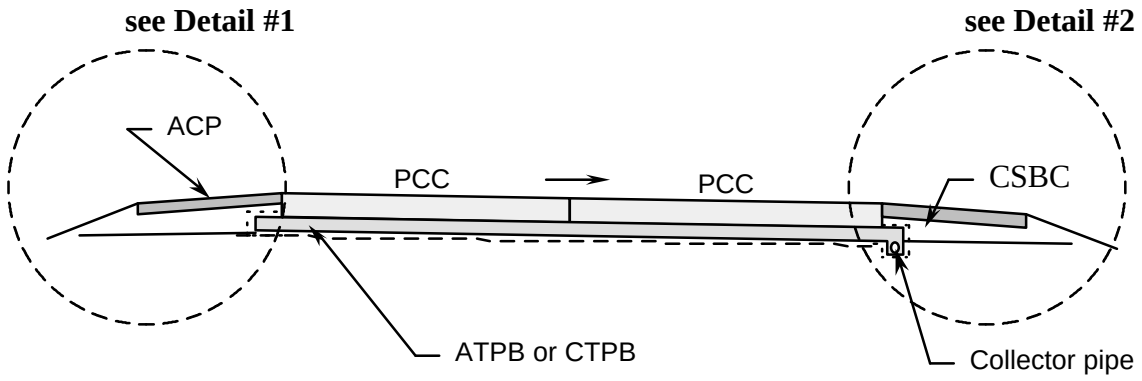
In general, only stabilized permeable bases will be used beneath PCCP as illustrated in Figures 9.1 and 9.2. The use of a permeable base is an option for flexible pavements as shown in Figure 9.3. Stabilized permeable bases can be stabilized with either asphalt cement or portland cement. A number of SHAs use the AASHTO No. 57 gradation [9.1] which is

Sieve Size (mm)	Percent Passing
37.5	100
25.0	95-100
12.5	25-60
4.75	0-10
2.36	0-5

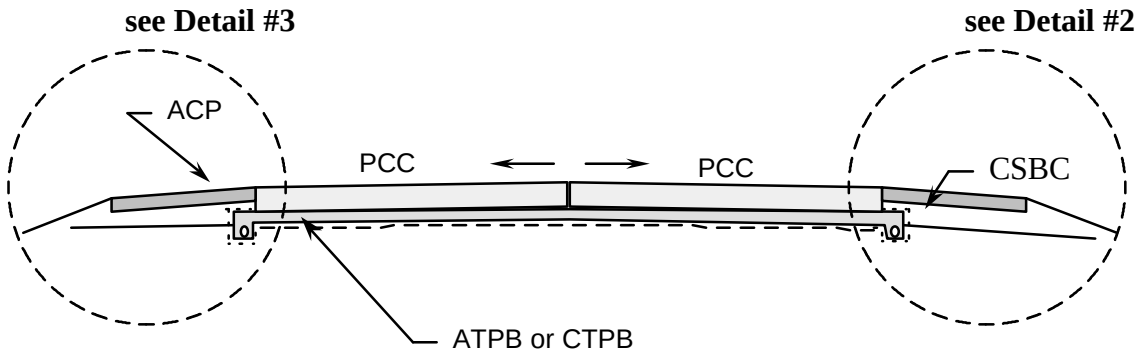
The stabilization material is generally 2 to 2.5 percent by weight of mix for asphalt cement (AR-8000 or higher) or about 3 to 4 bags of portland cement per cubic meter (2 to 3 bags per cubic yard).



(a) Permeable Base Drains to Daylighted Shoulder Ballast



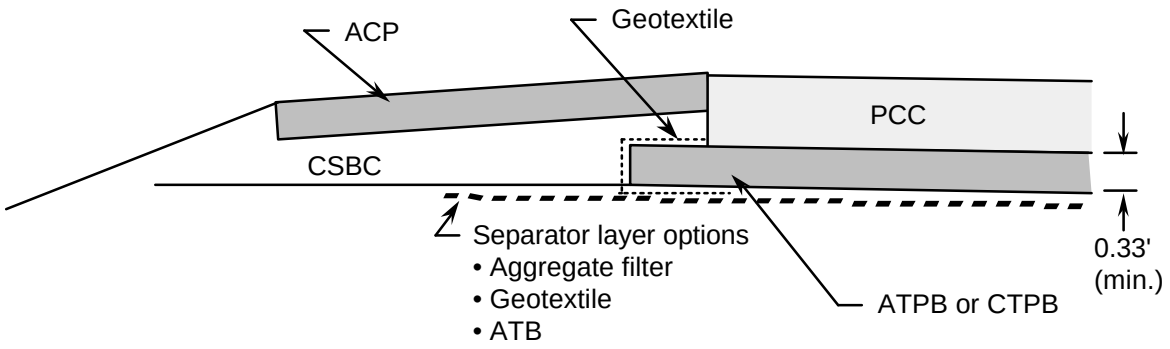
(b) Permeable Base Drains to Longitudinal Edge Drain



(c) Permeable Base Drains to Longitudinal Edge Drains

Figure 9.1. Illustration of Permeable Base Options for PCCP (ACP Surfaced Shoulders)

Detail #1



Detail #2

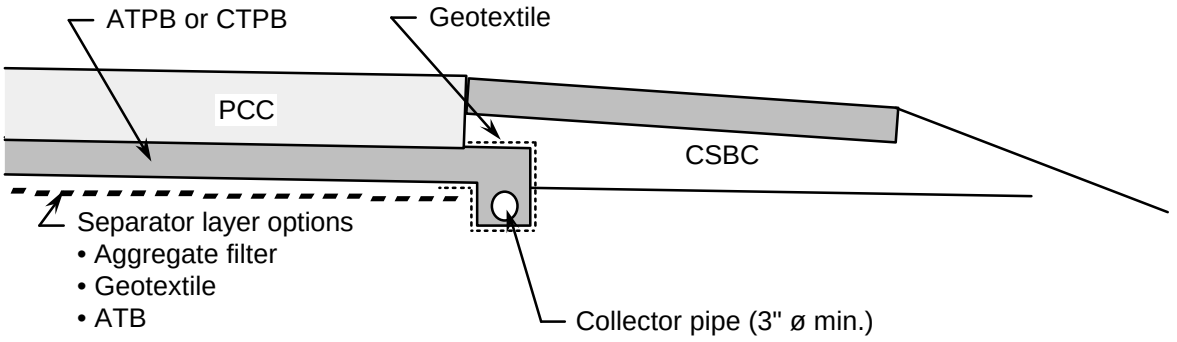


Figure 9.1. (cont.)

Detail #3

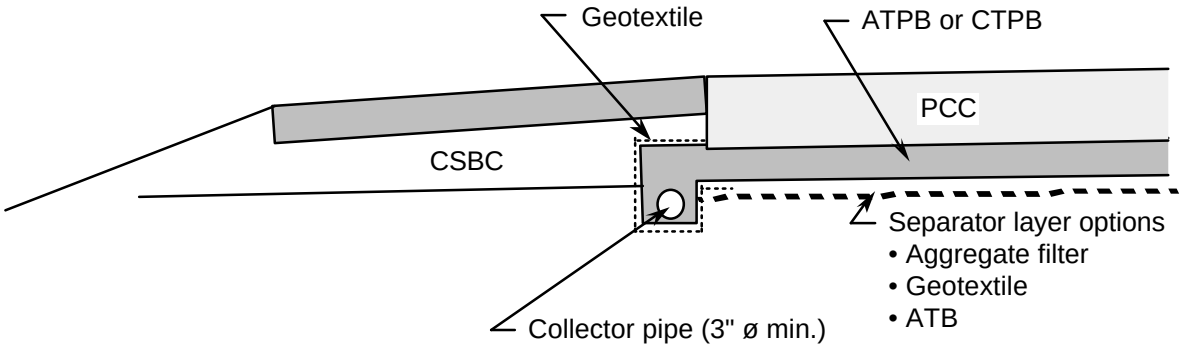
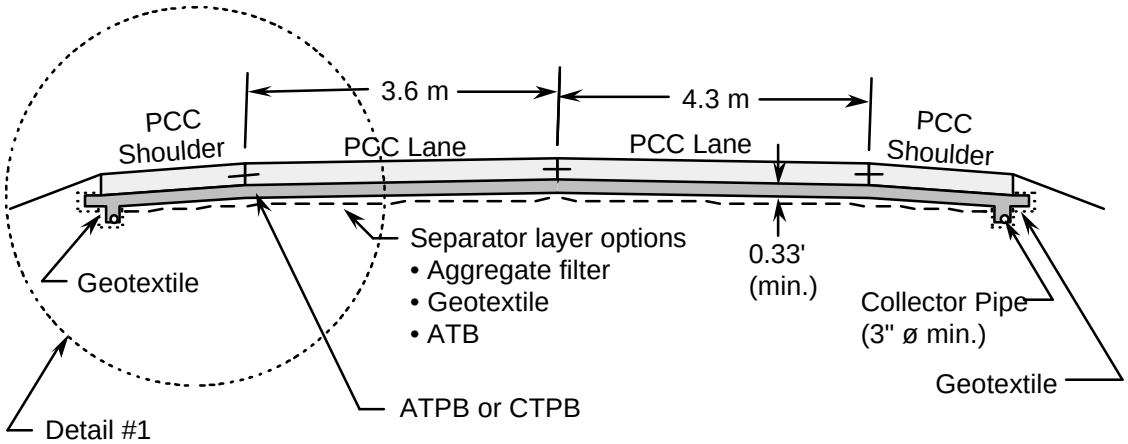


Figure 9.1. (cont.)



Detail #1

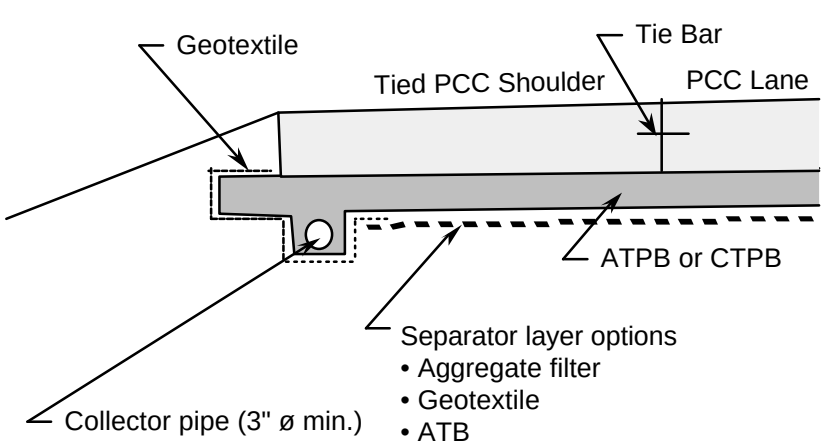
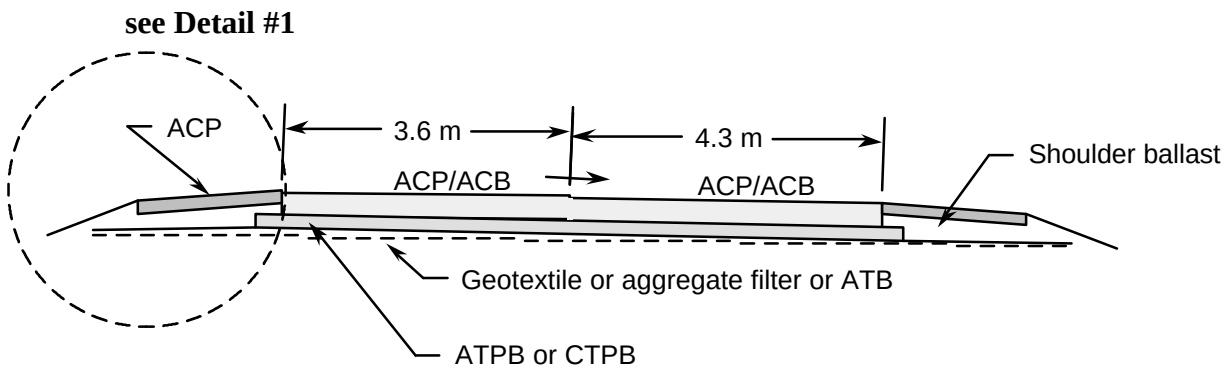
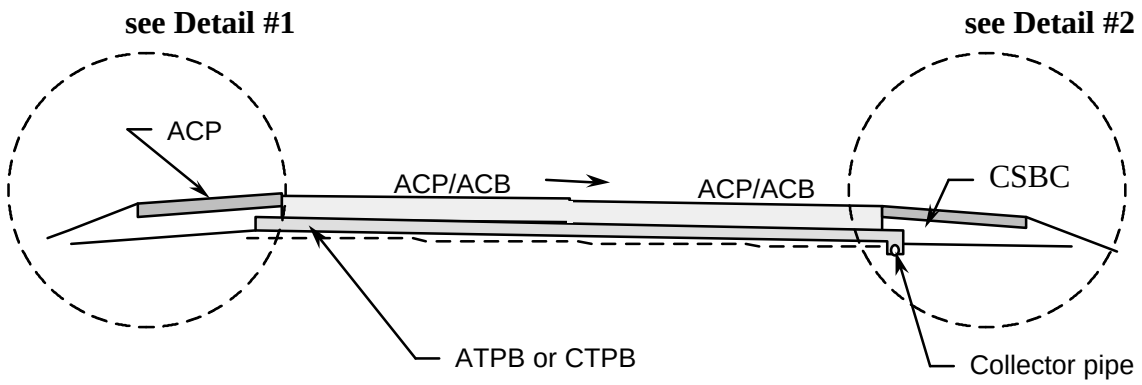


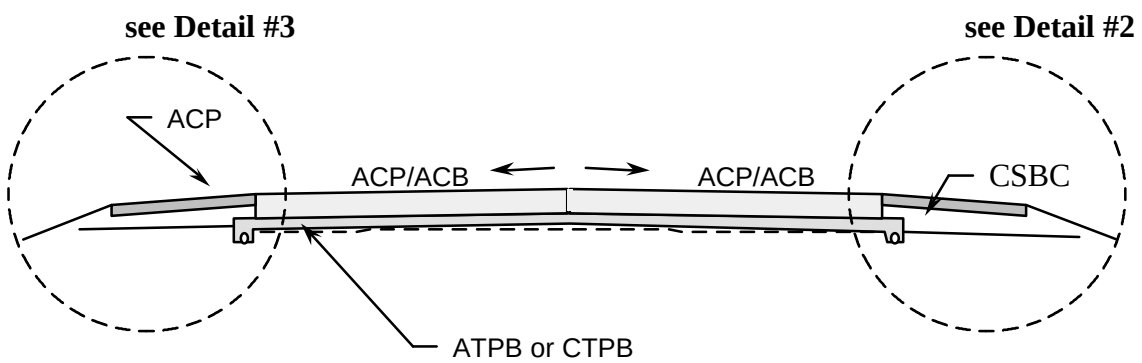
Figure 9.2. PCC Permeable Base Configuration with Tied PCC Shoulders



(a) Permeable Base Drains to Daylighted Shoulder Ballast



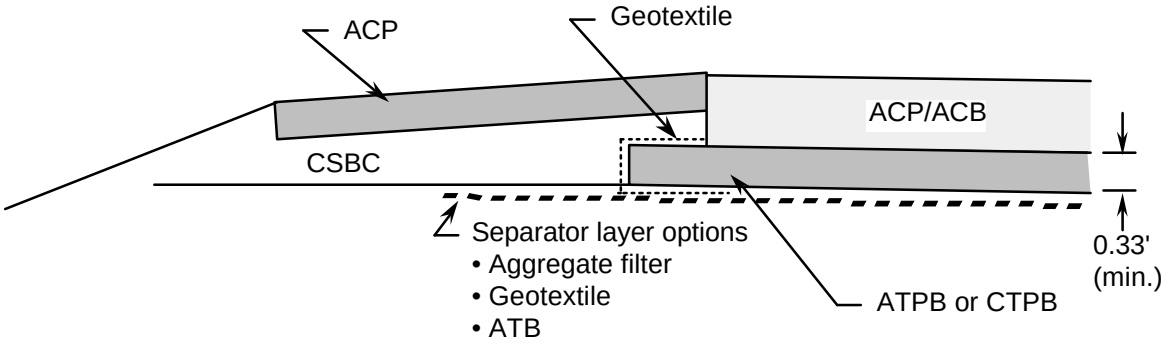
(b) Permeable Base Drains to Longitudinal Edge Drain



(c) Permeable Base Drains to Longitudinal Edge Drains

Figure 9.3. Illustration of Permeable Base Options for Flexible Pavement

Detail #1



Detail #2

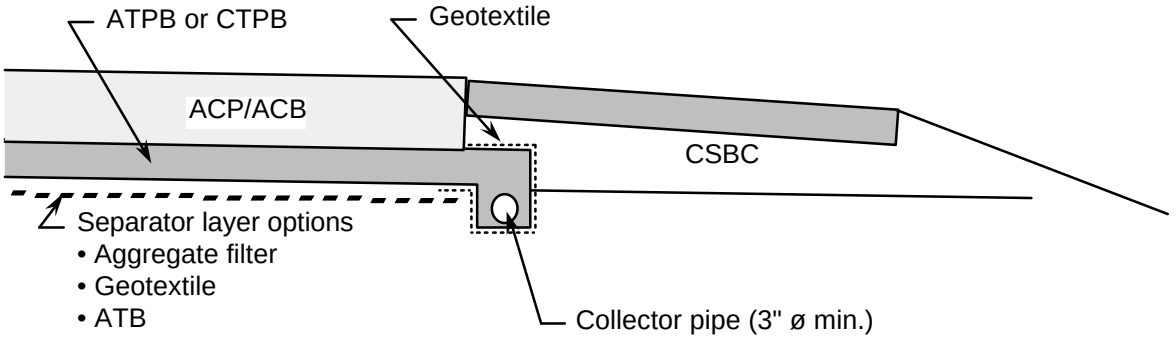


Figure 9.3. (cont.)

Detail #3

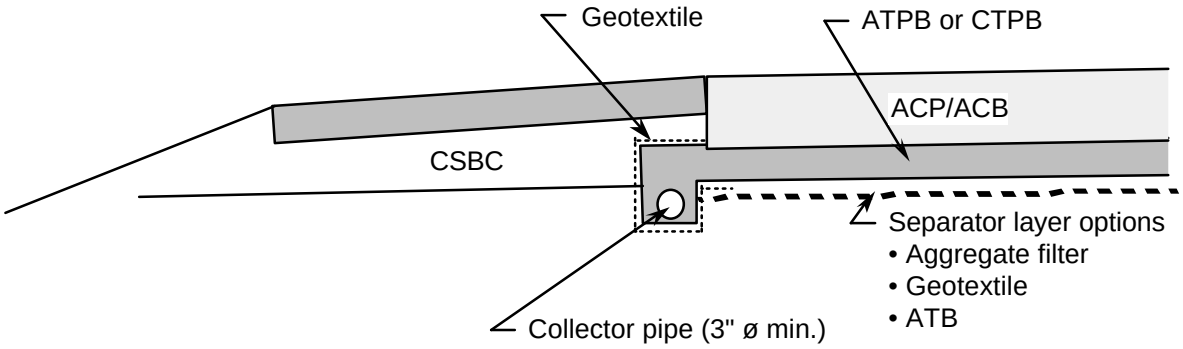


Figure 9.3. (cont.)

The minimum permeable base thickness shall be 100 mm. This thickness should be adequate if a base permeability of 900 m/day or greater is maintained.

Additional recommendations provided by the FHWA in their 1992 publication [9.6] include:

- Aggregate material
  - "Both unstabilized and stabilized permeable base material should consist of durable, crushed, angular aggregate with essentially no fines (minus No. 200 [0.075 mm] sieve). The aggregate material should have good interlock.
  - The FHWA recommends that only crushed stone be used for permeable bases.
  - L.A. Abrasion Wear should not exceed 45 percent. Aggregate material should have adequate soundness."
- Hydraulic Design of Permeable Bases
  - "Provide base material with a minimum coefficient of permeability of 1,000 ft/day. A coefficient of permeability of 2,000 to 3,000 ft/day would be preferable.
  - Provide as much slope as possible (for the permeable base that is). A minimum slope of 0.02 ft/ft is recommended.
  - Keep the length of the drainage path to a minimum.
  - Keep the length of the drainage path to a minimum.
  - A maximum outlet spacing of 250 ft is recommended.
  - Select base thickness based primarily on construction considerations. A minimum thickness of 4 in. is recommended."
- For Asphalt Stabilized Bases (ATPB)
  - "Application rate of asphalt cement should be 2 to 2.5 percent, by weight.
  - A harder grade of asphalt, AC 40 or AR 8000, is recommended.
  - For asphalt stabilized permeable bases, most SHAs specify one to three passes of a 5 to 10 ton steel wheel roller for compaction. Vibratory rollers are not recommended."

- For Cement Stabilized Bases (CTPB)
  - "Application rate of 2 to 3 bags of cement is recommended.
  - A number of SHAs have good success in using only vibrating screeds and plates of compaction."

The Oregon Department of Transportation (ODOT) completed a study in 1992 which examined free draining base materials [9.9]. Of special interest were their test results on asphalt treated permeable bases (ATPB). The ATPBs of interest were from two in-service pavements (Fir Grove Road and Rose Lodge Road). A brief summary of the laboratory results and associated conclusions include:

- Gradation

Sieve Size (mm)	ODOT Spec. Limits	Fir Grove Project*	Rose Lodge Project*	AASHTO No. 57 Spec. Limits
37.5	—	—	—	100
25.0	99-100	100	100	95-100
19.0	85-95	94	98	—
12.5	35-68	69	66	25-60
6.3	5-20	19	19	—
4.75	—	—	—	0-10
2.36	—	—	—	0-5
2.00	0-5	6	5	—
0.075	0-2	3	2	—
Asphalt Content	2-3	2.9	2.4	

*Based on extraction results

The ODOT specification limit and projects have gradations a bit finer than AASHTO No. 57.

- Resilient Modulus @ 25 °C (ASTM D4123)
  - Fir Grove Road
    - Average = 827 MPa (120 ksi)
    - Std. dev. = 331 MPa (48 ksi)
    - n = 6
  - Rose Lodge Road
    - Average = 579 MPa (84 ksi)
    - Std. dev. = 103 MPa (15 ksi)
    - n = 6

	Constant Head	Falling Head
• Permeability, m/day (ft/day)		
• Fir Grove Lane — Towers Road		
• Average	459 (1505)	611 (2005)
• Std. dev.	294 (965)	283 (929)
• n = 10		
• Rose Lodge—Polk County Line		
• Average	751 (2465)	622 (2041)
• Std. dev.	181 (595)	77 (253)
• n = 8		

- AASHTO Layer Coefficients (flexible design)

  a_{ATPB} = 0.14 - 0.19

Figure 9.1 shows three slightly different arrangements for permeable bases (PCCP with ACP shoulders). The dashed line implies that the permeable base is "wrapped" with a geotextile <u>or</u> is <u>underlain</u> by an aggregate filter or ATB. The terms "ATPB" and CTPB" are defined as "asphalt treated permeable base" and "cement treated permeable base" respectively. Such mixes should cost less than dense graded hot mix due to reduced binder and compaction costs [9.4].

Figure 9.1(a) shows that the permeable base drains into shoulder ballast which, in turn, is daylighted. WSDOT Shoulder Ballast (Standard Specifications Paragraph 9-03.9(2)) has the following specified gradation range:

Sieve Size (mm)	Percent Passing
63	100
19.0	40-80
6.3	5 max
0.150	0-2
% Fracture	75%

Recent information from the Wisconsin DOT (Roads and Streets, September 1994) illustrated the construction of a 100 mm thick CTPB layer for rigid pavement on I-90 near Stoughton, Wisconsin. The CTPB used AASHTO No. 67 stone with various amounts of portland cement. The higher cement content 150 kg/m^3 (250 lbs/yd^3) worked well under construction traffic and resulted in a permeability of 900 m/day (3,000 ft/day).

3.2 WSDOT ASPHALT TREATED PERMEABLE BASE

WSDOT ATPB is currently specified as a Special Provision (as of November 1992). The WSDOT gradation for ATPB conforms to AASHTO Grading No. 57 (as shown in Paragraph 3.1). This gradation is listed in the WSDOT Standard Specifications in Paragraph 9-03.1(4)C "Grading" as a coarse aggregate for PCC mixes. For estimating purposes, the asphalt content percentage shall range from 2.0 to 2.5 percent by weight of total mix (AR - 4000W).

The basic properties of ATPB using the AASHTO No. 57 grading have not been measured by WSDOT (as of December 1992). For now, and largely based on the ODOT research [9.9], the following properties will be used:

- Resilient Modulus @ 25 °C ≈ 690 MPa (100,000 psi)
- Permeability ≥ 300 m/day (1,000 ft/day)
- AASHTO Layer Coefficient = 0.13 (from 1993 *AASHTO Guide*, Figure 2.9, p. II-24)

3.3 CONSTRUCTION

A few notes of caution are warranted relative to construction of a permeable base. These include:

- Restrict or control construction traffic on the permeable base to minimize fine-grained material infiltration and potential permanent deformation (permeable bases do not have much "strength").

- Do not overcompact the permeable base. Many SHAs specify one or two passes of a 5 to 10 ton steel wheeled roller [9.1]. Overcompaction can cause "degradation of the material and a subsequent loss of permeability."

4. COLLECTOR SYSTEM

The cross sections shown in Figure 9.1 show two different systems for collecting the water from the permeable base — daylighted shoulder ballast or a longitudinal collector pipe. If a collector pipe is used it should be a minimum of 75 mm in diameter and conform to WSDOT Standard Specifications Paragraph 9-05.2(6) "Perforated PVC Underdrain Pipe."

Additional recommendations provided by the FHWA include [9.6]:

- "The capacity of the edgedrain system should always increase as the water flows through the system. The edgedrain capacity should be great enough to handle the flows coming to it.

- Conventional (smooth) pipe edgedrains are recommended because of their relatively high flow capacity and their ability to be maintained.
- The lateral outlet pipe should be a rigid pipe with a minimum slope of 3 percent.
- Maximum outlet of 250 ft [75 m] is recommended based on maintenance considerations.
- Concrete headwalls and outlet markers should be provided at pipe outlets.
- Subsurface drainage design should be coordinated with surface drainage. The invert of the outlet pipe should be located 6 in. [150 mm] above the 10-year design flow in the ditch.
- Edgedrain systems should be designed with maintenance in mind.

5. FILTER SYSTEM

As noted earlier, the permeable base must be protected from clogging due to infiltration of fine-grained particles. Three different options exist for achieving this.

- Use a geotextile.
- Use an aggregate filter layer.
- Place the permeable base on a stabilized base such as ATB.

The basic criteria for an appropriately graded filter layer include [from 9.1 and 9.6]

$$D_{15}\text{ (filter)} \leq 5D_{85}\text{ (subgrade)} \qquad \text{(Eq. 9.1)}$$

$$D_{50}\text{ (filter)} \leq 25\ D_{50}\text{ (subgrade)} \qquad \text{(Eq. 9.2)}$$

$$\text{Coefficient of Uniformity (CU)} = \frac{D_{60}\text{ (filter)}}{D_{10}\text{ (filter)}} \qquad \text{(Eq. 9.3)}$$

$$\text{and } 20 \leq \text{CU} \leq 40$$

where D_x = size at which "x" percent of the particles (by weight) are smaller,

and $$D_{15}\text{ (base)} \leq 5D_{85}\text{ (filter)} \qquad \text{(Eq. 9.4)}$$

$$D_{50}\text{ (base)} \leq 25D_{50}\text{ (filter)} \qquad \text{(Eq. 9.5)}$$

where Equation 9.1 is a filter requirement,

Equation 9.2 is a uniformity requirement
(i.e., the gradation curves are in "balance"),

Equation 9.4 controls piping, and

Equation 9.5 is a uniformity criteria for piping resistance.

From Reference 9.8, the following explanation of the Coefficient of Uniformity term (CU) is provided:

The CU term is an indication of the grading of a material.

> "For example, a uniform (one-size) material will have a small CU because the size of the D_{60} material is very similar in size to that of the D_{10}. Because it consists primarily of one-size material, it contains insufficient fines to fill the voids between the larger particles and consequently it will have an open, more porous structure despite compaction. As a result, it will be more easily displaced under load and have less supporting power. The most uniform granular material commonly encountered in engineering is Standard Ottawa sand, which has a CU of approximately 1.1. Conversely, a well-graded material will have a large CU because the D_{60} will be much larger than D_{10}. A well-graded dense aggregate base material plotted on the maximum density line (recall discussion of 0.45 power maximum density line in SECTION 2.0) will have a CU between 50 and 60. A well-graded material is relatively stable, can be readily compacted to a very dense condition, and will develop high shear resistance and bearing capacity."

If a permeable base with a AASHTO No. 57 mid-range grading is used, both WSDOT Crushed Surfacing Base Course (CSBC) and Crushed Surfacing Top Course (CSTC) mid-range gradations should be able to meet the filter requirements set forth in Equations 9.1 through 9.5. Naturally, whether CSBC or CSTC can be adequate filters depends on the actual gradation of the permeable base and the final gradations of CSBC and CSTC. The preceding suggests that "traditional" WSDOT unstabilized materials at least could serve as suitable filter layers.

6. PERMEABILITY OF WSDOT MATERIALS

By use of data originally developed by Cedergren et al. [9.2] and Ridgeway [9.3], a very approximate estimate of permeability for some of WSDOT's unstabilized materials was made (again, using mid-range specification gradations):

- Shoulder Ballast ≈ 6 000 m/day (20,000 ft/day) or less
- Crushed Surfacing Base Course ≈ 0.01 to 3 m/day (0.05 to 10 ft/day)
- Crushed Surfacing Top Course ≈ 0.01 to 3 m/day (0.05 to 10 ft/day)

Thus, with the exception of Shoulder Ballast, the "traditional" base course materials used by WSDOT cannot be considered as permeable.

A generalized procedure for estimating the permeability of granular and filter materials was presented by Moulton [9.5]. Specifically, to estimate the coefficient of permeability:

$$k = \frac{621{,}400\ (D_{10})^{1.478}\ (n)^{6.654}}{(P_{200})^{0.597}} \qquad \text{(Eq. 9.9)}$$

where k = coefficient of permeability (ft/day),

D_{10} = effective grain size at 10 percent passing (mm),

P_{200} = percent passing the No. 200 sieve,

n = porosity $= \left(1 - \frac{\gamma_d}{62.4(G)}\right)$,

G = specific gravity (generally assumed ≃ 2.70), and

γ_d = dry density (lb/ft^3).

For example, if CSTC has P_{200} = 5 %
D_{10} = 0.2 mm
γ_d = 120 lb/ft^3

then $k \simeq$ 5.5 ft/day (by use of Eq. 9.9).

P_{200} = 10 %
D_{10} = 0.074 mm
γ_d = 120 lb/ft^3

then $k \simeq$ 0.8 ft/day (by use of Eq. 9.9).

7. MAINTENANCE

Some of the advice offered by the FHWA [9.6] on maintenance of a pavement subsurface drainage system follows.

7.1 OUTLETS

SHAs should use concrete headwalls, reference markers or painted arrows to help maintenance personnel locate outlets for cleaning purposes. Mow or spray vegetation around the outlet.

7.2 EDGEDRAINS

Edgedrains should be inspected at least once a year to determine their condition. Flushing of the pipe system should be performed as necessary.

7.3 BOTTOM LINE

If poor (or no) maintenance of the drainage system is performed, there is a real risk that the pavement section will become flooded. This is unacceptable in that pavement deterioration will undoubtedly be increased.

8. BASIS FOR WSDOT DESIGN

Permeable bases must drain within a relatively short time in order to minimize moisture damage to the pavement structure. One set of criteria provided in the 1986 *AASHTO Guide for Design of Pavement Structures,* Vol. 2, Appendix DD is based on draining 50 percent of the free water as follows:

Quality of Drainage	Time to Drain
Excellent	2 hrs
Good	1 day
Fair	7 days
Poor	1 month
Very Poor	Does not drain

The FHWA in "Drainable Pavement Systems" [9.6] recommends the following:

- Highest class roads with greatest amount of traffic: 1 hour to drain 50 percent of the drainable water.

- Most other Interstate highways and freeways: 2 hours to drain 50 percent of the drainable water.

Procedures contained in Reference 9.6 were used to estimate the time required for a permeable base to drain 50 percent of the water from a "storm" (the input values were coefficient of permeability (k) = 1,000, 3,000 and 10,000 ft/day, resultant slope of the drainage layer (S_R) = 0.02 ft/ft (taken to be the same as the standard WSDOT cross slope), resultant length of drainage distance (L_R) = 12, 24 36 ft (i.e., the drainage distance is one, two or three lanes), thickness of drainable base (H) = 0.33 ft, effective porosity = (porosity) (0.8), porosity = 1 - γ_d/(62.4) (G_{sb}), γ_d = 120 lb/ft^3, G_{sb} = 2.7). The results are:

Input Values		Time to drain 50 % of Water
k	Pavement Width (L_R)	
1,000 ft/day	12 ft	0.7 hr
1,000 ft/day	24 ft	1.8 hr
1,000 ft/day	36 ft	3.0 hr
3,000 ft/day	12 ft	0.2 hr
3,000 ft/day	24 ft	0.6 hr
3,000 ft/day	36 ft	1.0 hr
10,000 ft/day	12 ft	0.1 hr
10,000 ft/day	24 ft	0.2 hr
10,000 ft/day	36 ft	0.3 hr

Thus, the above numbers can be used to suggest that WSDOT drainable bases <u>should</u> have coefficient of permeabilities of 900 m/day (3,000 ft/day) or better. Further, the drainage path can include up to three lanes if the lowest coefficient of permeability examined is excluded (1,000 ft/day). The minimum drainable layer thickness of 100 mm (0.33 ft) appears to be adequate.

Next, the edgedrain capacity must be checked. To estimate the edgedrain flow rate, the following equation can be used (time to drain approach) from Reference 9.6:

$$Q_p = (W)\,(L)\,(H)\,(n_e)\,(u)\left(\frac{1}{t_D}\right)(24)$$

where Q_p = design flow rate for pipe flow, ft^3/day,
W = width of permeable base, ft,
L = outlet spacing, ft,
H = thickness of base, ft,
n_e = effective porosity,
u = percent drained, and
t_D = drainage time period, hrs.

If we let

$L = 250$ ft,
$H = 0.33$ ft,
$n_e = 0.23$,
$u = 0.5$ (or 50 %),
$t_D = 1$ hr, and
$W = 12$, 24, and 36 ft,

then $Q_p = 2730$ ft^3/day (W = 12 ft),
$Q_p = 5460$ ft^3/day (W = 24 ft), and
$Q_p = 8200$ ft^3/day (W = 36 ft).

The capacity of a circular pipe flowing full can be estimated by use of Manning's equation [9.6]:

$$Q = \left(\frac{53.01}{n}\right)(D)^{2.67}(S)^{0.5}$$

where Q = pipe capacity, ft^3/day,
D = pipe diameter, in.,
S = slope, ft/ft, and
n = Manning's roughness coefficient
= 0.012 (smooth pipe)
= 0.024 (corrugated pipe).

If we use

$D = 3$ in.,
$S = 0.01$ ft/ft (1% slope), and
$n = 0.012$ (smooth pipe),

then $Q \simeq 8{,}300$ ft^3/day.

Thus, for a pipe diameter of 75 mm (3 in.), an outlet spacing of 75 m (250 ft) and a minimum slope of 1 percent, the maximum flow from a 100 mm (4 in.) thick permeable base should be accommodated for up to three traffic lanes. Clearly, a 75 mm (3 in.) diameter edgedrain pipe will suffice for most conditions. A 100 mm (4 in.) diameter pipe can accommodate a slope of about 0.25 percent for up to three lanes of traffic (other assumptions the same); however, the FHWA recommends a minimum slope of 0.35 percent.

9. SUMMARY

The permeable base sections underlying PCC as shown in Figure 9.1 should function well in most parts for Washington state. Naturally, local conditions may require a special design. The FHWA report entitled "Highway Subdrainage Design" [9.5] or "Drainable Pavement Systems" [9.6] should be used if a special design is contemplated.

10. COMMENTS ON WASHINGTON WEATHER

Believe it or not there are places in the world which receive substantially more rainfall than, say, western Washington. For example, a few extreme locations include (after Griffiths [9.7]):

• Highest annual total precipitation	26 467 mm Cherrapunji, India
• Highest annual mean precipitation	11 684 mm Kauai, Hawaii
• Highest one day total	1 870 mm Cilaos, Reunion
• Highest one minute total	31 mm Unionville, Maryland
• Highest number of rain days per year on average	325 days/yr Bahia Felix, Chile

The world climate records set within Washington State relate to snow:

• Highest annual total snow	28 500 mm or 93.5 ft Paradise Ranger Station, Mt. Rainier, Washington
• Highest annual mean snow	14 783 mm or 48.5 ft Mt. Rainier, Washington

SECTION 9.0

REFERENCES

9.1 Federal Highway Administration, "Subsurface Pavement Drainage," Draft Technical Advisory, Federal Highway Administration, Washington, D.C., April 19, 1990.

9.2 Cedergren, H.R., O'Brien, K.H., Armas, J.A., "Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections," Report FHWA-RD-72-30, Federal Highway Administration, Washington, D.C., June 1972.

9.3 Ridgeway, H.H., "Pavement Subsurface Drainage Systems," NCHRP Synthesis No. 96, Transportation Research Board, Washington, D.C., November 1982.

9.4 Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D. and Kennedy, T.W., *Hot Mix Asphalt Materials, Mixture Design, and Construction*, NAPA Education Foundation, Lanham, Maryland, 1991.

9.5 Moulton, L.K., "Highway Subdrainage Drainage," Report FHWA-TS-80-224, Federal Highway Administration, Washington, D.C., 1980.

9.6 Federal Highway Administration, "Drainable Pavement Systems Participant Notebook," Publication No. FHWA-SA-92-008, Federal Highway Administration, Washington, D.C., March 1992.

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SECTION 10.0

CONSTRUCTION CONSIDERATIONS

This SECTION is used to overview a few of the many important pavement construction aspects commonly encountered on WSDOT projects.

1. INTRODUCTION

This SECTION will be composed of seven subsections (including this one) the purpose of which is to note some of the pavement construction considerations which are important.

The governing documents for WSDOT pavement design and operations are, as always, the applicable WSDOT manuals and Standard Specifications. This SECTION contains selected information from those documents.

The subsequent construction considerations will be divided into the following subsections:

- subgrade/embankments
- bituminous surface course treatments
- asphalt concrete
- portland cement concrete
- surface smoothness
- rehabilitation and maintenance

2. SUBGRADE/EMBANKMENTS

2.1 RECYCLED MATERIALS

There is a need to recycle various materials so that they do not go to landfills. As such, it is appropriate to use materials such as glass, broken PCC, etc., in the roadway embankment. With proper blending with other materials little or no decrease in embankment "strength" need result. If the recycled materials are placed deep within an embankment (1.5 to 3.0 m or deeper), then "strength" is relatively unimportant but resistance to excessive consolidation (settlement) is.

3. BITUMINOUS SURFACE TREATMENTS

3.1 TERMINOLOGY

Some of the various types of seals which have (or can be) used by WSDOT include (from Wyckoff [10.4]):

(a) Fog Seal

"An application of a dilute emulsion without an aggregate cover." The "purpose is to seal and enrich an under-asphalted surface or to tighten and waterproof an open texture pavement. Fog seals can also be used as preseals."

(b) Sand Seal

A sand seal "is an application of an asphalt followed by a sand cover aggregate." It "is intended to be used on pavements that have lost some of their matrix, and it is desirable for tightening the pavement texture and reducing ravelling."

(c) Slurry Seal

"It is a mixture of a specially graded aggregate and an asphalt emulsion and is applied with a squeegee device. A slurry will seal an existing pavement and produce some minor leveling without the inconvenience of base leveling of loose cover stone. It can also be used for mass crack filling, to improve skid resistance, to enhance appearance, and to reduce studded tire wear."

(d) Cape Seal

"A cape seal is a chip seal topped with a slurry seal. A cape seal produces a seal with no loose cover stone. The maximum size of chip used for the seal coat will establish the depth of the mat." This technique apparently originated in South Africa, hence the name "Cape" after Cape Town, South Africa.

(e) Chip Seal (Seal Coat)

"A chip seal is an application of asphalt followed with an aggregate cover" and "is constructed to produce an initial pavement or maintain an existing asphalt pavement."

3.2 APPLICATION RATES

From the 1994 WSDOT Standard Specifications:

BST Type	Applied Asphalt, ℓ/m^2 (gal/yd^2)	Aggregate Size, mm (in.)	Applied Aggregate, kg/m^2 (lb/yd^2)
• Class A			
• Prime Coat	1.6-2.5 (0.35-0.55)	12.5-6.3 or 19-12.5 (1/2-1/4 or 3/4-1/2)	14-22 (25-40)
• Tack Coat	1.6-2.3 (0.35-0.50)	12.5-6.3 (1/2-1/4) 6.3-0 (1/4-0)	14-16 (25-30) 2-3 (4-6)
• Class B	1.8-2.7 (0.40-0.60)	16.0-6.3 (5/8-1/4) 6.3-0 (1/4-0)	14-22 (25-40) 2-3 (4-6)
• Class C	1.6-2.3 (0.35-0.50)	12.5-6.3 (1/2-1/4) 6.3-0 (1/4-0)	11-16 (20-30) 2-3 (4-6)
• Class D	0.9-1.6 (0.20-0.35)	9.5-2.0 (3/8-No. 10)	10-14 (18-25)
• Preseal	0.7-0.9 (0.15-0.20)	6.3-0 (1/4-0)	4-8 (8-15)

3.3 CONSTRUCTION PRACTICES

The following practices are from the WSDOT reports "Asphalt Seal Coats" [10.4] and "1989 Westside Chip Seal Study" [10.5].

(a) Method for determining asphalt application rate

After the seal coat has been rolled, remove a random sample of the largest rocks. The embedment of the rock into the asphalt should be 50 to 70 percent.

(b) Rock application

Place the aggregate as quickly as possible into the applied asphalt (WSDOT Standard Specification Paragraph 5-02.3(3). "Application of Asphalt" requires this to take place within <u>one minute</u>). The asphalt must be <u>fluid</u> so the rock can be properly embedded and the asphalt is the <u>most fluid</u> immediately after spray application.

(c) Rolling

Steel wheeled rollers are preferred on Class A prime coat shots on a new granular base or granular overlay. Pneumatic rollers are preferred on seal coats since the rubber tires will minimize rock fractures and compact in depressions. Rolling "orients" the rock (i.e., rock flat sides down).

(d) Choke

Choke is 6.3-0 mm (1/4 in. - 0) aggregate (according to Jackson et al. [10.5]; 9.5-2.0 mm (3/8 in.-No. 10) stone can be used to reduce dust problems occasionally associated with the 6.3-0 mm (1/4 in.-0 rock)). Choke stone is used to prevent new seal coat rock from being "turned" or "rolled over" under traffic. The choke stone essentially "locks" the seal coat aggregate in place. The choke should be rolled before the highway is opened to traffic.

(e) Post-seal inspection

Check for aggregate embedment (see (a)). If inadequate application of asphalt occurred, a fog seal can be used to "tie down" the seal rock. The fog seal is generally applied at a rate of 0.5 to 0.9 ℓ/m^2 (0.1 to 0.2 gal/yd^2) (CSS-1 asphalt emulsion diluted with an equal amount of water). A tighter seal can be achieved by use of choke stone.

(f) Miscellaneous

- Polymer modified asphalt:

  Such binders have shown numerous advantages such as (from Jackson et al. [10.5]):

  - Less windshield damage
  - Better aggregate adhesion
  - Less rock loss due to brooming
  - Open to traffic sooner

- Class D seals with the smaller maximum aggregate size provide a smoother ride for bicycles.

- Causes of BST related flushing (bleeding or excessive binder on the pavement surface):
    - Bleed throughs from existing flushed pavements and cold mix patches
    - Too much asphalt
    - Improper construction of transverse joints
    - Allowing asphalt emulsion to "break" before applying aggregates
    - Improper crack sealing techniques and/or materials preceding BST application
    - Causes of BST related raveling:
        - Dry or open pavements
        - New hot mix patches
        - Shaded areas
        - Excessive amounts of applied aggregate
        - Aggregate too wet or dirty
        - Allowing asphalt emulsions to break before applying aggregates
        - Late season paving
        - Too little asphalt binder

    Sensible steps can be taken to mitigate all of the above causes of flushing and raveling (refer to reports such as References 10.4 and 10.5).

3.4 LATE SEASON PAVING

Placing BSTs (all classes including Class A (penetration treatment) and Classes B, C, D (seal coat)) during late August or September all too often results in poor pavement performance. A principal reason for this is that most WSDOT BSTs use a CRS-2 emulsified asphalt. This binder must have an adequate amount of warm weather to cure properly. Late season BST paving projects often do not allow the cure process to fully occur. The result is a loss of rock and, eventually, severe flushing.

Specific provisions in the WSDOT Standard Specifications (1994) include:

- Paragraph 5-02.3(10) Unfavorable Weather

    "The application of any asphalt (binder) to the roadway shall be restricted to the following conditions:

    - The roadway surface temperature shall be at least 16 °C (60 °F) and the air temperature at least 16 °C (60 °F) and raising, or

- The air temperature shall not be less than 21 °C (70 °F) when falling and the wind shall be less than 10 miles per hour as estimated by the Engineer."

Additionally noted is that "Construction of bituminous surface treatments on any traveled way shall not be carried out before May 15 or after August 15 of any year except upon written order of the District Administrator."

4. ASPHALT CONCRETE

4.1 AIR VOIDS

For dense asphalt concrete (AC), it is extremely important that the AC be adequately compacted. An approximate "rule-of-thumb" is for every 1 percent increase in air voids (above 6-7 percent), a minimum of 10 percent of the pavement life is lost (after Linden et al. [10.67]).

According to Roberts et al. [10.1], there is considerable evidence that dense graded mixes should not exceed 8 percent nor never fall below 3 percent air voids during the service life of the layer. Why?

- High air voids allow water and air to penetrate the mix, potentially resulting in accelerated
  - water damage,
  - oxidation of the binder,
  - raveling, and
  - cracking.

- Low air voids contribute to rutting and shoving of asphalt mixtures.

A 1987 survey by Linden et al. [10.6] of the 50 SHAs (48 responded to the questionnaire) revealed the following with respect to air voids and compaction of AC:

- Construction Compaction Requirement
  - 18 SHAs (38 percent) use percent of laboratory compacted density,
  - 21 SHAs (44 percent) use percent of theoretical maximum density (includes WSDOT), and
  - 6 SHAs (12 percent) use percent of control strip density.

- Construction Compaction Control Tests
  - 18 SHAs (38 percent) use nuclear gages,
  - 12 SHAs (24 percent) use AC cores, and
  - 18 SHAs (38 percent) use both the nuclear gage and AC cores (the AC cores are probably used to calibrate the nuclear gage).
- Minimum Field Density Limits

Specification Limit	Number of SHAs
90% of TMD*	1
91% of TMD	1
92% of TMD	12
92.5% of TMD	2
93% of TMD	5
93% of Marshall Density	1
95% of Marshall Density	9
96% of Marshall Density	2
97% of Marshall Density	1
95% of Field Marshall Density	2
98% of Field Marshall Density	1
95% of Control Strip Density	1
97.5% of Control Strip Density	1
98% of Control Strip Density	4
95% of Hveem Density	1
Other Limits Used	4
	48

*TMD = theoretical maximum density

- Minimum Compacted Course Thickness Requiring Compaction Control

Minimum Thickness (mm)	Number of SHAs
19.0	5
25.0	13
28.5	1
31.5	5
37.5	10
50.0	2
No minimum	8
No answer	4
	48

- Average and Range of Field Air Voids in Pavements Constructed in Past Five Years (i.e., 1983-1987)

	Pavement Air Voids (%)	
	Average	Range
• Maximum %	9.9	5-15
• Minimum %	3.5	1-6
• Average %	6.5	2.8-10

- Typical "Life" of AC Surfacing for each SHA

"Life was defined as the time between construction and the time when the next overlay or rehabilitation was needed.

Average Pavement Life (yrs)	Number of SHAs	Average Pavement Life (yrs)	Number of SHAs
7	1	12.5	5
8	2	13	1
9	3	13.5	3
10	10	15	7
10.5	1	17.5	3
11	1	20	2
12	6	21	1
			46

The mean is 12.5 years with 74 percent of the SHAs reporting an average life of 10 to 15 years.

- Principal Mode of AC Failure at the End of Surfacing Life

Mode	Number of SHAs*
Fatigue Cracking	20
Rutting	14
Cracking (non-specific)	12
Thermal Cracking	6
Stripping	5
Raveling	3
Reflective Cracking	2
Other Modes	10

*Multiple modes noted by some SHAs.

- Pavement Life Reduction as a Function of Air Voids

Three separate sources were used by Linden et al. to develop the information shown below — results in the literature, the SHA survey, and results from the WSPMS:

Air Voids (%)	Pavement Life Reduction (%)		
	Literature*	SHA Survey**	WSPMS
7	0	7	0
8	10	13	2
9	20	21	6
10	30	27	17
11	40	38	—
12	50	46	36

*Lower bound of range
**Average

Up to 1990, WSDOT used a compaction requirement of 92 percent of TMD. After 1990, WSDOT changed to a quality assurance AC specification and modified the compaction requirement accordingly. An early assessment of the net effect conducted in 1989 suggested that air void levels decreased slightly on QA projects (vs. non-QA projects).

4.2 MINIMUM PAVING TEMPERATURES

A number of problems can occur when hot-mixes are placed at temperatures too low to allow for adequate compaction. The problem is largely one of viscosity of the asphalt cement binder which increases substantially with decreasing temperature. Three distinct situations are noted:

- cessation temperature,
- late season paving, and
- night paving operations.

4.2.1 COMPACTION TEMPERATURES

Compaction of asphalt concrete must be completed before the mat temperature drops below 79 °C (175 °F) (often referred to as the "cessation temperature"). The rate at which asphalt concrete cools is a function of (after Roberts et al. [10.1]):

- mat thickness,
- temperature of the mat when placed,
- ambient (air) temperature,
- temperature of the base on which the AC is placed, and
- wind conditions.

The WSDOT Standard Specifications (1994) recognize these conditions by noting:

- Paragraph 5-04.3(10)A: "Vibrating rollers shall not be operated in the vibratory mode when the internal temperature of the mix is less than 175 °F ···[or]···when checking or cracking of the mat occurs at a greater temperature."

- Paragraph 5-04.3(16) Weather Limitations: "Asphalt concrete shall not be placed on any wet surface, or when the average surface temperatures are less than those specified in the following table, or when weather conditions otherwise prevent the proper handling or finishing of the bituminous mixtures:

Surface Temperature Limitations

Compacted Thickness	Surface Course	Sub-Surface Courses
< 30 mm (0.10 ft)	13 °C (55 °F)	13 °C (55 °F)
30 to 60 mm (0.10 to 0.20 ft)	7 °C (45 °F)	2 °C (35 °F)

Surface Temperature Limitations

Compacted Thickness	Surface Course	Sub-Surface Courses
63 to 105 mm (0.21 to 0.35 ft)	2 °C (35 °F)	2 °C (35 °F)
> 105 mm (0.35 ft)	—	-4 °C (25 °F)*

*Only on dry subgrade, not frozen and when air temperature is rising."

To further reinforce this important element of good AC compaction, the time required for dense AC to cool to 79 °C (175 °F) for various mat thicknesses, base and mix temperatures follow (after Roberts et al. [10.1]):

Mat Thickness	Mix Temperature	Base Temperature	Approximate Time to Cool to 79 °C (175 °F)*
25 mm (1")	149 °C (300 °F)	16 °C (60 °F)	8 minutes
25 mm (1")	149 °C (300 °F)	-4 °C (25 °F)	6 minutes
50 mm (2")	121 °C (250 °F)	16 °C (60 °F)	12 minutes
50 mm (2")	121 °C (250 °F)	-4 °C (25 °F)	8 minutes
105 mm (4.2")	121 °C (250 °F)	16 °C (60 °F)	37 minutes
105 mm (4.2")	121 °C (250 °F)	-4 °C (25 °F)	29 minutes

*Assumes wind velocity of 10 knots and air temperature same as the base temperature.

A somewhat better way to examine this issue is to list the time for the mat to cool to 79 °C (175 °F) for the surface temperature limitations listed in Paragraph 5-04.3(16) of the WSDOT Standard Specifications:

Compacted Thickness	Surface Temperature	Approximate Time to Cool to 175 °F (79 °C)
< 30 mm (0.10 ft) (use 0.10 ft and mix temp = 121 °C (250 °F)	13 °C (55 °F)	5 minutes
30 to 60 mm (0.10 to 0.20 ft) (use 0.15 ft and mix temp = 121 °C (250 °F)	7 °C (45 °F)	9 minutes
63 to l05 mm (0.21 to 0.35 ft) (use 0.28 ft and mix temp = 121 °C (250 °F)	2 °C (35 °F)	23 minutes
> 105 mm 0.35 ft (use 0.35 ft and mix temp = 121 °C (250 °F)	-4 °C (25 °F)	30 minutes

4.2.2 LATE SEASON PAVING

The primary problem with late season paving is achieving adequate compaction of the hot-mix. The problems associated with inadequate compaction were discussed earlier in Paragraph 4.2.1; however, as noted in the WSDOT Standard Specifications in Paragraph 5-04.3(10)A:

> "The type of rollers to be used and their relative position in the compaction sequence shall generally be the Contractor's option, provided specification densities are attained. An exception shall be that the pneumatic tired roller shall be used between October 1 and April 1. Coverages with a vibratory or steel wheel roller may precede pneumatic tired rolling."

Further, WSDOT Standard Specifications, Paragraph 5-04.3(16) states:

> "Asphalt concrete pavement for wearing course shall not be placed on any traveled way after October 15 without written approval from the District Administrator."

The use of pneumatic tired rollers in late season paving can be explained by the general advantages of this type of roller [10.1]:

- provide more uniform compaction than steel-wheeled,
- provide a tighter, more dense surface, and

- compact the mix without causing checking and in removing checking caused by steel-wheeled rollers.

Secondary compaction due to traffic is not likely to occur for late season paving. The secondary compaction is viewed as a benefit in that it helps to further seal and consolidate the pavement surface.

4.2.3 NIGHT PAVING OPERATIONS

The cessation temperature of 79 °C (175 °F) discussed previously is applicable day or night. Naturally, this can be a problem for night paving even during the so called construction season. Adequate mix temperature is particularly difficult to maintain for WSDOT Class D (open graded wearing course). Due to the open texture of the mix, it cools at a higher rate than do mixes such as WSDOT Class A or B. Care should be taken in using this mix for a night paving operation. Note that Paragraph 5-04.3(16) of the WSDOT Standard Specifications states "asphalt concrete Class D shall not be placed when the air temperature is less than 60 °F [16 °C]"

4.3 PRELEVEL VS. GRINDING

Given the advancements in grinding equipment, this process may, for many situations, be a superior alternative to preleveling. Prelevel quantities are difficult to accurately estimate and the thickness of pre-level dictates that density testing with nuclear gages cannot be done. Thus, adequate mix density is difficult to achieve and measure.

4.4 OTHER CONSIDERATIONS FOR LAYER THICKNESS

4.4.1 MAXIMUM LIFT THICKNESSES

Paragraph 5-04.3(9) "Spreading and Finishing" of the WSDOT Standard Specifications notes that the nomial maximum depth of any layer of any course shall not exceed the following:

AC Class E	105 mm (0.35 ft)
AC Class and B (used as a Base Course)	105 mm (0.35 ft)
AC Class A, B and F	75 mm (0.25 ft)
AC Class G	30 mm (0.10 ft)
AC Class D	24 mm (0.08 ft)

4.4.2 THIN OVERLAYS

Thin overlays of about 30 mm (0.1 ft) or less can exhibit a variety of performance problems. Some of the causes of these problems were listed by Roberts et al. [10.1]:

- cool too quickly to be adequately compacted (i.e., high air voids),
- susceptible to debonding from the prior surfacing, especially when placed on weak pavement sections, and
- are not thick enough to provide substantial structural improvement or overcome modest levels of pavement roughness.

Interestingly, the Maintenance Superintendents of District 3 strongly concur with the above deficiencies. District 3 has traditionally (up until 1992) used a large number of thin Class G overlays.

The minimum overlay thickness commonly used by WSDOT which requires density control is 37 mm (0.12 ft) (any overlay thickness greater than 30 mm (0.10 ft) requires compaction control); however, a minimum thickness of 46 mm (0.15 ft) is strongly encouraged. The thicker overlay significantly increases the compaction time (refer to Paragraph 4.2.1). Further, the additional 9 mm (0.03 ft) of thickness (a 25 percent increase) should not increase the overlay cost by 25 percent due to various fixed costs associated with paving contracts (for WSDOT and the contractor).

4.4.3 OTHER DESIGN AND CONSTRUCTION ISSUES-PHOTOGRAPHS ONLY

5. PORTLAND CEMENT CONCRETE PAVEMENT

5.1 WIDENING PROJECTS

When existing PCCP is to be widened with additional PCCP, the following items should be followed:

- tie the slabs ("old" and "new") together with tie bars,
- match the "old" and "new" joints to the extent possible.

Further, the widened ("new") PCC should be a full lane width whenever possible. PCC trends to crack in a more or less square pattern and as such the length to width ratio (length is measured as the distance between contraction joints) should be 1:1. Often, this is not possible particularly on Interstate widening projects (HOV lanes, etc.). All too often added PCC sections of about 1.2 to 1.8 m (4 to 6 ft) with a 4.6 m (15 ft) contraction joint spacing will eventually crack into a square pattern with the resulting loss of slab structural integrity.

5.2 LANE WIDTHS/TIED SHOULDERS

The outside lane, whenever possible, should have a 4.3 m (14 ft) width. This results in lower edge stresses and improved pavement performance.

Tied PCC shoulders (tied by use of tie bars to the outside lane) is an effective means to reduce stresses in the outside lane slabs thereby enhancing pavement performance of the pavement system. The PCC shoulder slab thicknesses should be the same as the outside lane. Tied PCC shoulders are most beneficial in new or reconstructed PCC projects. Retrofitting tied PCC shoulders to an existing PCC lane appears to be of limited benefit (refer to SECTION 7.0, Paragraph 2.2.4).

5.3 JOINTS

5.3.1 TYPES OF JOINTS

There are three principal joints which will be discussed:

- contraction joints
- longitudinal joints
- construction joints

5.3.1.1 Contraction Joints

The contraction joint spacing used up to 1993 by WSDOT (repeated spacing of 2.7, 3.0, 4.3, and 4.0 m (9, 10, 14 and 13 ft)) has exhibited adequate performance to date. It is anticipated that greater use of dowel bars will occur in the future. When dowel bars are used the random spacing and skew need not be used. Instead, a standard, straight 3.7 to 4.6 m (12 to 15 ft) contraction joint spacing should be used to simplify the construction process. Generally, dowel bars of 38 mm (1-1/4 in.) diameter (or larger) by 457 mm (18 in.) long will be used. Spacing for dowel bars is 305 mm (12 in.) as called for in WSDOT Standard Plan A-1.

5.3.1.2 Longitudinal Joints

The WSDOT Construction Manual provides insight into the timing of sawing of joints. As can be noted on I-405 (Bellevue) and I-90 (both west and east sides of Snoqualmie Pass), both the method of forming the longitudinal joint and when it is formed is important. Thin joint forming inserts should not be used. Further, if sawing is used, do it early enough to preclude the random cracking observed on PCCP as noted for I-405 and I-90. This process is even more critical in higher elevation areas which generally experience high thermal gradients from day to night.

5.3.1.3 Construction Joints

As noted in the WSDOT Construction Manual and Standard Plan A-1, all construction joints are doweled. This is necessary since construction joints have no aggregate interlock for load transfer. This occurs due to the use of transverse header boards to form the terminus of paving, i.e., the two faces of the joint are relatively smooth.

5.4 JOINT CONSTRUCTION

Recent recommendations provided in the FHWA TA 5040.30 [10.2] relative to PCCP joint construction include the following partial list.

(a) Securing Dowel Baskets: When using dowel baskets, the dowels must be properly aligned and the dowel basket firmly anchored to the base. The FHWA recommends that the dowel baskets be secured with steel stakes with a minimum diameter of 8 mm (0.3 in.) embedded at least 100 mm (4 in.) in stabilized bases, 150 mm (6 in.) in treated permeable bases and 250 mm (10 in.) for untreated bases or subgrade. Further, a minimum of 8 stakes per basket is recommended.

(b) Dowel Coating: All dowels must be epoxy coated. Further, dowels should be lightly coated with grease or other substance for their entire length to prevent bonding to the PCC. In the past, it was common to coat only one-half of each dowel bar but apparently this practice has resulted in some performance problems. The FHWA notes "...The dowel must be free to slide in the concrete so that the two pavement slabs move independently, thus preventing excessive pavement stresses. Only a thin coating should be used, as a thick coating may result in large voids in the concrete around the dowels."

(c) Joint Sawing

(i) Which Joints to Saw? The FHWA TA 5040.30 recommends that all joints be sawed (both transverse and longitudinal).

(ii) When to Saw? The FHWA notes "Time of initial sawing, both in the transverse and longitudinal directions, is critical in preventing uncontrolled shrinkage cracking. It is very important that sawing begin as soon as the concrete is strong enough to both support the sawing equipment and to prevent raveling during the sawing operation···This is particularly critical during hot weather. Once sawing begins, it should be a continuous operation and should only be stopped if raveling begins to occur." Further, the FHWA adds "Skip sawing is not recommended, as this practice results in a wide range of crack widths that form beneath the sawed joints." This last statement is in partial conflict with WSDOT Standard Specifications (1994) Paragraph 5-05.3(8)A (Contraction Joints) which states "Initial or control transverse contraction joints shall be sawed at intervals of 64 feet [20 m], or at an interval which will most effectively minimize the possibility of

uncontrolled cracking with a minimum number of initial contraction joints." WSDOT field observations have noted that the joints initially cut at 28 m (64 ft) (allowable in the 1991 WSDOT Standard Specifications) do have rather large crack openings but they "close" within the first year of service. The reason for this is analogous to the expansion joint discussion in SECTION 4.0, Paragraph 6.4.3.

(iii) How deep to saw?

- Transverse contraction joints: An initial sawcut of D/3 (where D = depth of slab) is recommended by the FHWA and particularly so for slab thicknesses greater than 250 mm (10 in.). In no case should the sawcut be less than D/4. The 1993 version of the *AASHTO Guide for Design of Pavement Structures* recommends D/4.

- Longitudinal joints: An minimum initial sawcut of D/3 is recommended by the FHWA and AASHTO.

(iv) Use of plastic inserts? The use of plastic inserts to form joints is <u>not recommended</u>.

6. CONSTRUCTION SURFACE SMOOTHNESS

6.1 INTRODUCTION

The 1994 WSDOT Standard Specifications contains two approaches toward achieving a relatively smooth pavement surface following construction. One approach uses a 3 m (10 ft) long straightedge and the other the California Profilograph. The basic requirements associated with each will be briefly described.

6.2 STRAIGHTEDGE

The 1994 WSDOT Standard Specifications require that a 3 m (10 ft) long straightedge be used to check surface smoothness for

- asphalt concrete pavement (5-04.3(13)), and
- small or irregular PCCP areas (5.05.3(12)).

For both pavement types, the completed wearing course

> "… shall not vary more than 1/8 in. [3.2 mm] from the lower edge of a 10 ft [3 m] straightedge placed on the surface parallel to the centerline. The transverse slope of the complete surface … shall not vary more than 1/4 in. [6.4 mm] in 10 ft [3 m] …"

If the above requirement is exceeded, the required corrective actions can vary depending on the pavement surface type.

6.3 CALIFORNIA PROFILOGRAPH

The California Profilograph is used by WSDOT to check the surface smoothness of PCCP projects for both new construction and rehabilitation (PCCP rehabilitation includes removing faults at joints). The profile is checked in accordance with WSDOT Test Method No. 807 ("Method of Operation of California Profilograph and Evaluation of Profiles"). The basic requirement is contained in Section 5-05.3(12) "Surface Smoothness" of the 1994 WSDOT Standard Specifications for Cement Concrete Pavement:

- A daily profile index will be computed on PCCP paving projects. If the lane is 3.6 m (12 ft) wide (or less), the daily profile index will be the average of two profiles approximately 0.9 m (3 ft) from and parallel to each pavement lane edge. The daily profile index of the finished pavement will be 110 mm per kilometer (7 in. per mile), or less.

- All PCCP pavement areas with "high point" deviations in excess of 7.6 mm (0.3 in.) "… shall be reduced by abrasive means until such deviations do not exceed 0.3 in. [7.6 mm] as determined by reruns of the profilograph."

For additional information on these specification requirements refer to SECTION 3.0 (Paragraph 3.2.2) of this Guide.

7. REHABILITATION AND MAINTENANCE ISSUES

7.1 GRANULAR OVERLAYS (CUSHION COURSES)

Granular overlays are an effective means to strengthen pavement sections. This overlay technique has been used primarily in Regions 2 and 6 on low to modest volume WSDOT routes.

A key element to good performance for this type of overlay is to avoid trapping water within the granular section. One source of water is from saturated subgrade/lower pavement layers (particularly so during the winter/spring in central and eastern Washington due to capillary action caused by freezing (winter) followed by melting (spring thaw)). An effective way to combat this problem is to place a BST preseal or Class D seal on the existing cracked pavement surface prior to placement of the granular overlay. The BST shot is intended to seal the existing cracks.

A second issue with regard to performance is the density of the granular layer. Normally, WSDOT CSTC or CSBC is used. South African experience [10.3] with granular overlays suggests that the highest possible density be obtained in the granular

layer (generally exceeds 100 percent modified AASHTO). The higher density results in a higher stiffness; hence, lower deflections and improved performance.

The WSDOT Standard Specifications require that each crushed surfacing layer be "compacted to at least 95 percent of the standard density determined by WSDOT test method No. 606 before the next succeeding layer of surfacing or pavement is placed." A desirable goal is to achieve 100 percent compaction.

7.2 OPEN GRADED WEARING COURSES

It is not advisable to overlay an existing Class D open graded with either a dense or open graded mix. The reason is that water will likely be trapped in the prior, open graded layer which could result in stripping.

A second problem which arises for Class D mixes is studded tire wear. These open graded mixes along with small maximum course aggregate size (essentially 9.5 mm (3/8 in.)) are very susceptible to accelerated wear in the wheelpaths. This kind of rutting is distinct from that caused by heavier vehicles in that the "rut" has "sharp" edges and, of course, fits the wheel spacing of typical automobiles. These kinds of ruts seem to affect some vehicles (such as pickup/campers) more than others. The bottom line is the question of where should such mixes be used and not used. Locations which are more susceptible to this type of wear include:

- high traffic (automobile) volumes (such as SR520 (Evergreen Point Bridge)),
- uphill grades (such as I-5 northbound near the Nisqually River),
- any area with a higher than normal use of studded tires.

It is not advisable to use WSDOT Class D's in urban areas or on routes with an ADT of 15,000 or greater.

7.3 FOG SEALS

An initial fog seal should be applied to WSDOT Class D mixes one year after construction. Thereafter, fog seal is required at least every 5 years for WSDOT Class D's. The fog seal helps to retain a binder film which is less susceptible to aging and traffic effects. This fog seal must be done if adequate performance is to be obtained from Class D mixes.

Typically, the fog seal is a CSS-1 emulsified asphalt diluted with water at a rate of one part water to one part emulsion. Application rates range from 0.2 to 0.5 ℓ/m^2 (0.05 to 0.10 gallon per square yard). Special Task Emulsion (STE) can be used (undiluted) as a fog seal in special cases where the amount of water to be evaporated from the fog seal needs to be minimized — such as cool weather conditions.

SECTION 10.0

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SECTION 11.0

SPECIAL PAVEMENT TYPES

This SECTION is used to describe pavement types not currently or commonly used by WSDOT, but are of interest for specific applications.

1. INTRODUCTION

This SECTION will contain information on those pavement types not commonly used by WSDOT but are of interest for specific applications. Only one special pavement type will be discussed for now—Concrete Block Pavement.

2. CONCRETE BLOCK PAVEMENT

2.1 INTRODUCTION

An overview report by Abate [11.1] provides a summary of some of the principal features of this pavement system. This pavement type has been used widely in Europe and is gaining use in the U.S. Concrete block pavement (CBP) applications have included

- seaports (container yards),
- industrial applications,
- airport aprons and taxiways, and
- roads and streets.

One of the potential applications is at intersections which are currently surfaced with AC and exhibit severe rutting and/or shoving.

A CBP is made up of precisely dimensioned individual concrete blocks which fit closely together to form a segment of pavement surface. A CBP surface course shares some similar performance characteristics with flexible pavement. Common names for the concrete blocks include pavers, paving blocks, and interlocking paving blocks. Paver sizes are a nominal 100 x 200 mm (4 x 8 in.) with thicknesses from 60 to 100 mm (2 1/2 to 4 in). They are usually laid manually but mechanical installation methods are also available. A 25 to 50 mm (1 to 2 in.) bedding sand layer is used under the pavers. They are set into the sand and then vibrated into place which forces some sand into the joints between the pavers. Jointing sand is then swept into the joints between the pavers and they are again vibrated to wedge the jointing sand into place.

Although the pavers are not bonded together with mortar, they are able to transfer loads sideways from one paver to the next. The friction of the sand in the joints provides an avenue for shear transfer between the individual blocks. However, this shear transfer will only be possible with narrow joints 1.5 to 3 mm (1/16 to 1/8 in.) wide. According to ASTM C936 (Standard Specification for Solid Interlocking Concrete Paving Units), paver length and width dimensions must be accurate to within 1.6 mm (1/16 in.). Pavers are manufactured according to ASTM C936 and must have an average compressive strength of 55 MPa (8,000 psi).

From the constructibility perspective, CBP is similar to other pavements with two exceptions. First, the sand bedding layer, which can be dumped and then screeded manually, or can be placed using a modified asphalt concrete spreader. Secondly the pavers are usually laid manually but can also be placed using various machines specifically designed for laying pavers. Although placement by either method is slow, completed and compacted sections can be put into use immediately.

From the design perspective, CBP presents difficulties with respect to modeling using analytical techniques such as layered elastic analysis. Conventional pavement materials (asphalt concrete, portland cement concrete, crushed stone) are not truly elastic, but they can be treated as such since their modulus of elasticity can be calculated fairly consistently. Although the elastic modulus of the concrete pavers and the elastic modulus of the sand can be measured individually, the composite system of pavers and sand in a CBP coupled with the gradual stiffening over time of this composite system precludes consistent measurement of an effective or equivalent modulus of elasticity. However, layered elastic pavement theory can be used to design block pavements if an effective modulus of elasticity for the composite system can be determined.

From the performance perspective, a CBP may be preferable to conventional pavements for some specific applications (e.g., ports and aircraft aprons). Measurement of a CBP's performance is similar to that of either asphaltic concrete (AC) or portland cement concrete (PCC). In addition, strict adherence to construction specifications and an experienced supervisor is important in achieving a successful CBP project.

From the cost perspective, installation of CBP varies greatly and depends on several factors: local labor cost, bedding sand thickness, paver size and shape, distance pavers must be shipped from the manufacturer, amount of cutting required, and the size of the pavement. In the U.S., pavers are usually more expensive than conventional pavements.

Refer to Figure 11.1 for a typical CBP cross-section.

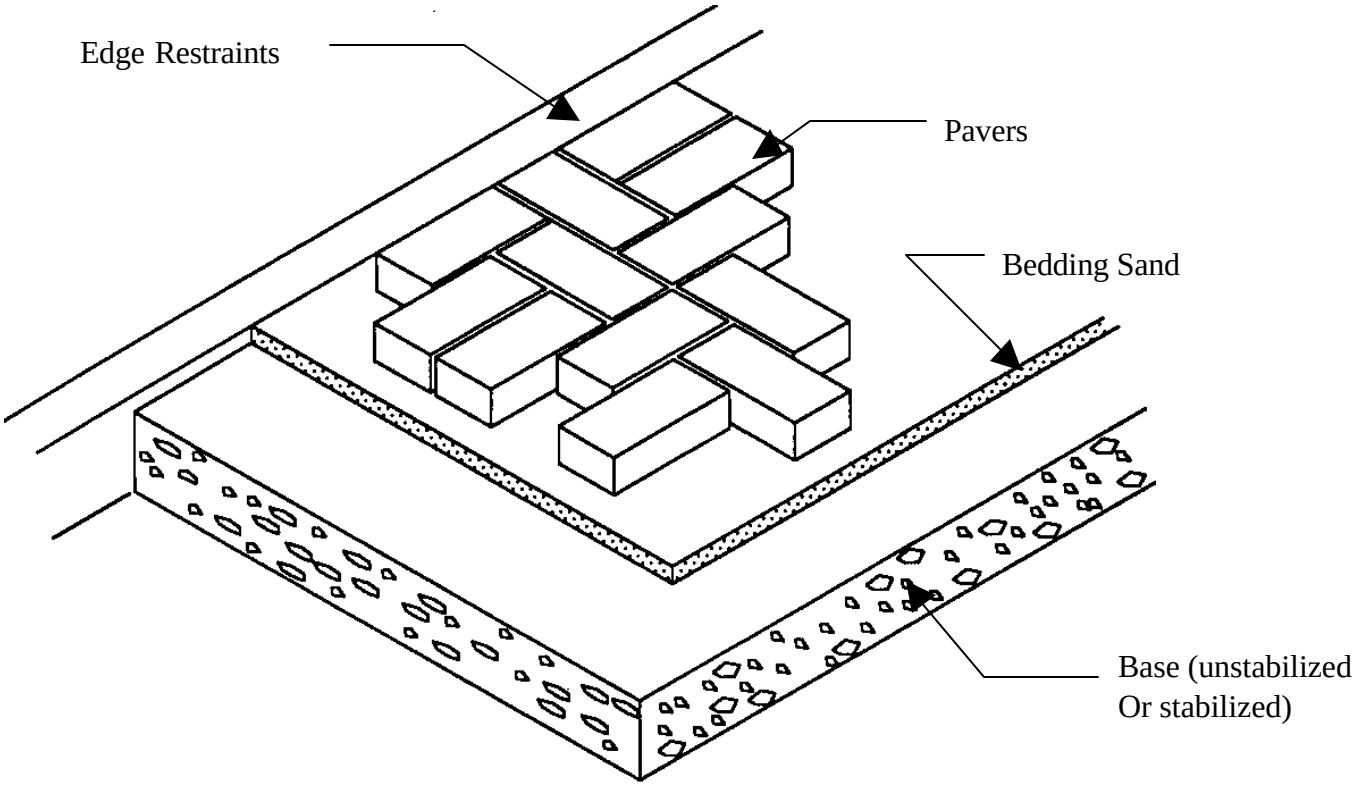

Figure 1.11. Typical components of a Concrete Block Pavement

The details of designing and constructing a CBP are important. For example, the following must be considered:

- edge restraints
- concrete block layout pattern
- bedding sand layer
  - thickness
  - gradation
- jointing sand and sealant
- base course
  - unstabilized
  - stabilized
- installation/construction

All of these items are critical for a successful CBP project. Each item is addressed in the Abate [1.1] overview report.

2.2 CBP DESIGN

As a result of interlock, and its segmented construction, most of the information summarized by Abate [11.1] suggests that a CBP behaves somewhat similar to a flexible pavement, at least with respect to rutting. However, there are differences between the two. A CBP will show an increase in permanent deformation early in its pavement life. As lock-up in the CBP develops, the elastic modulus increases and permanent deformations cease. On the other hand, a flexible pavement which ruts will have increasing permanent deformations with time.

CBP design methods can be divided into four categories:

- Design based on experience,
- Empirical designs based on full-scale trafficking tests,
- Modifications of existing design procedures for flexible pavements, and
- Mechanistic designs based on specific design parameters.

WSDOT has little design and construction experience with CBP as of June 1994. Since WSDOT has extensive experience with mechanistic-empirical design, such methods are of special interest. Conversely, a modification of the AASHTO flexible pavement design procedure was developed by Rada et al. [11.2] and may be easy to use and implement within WSDOT. With this design process (modified AASHTO that is), the paver/sand combination is treated as a composite layer for the purpose of obtaining a layer coefficient. Essentially, the remaining design requirements are very similar to designing a flexible pavement by the *AASHTO Guide* (i.e., use layer coefficients for all layers

except the subgrade; the subgrade is characterized by resilient modulus, traffic is characterized by ESALs).

WSDOT will, presumably, design and build this type of pavement. Only then will design, construction, and performance information be obtained which can be used to better understand the CBP system. Additionally, information from various conferences such as the recently held Concrete Block Paving Second International Workshop (Oslo, Norway, June 1994 [11.3]) will also be of significant help.

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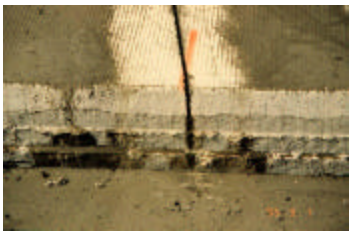
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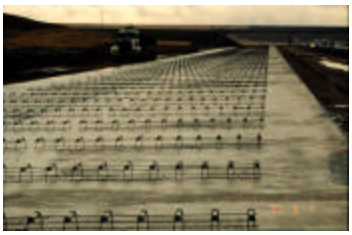
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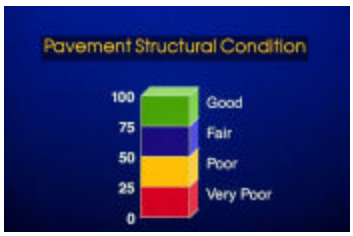

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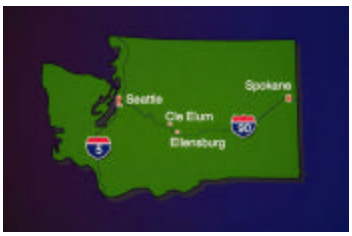

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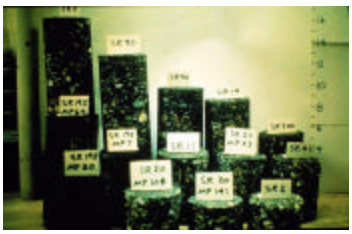
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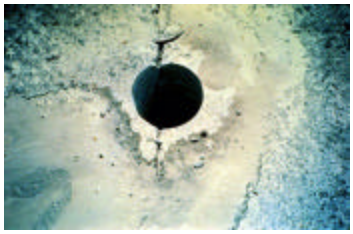
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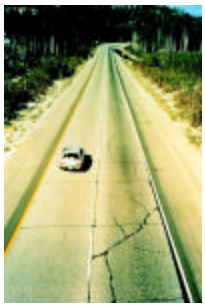

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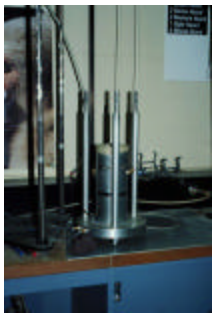
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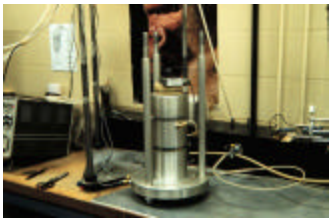
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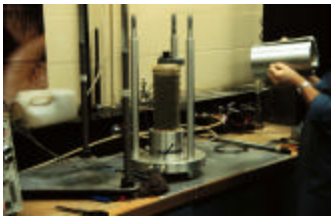
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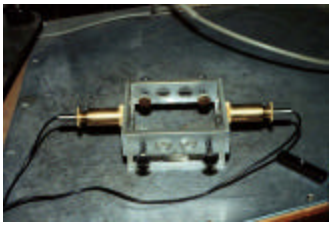
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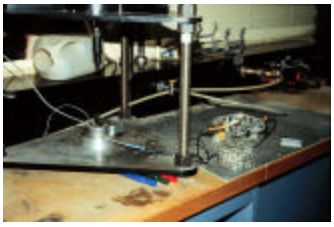
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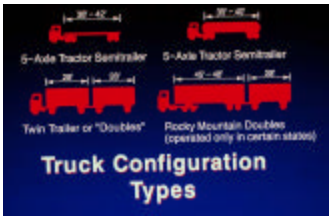

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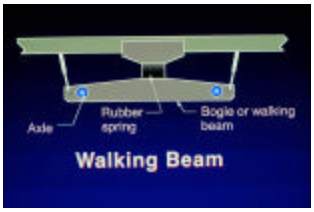

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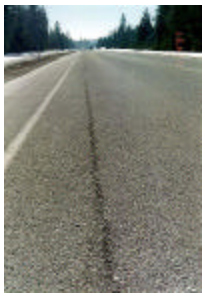
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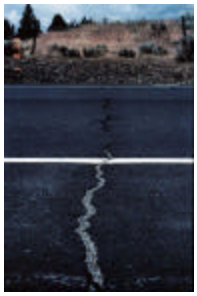
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