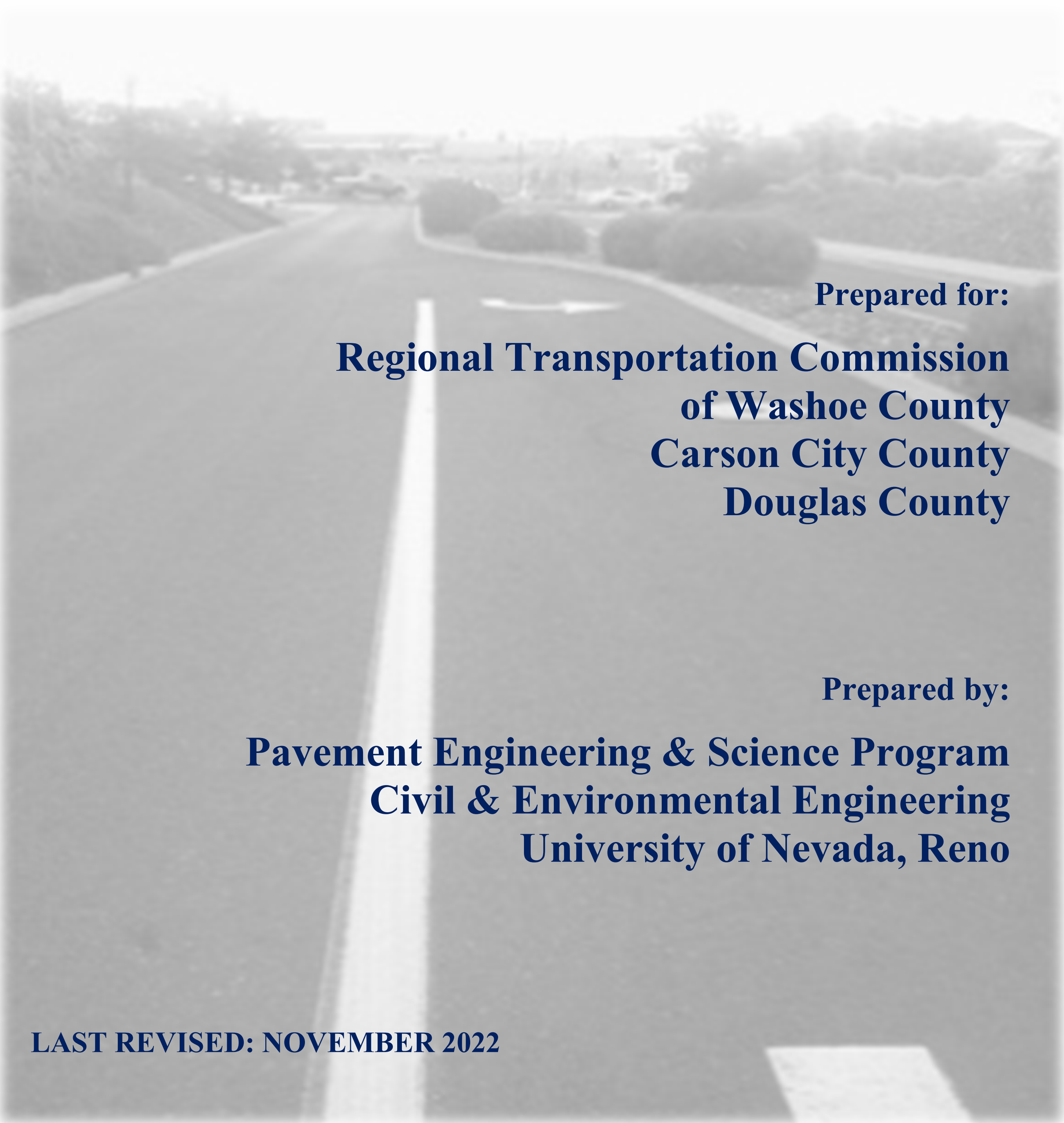
Structural Design Guide for Flexible Pavement



Prepared for:

Regional Transportation Commission of Washoe County
Carson City County
Douglas County

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PREFACE

Each year the Regional Transportation Commission of Washoe County (RTC) works with regional agencies on regional road projects throughout Washoe County. These include capacity projects, multimodal projects, pavement preservation projects, and other projects. A majority of regional road projects constructed in Washoe County utilize flexible pavement in the structural section. For the design of flexible pavement sections, the RTC requires the use of the 1993 edition of the American Association of State Highways and Transportation Officials (AASHTO) Guide for Design of Pavement Structure. Excerpts from the AASHTO Guide have been incorporated into this 2022 Structural Design Guide for Flexible Pavement Guide which has been developed by the Pavement Engineering & Science Program within the Civil & Environmental Engineering Department at the University of Nevada, Reno.

This Guide is intended to be used on all RTC administered projects within the regional road network of Washoe County. This Guide represents a major update from the 2007 Flexible Pavement Design Manual and presents an all-inclusive process for the structural design of flexible pavements within Nevada's North-Western Region.

This Guide covers the following critical aspects of the structural design of flexible pavements:

1. Definitions of distresses encountered in flexible pavements and in-depth discussions on their potential causes and recommended steps to prevent their occurrence.
2. Material type selection guide to support pavement engineers specifying, selecting, and approving material types and mix designs associated with pavement designs.
3. Characterization of materials and traffic loads, which play a critical role in the structural design of flexible pavements.
4. Selection and design of new and rehabilitation strategies along with examples.
5. Discussion of an effective pavement preservation program to ensure that the designed and constructed flexible pavement will perform at a high level of service throughout its design life.

This Guide provides consultants retained by the RTC a uniform and detailed procedure for designing flexible pavements using AASHTO design procedures. Other agencies in Washoe County may also benefit from the information presented.

11-01-2022

Dale Keller, P.E. Date

Regional Transportation Commission of Washoe County
Engineering Director

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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LIST OF ACRONYMS AND SYMBOLS

a_1	structural layer coefficient of AC layer
a_2	structural layer coefficient of base layer
a_3	structural layer coefficient of sub-base layer
a_1*	structural layer coefficient of existing AC layer
a_2*	structural layer coefficient of existing base layer
a_3*	structural layer coefficient of existing sub-base layer
a_{OL}	structural layer coefficient of AC overlay
$AADT_1$	annual average daily traffic for the first year
$AADTT_0$	initial two-way average annual daily truck traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACM	alternative contracting method
ALDF	axle load distribution factor
ATPB	asphalt treated permeable base
AVC	automatic vehicle classification
C_H	thickness correction term
CBM	cement bound material
CBR	California bearing ratio
CF	correction factor
CIR	cold in-place recycling
CTB	cement treated base
ΔDI	incremental damage index
$\Delta_{p(AC)}$	accumulated permanent or plastic vertical deformation in AC layer/sublayer
$\Delta_{p(soil)}$	permanent or plastic deformation for the unbound layer/sublayer
d_0	center FWD surface deflection
D	directional distribution factor
D_1	thickness of surface layer

D_2	thickness of base layer
D_3	thickness of sub-base layer
$D_{AC\text{-}mill}$	thickness of milled AC layer
D_{CIR}	depth of CIR layer
D_{OL}	thickness of AC overlay
DDF	direction distribution factor
DI_{Bottom}	cumulative damage index at the bottom of AC layer
DOT	Department of Transportation
E_{AC}	dynamic modulus of AC mix
E_p	effective modulus of existing pavement structure
ESAL	equivalent single axle load
FC_{Bottom}	area of alligator cracking that initiates at the bottom of AC layer, percent of total lane area
FHWA	Federal Highway Administration
FWD	falling weight deflectometer
JMF	job mix formula
G	growth factor
H_{AC}	total AC layer thickness
$h_{(AC)}$	thickness of the AC layer/sublayer
h_{soil}	thickness of the unbound layer/sublayer
k_z	depth confinement factor
L	design lane distribution factor
LCCA	life-cycle cost analysis
LDF	lane distribution factor
LEF	load equivalency factor
LTPP	Long-Term Pavement Performance
m	drainage coefficient
m_2	drainage coefficient of base layer
m_3	drainage coefficient of sub-base layer

M-E	mechanistic-empirical
Mr	resilient modulus
$Mr_{effective}$	effective M_r of the subgrade layer
MEPDG	Mechanistic-Empirical Pavement Design Guide
MAF	monthly/seasonal adjustment factor
n	number of axle load applications
$N_{1.5}$	total ESAL to a P_t of 1.5
$N_{f\text{-}AC}$	allowable number of axle load applications to AC fatigue cracking
N_p	total Applied ESAL to Date
N_T	total number of trucks
NDOT	Nevada DOT
NDT	non-destructive testing
NMAS	nominal maximum aggregate size
No.	number
OGFC	open-graded friction course
P	FWD load
P40	percent passing sieve No. 40 (0.425 mm)
P_i	initial serviceability
P_t	terminal serviceability
PES	Pavement Engineering & Science
PCC	Portland cement concrete
PG	performance grade
PMS	pavement management system
PSI	present serviceability index
r	growth rate
R	reliability factor
R-value	resistance value
RAP	reclaimed asphalt pavement

RL	remaining Life
S_0	overall standard deviation
SN	structural number
SN_{eff}	effective structural number of existing pavement
SN_f	structural number for design future traffic
SN_{OL}	structural number of the AC overlay
T	percent of trucks in the traffic volume
T_f	truck factor
U_r	seasonal pavement relative damage
$U_{r(average)}$	average pavement relative damage
UCS	unconfined compressive strength
US	United States
VCD	vehicle class distribution
V_a	percent air voids in asphalt mixture
V_{be}	effective asphalt content by volume
vpd	vehicle per day
W_{18}	design lane traffic
W_c	water content
WIM	weigh-in-motion
Y	design life
ε_0	intercept determined from laboratory repeated load permanent deformation tests
$\varepsilon_{p(AC)}$	accumulated permanent or plastic axial strain in the AC layer/sublayer
ε_r	resilient strain imposed in laboratory test
$\varepsilon_{r(AC)}$	resilient or elastic strain calculated by the structural response model at the mid-depth of each AC sublayer
ε_t	tensile strain
ε_v	vertical compressive strain
γ	shear strain
γ_d	maximum dry density

CHAPTER 1. INTRODUCTION TO FLEXIBLE PAVEMENT ENGINEERING

The overall objective of a road pavement is to provide a durable, comfortable, and safe traveling surface for private and commercial public users under all environmental conditions. A well-designed and constructed pavement structure is expected to provide these structural and functional characteristics over its design life with minimal need for maintenance activities.

Pavements are classified according to the type of materials used to construct the driving surface. Pavements with an asphalt concrete (AC) driving surface are classified as flexible while pavements with a Portland cement concrete (PCC) driving surface are classified as rigid. Since this Guide is specific to the design of AC pavements, only flexible pavements will be presented.

The term "flexible" indicates the pavement structure carries the loads through the bending of the AC surface layer, which tends to distribute the load over a relatively small area. Therefore, a flexible pavement is designed as a multi-layer system with the objective of reducing the magnitude of load-generated stresses to a level well below the strength of each layer. Figure 1 shows a typical structure of a flexible pavement including all layers above the natural soil (i.e., Subgrade). It should be noted that the Sub-base layer may be omitted depending on the specific design requirements and the availability of suitable materials.

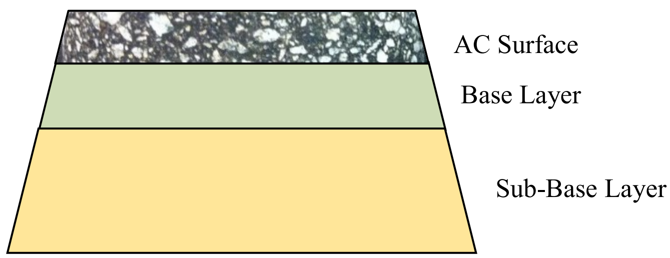

Figure 1. Typical flexible pavement structure.

The role of each layer in the flexible pavement structure is to reduce the stresses to a level that can be accommodated by the layer below as illustrated in figure 2.

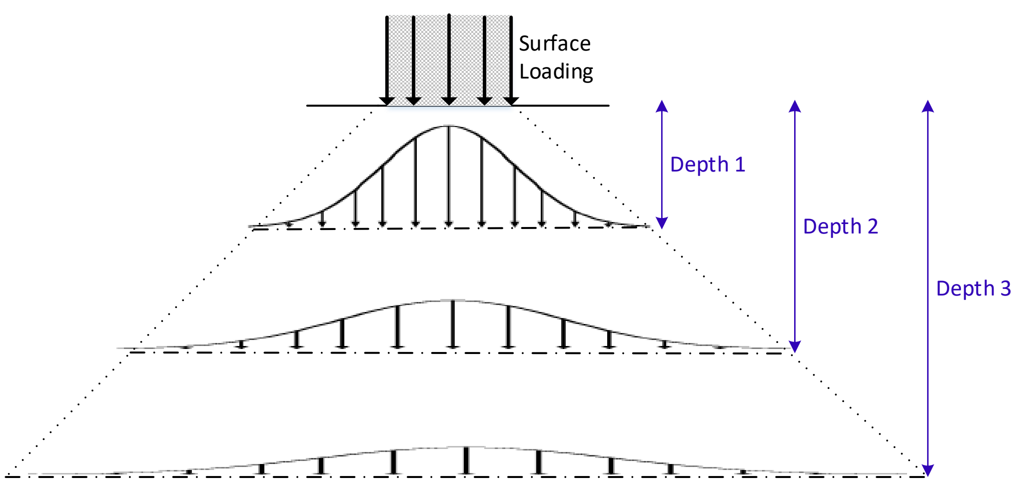

Figure 2. Relative magnitude of load-induced vertical stresses at various depths.

Since the highest stresses are encountered at the tire-pavement interface, the structural strength requirement of a layer decreases as its distance from the surface increases. The layers in a flexible pavement structure are defined below:

- **Surface layer**: The surface layer is constructed with AC mixtures and is typically composed of the following courses:
    - The AC surface course (or wearing course) is the driving surface designed to resist rutting caused by the high vertical and shear stresses at the tire-pavement interface and thermal cracking caused by low temperature stresses with excellent durability and skid resistance.
    - The AC base course (or intermediate course) is the lower part of the surface layer that is designed to resist rutting and fatigue cracking with excellent durability.

- **Base layer:** The base layer of a flexible pavement is a critical structural component that can be constructed with either unbound granular material or cement/asphalt bound material. Both types of the base layer must resist rutting as well as provide protection to the sub-base and subgrade layers.

- **Sub-base layer:** The sub-base layer of a flexible pavement is a structural layer with a lower structural value than the aggregate base layer. The sub-base layer is constructed with unbound granular material that must resist rutting and protect the subgrade layer. The gradation of the unbound granular materials is selected to possess the required drainage capacity for the structure for given site and environmental conditions. It also acts as a filter to prevent migration of fine particles from the subgrade into the base.

- **Subgrade layer:** The subgrade layer represents the top three feet of the natural soil. The material for the subgrade layer consists of local materials that may be processed to reduce its plastic behavior (e.g., lime stabilization, cement stabilization, etc.).

As shown in figure 2, none of the individual layers in the flexible pavement structure is stiff enough to carry the load-induced stresses by itself. The various layers interact within the flexible system to collectively reduce the stresses to the magnitude that can be carried by the materials at the respective level within the structure without causing a structural failure. A rigid pavement structure has the opposite behavior, where the PCC slab is extremely rigid relative to the other layers in the system and carries the majority of the load-induced stresses within its designed thickness.

There are approximately 4.5 million miles of paved roads in the United States (US) with over 90% of them constructed with flexible pavements. This proportion does not change drastically when applied to road networks world-wide. Specifically, flexible pavements are very popular on local streets and parking lots due to their significantly lower initial cost. One unique aspect of flexible pavements is the need for well-scheduled preservation activities in order to maintain performance at a high level throughout the design life.

This Guide presents an all-inclusive process for the structural design of flexible pavements within Nevada's North-Western Region. The Guide covers the following critical aspects of the structural design of flexible pavements:

- Definitions of distresses encountered on flexible pavements and in-depth discussions on their potential causes and recommended steps to prevent their occurrence. It is believed that a good understanding of the causes and remedies of the distresses observed on a specific flexible pavement section would lead to the selection of the appropriate treatment and the specification of the most durable materials and mixtures. For instance, during treatment selection the pavement engineer should either take steps to mitigate reflective cracking in mill and fill treatments; or take into account how well the treatment will remove the existing distresses and how the unaddressed distresses (such as full-depth cracking) will impact the in-service life of the treatment (e.g., reflective cracking).
- Guidelines for specifying, selecting, and approving material types and mixture designs associated with pavement designs. Selection of the most appropriate material types for unique project conditions is critical to achieving good long-term pavement performance. This includes asphalt concrete mixtures, base courses, and in-place recycled materials with due considerations given to roadway functional classification, location within pavement structure, and lift thicknesses.
- Characterization of traffic loads, which play a critical role in the structural design of flexible pavements. The load-induced stresses generated at all levels within the flexible pavement structure are dominated by traffic characteristics. To generate a successful structural design, it should be well recognized that the flexible pavement shall carry the imposed traffic loads under all climatic conditions. Therefore, the pavement engineer should think of traffic loads as his/her customer where a high level of service is expected under all conditions.
- Guidelines for designing flexible pavement alternatives. Conducting the design of the selected appropriate treatment(s) is done after establishing a good understanding of the potentially encountered distresses and the level of traffic loads to be served by the flexible pavement. In the case of a new road, the only logical treatment is a new design. For an existing road, the appropriate treatment depends on the encountered distresses and may include: overlay over the existing pavement, coldmill and overlay, overlay over cold in-place recycled portion (full or partial) of the existing AC layer, or overlay over a roadbed modified layer. The properties of the existing pavement layers are obtained through a non-destructive testing program and/or a geotechnical investigation.
- Definition of a pavement preservation program and associated treatments. The final step of the all-inclusive pavement design process is to implement an effective pavement preservation program to ensure that the designed-constructed flexible pavement will perform at a high level of service throughout its design life. The responsibility of the pavement engineer continues beyond the structural design through the selection of effective treatments and the appropriate time for their application. Once again, a proper understanding of the distresses and their potential remedies will play a major role in preserving a flexible pavement with a high level of service.

Project bundling by combining multiple street maintenance or rehabilitation projects into a single contract award is an effective alternative contracting method (ACM) that is typically used to minimize bid prices and maximize competition.[1] Such ACM can accelerate project delivery and consolidate scope, while providing flexible delivery scheduling leading to shortened delivery times and reduced cost.

[1]Von Quintus, H., and Nabizadeh, H. (2020). "Alternative Contracting Methods for Pavement Preservation Projects," Whitepaper, a deliverable to U.S. Department of Transportation, Federal Highway Administration, Delivered by UNR.

CHAPTER 2. PERFORMANCE OF FLEXIBLE PAVEMENTS

As soon as the construction of the flexible pavement is completed, it is subjected to traffic loads and environmental factors. The response of the pavement to the combined actions of traffic and environment controls its long-term performance that is divided into two major categories: functional and structural. A successful flexible pavement must maintain a high level of performance in both categories over its design life. The two performance categories are highly inter-related where the performance in one category significantly influences its performance in the other.

2.1 Functional Performance

The functional performance of a flexible pavement is measured in terms of its ability to resist the following distresses: roughness, loss of friction, and noise. These distresses are the most noticeable by the traveling public since they are directly related to their comfort and safety. The following represents brief descriptions of the three functional distresses, potential causes, and recommended remedies. It should be noted that the recommended remedies are effective only in cases where structural failures such as rutting, shoving, and cracking are not present.

2.1.a Roughness

Roughness is defined as the deviation of the pavement surface from a true planar surface in the longitudinal direction. Roughness can be generated by two causes: a) construction, or b) combination of multiple structural distresses. A flexible pavement that exhibits roughness as the only distress mode with no other structural distresses, was possibly built rough and may be rehabilitated through milling of the top 1–2 inches of the surface and replacing it with new materials. A flexible pavement exhibiting roughness with other associated distresses should be investigated and the appropriate treatment selected based on the extent and severity of the various distresses. Roughness is further discussed under Section 2.2.c on combination distresses.

2.1.b Loss of Friction

Loss of friction is defined as the inability of the pavement to provide sufficient skid resistance under the braking action of traffic. Loss of friction can be caused by two factors; a) polished aggregate or b) bleeding of the AC mix. A flexible pavement that exhibits loss of friction due to polishing can be rehabilitated by milling the top 1–2 inch of the polished surface and replacing it with new material. In some instances, a surface treatment (e.g., slurry or chip seal) can be applied to restore the skid resistance of the surface. A flexible pavement exhibiting loss of friction due to bleeding should be investigated to identify the cause and select the appropriate treatment as presented in Section 2.2.

2.1.c Noise

A noisy flexible pavement can be very annoying to the traveling public and neighborhoods near the road. Noise can be generated by the interaction between the tire rubber and pavement surface, especially at high travel speed. One of the effective methods to remedy a noisy flexible pavement is to apply a noise absorbing thin layer such as an open-graded friction course or a stone-matrix asphalt mix. Significant reductions in traffic noise have also been measured with tire-rubber modified gap-graded AC mixes.

2.2 Structural Performance

The structural performance of a flexible pavement is measured in terms of its ability to resist a series of load-related and environment-related distresses. Figure 3 presents the various strains generated throughout the pavement structure as it is subjected to the traveling traffic loads. The shear strain (γ) is related to rutting and shoving within the top 2–3 inches of the AC layer. The vertical compressive strain (ε_V) is related to rutting throughout the various layers as well as the bleeding of the AC mix. The tensile strain (ε_t) at the bottom of the AC layer is related to the fatigue cracking of the surface layer.

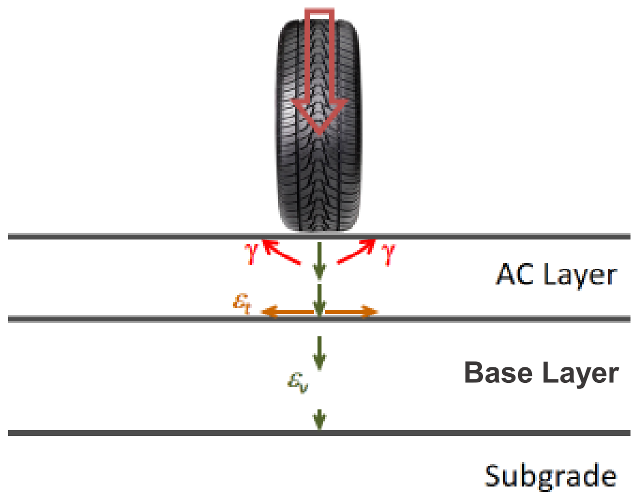

Figure 3. Responses of flexible pavement under traffic load.

In addition to the responses shown in figure 3, the flexible pavement also responds to the environment in terms of shrinking of the AC layer due to temperature drop and temperature cycling within each 24-hour period.

Figure 4 groups the distresses experienced by flexible pavements under the action of traffic loads and the environment. Reflective cracking is listed under both the load and non-load related groups since it could occur with and without the application of traffic loads.

The following sections present the most common causes, approaches to prevent, and practical methods of repairing each of the identified distresses. It is believed that with a good understanding of these three phases of each distress, the pavement engineer will be able to develop a design to prevent the distresses from re-occurring or to repair the distress after it occurred. It should be well recognized that some of the prevention techniques cannot be handled by the structural design of the flexible pavement and must be implemented at the material selection and mix design stage.

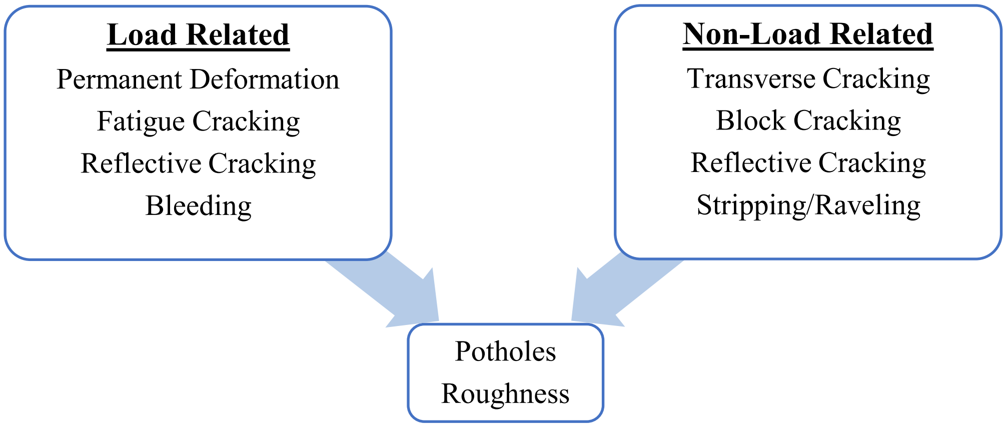

Figure 4. Major groups of common flexible pavement distresses.

2.2.a Load Related Distresses

Load related distresses are defined as distresses caused or accelerated by traffic loads. This type of distress typically occurs in the wheel-paths; however, it may extend outside the wheel-paths as it progresses further into higher levels of severity.

Permanent Deformation

Permanent deformation in flexible pavements can occur in two different modes: a) shoving, or b) rutting. Both modes have the following common characteristics: 1) driven by traffic loads, 2) occur in hot climate, and 3) occur during the early part of pavement life (i.e., less than 5 years).

Shoving: Figure 5 shows a typical shoving failure of a flexible pavement. It consists of horizontal non-uniform permanent distortions of the top 2–3 inches of the AC layer caused by the braking action of heavy vehicles commonly observed at intersections. The most severe type of shoving is typically encountered at the end of off-ramps where heavy vehicles exert high braking forces in order to reduce the speed from 60 mph to a stop in a relatively short distance.

- Prevention: shoving can only be prevented at the materials selection and mix design stage. The structural section of the pavement has minimal contribution towards its resistance to shoving. The following steps can be implemented to prevent shoving failures:
    - Use a higher performance grade (PG) asphalt binder (i.e., grade bumping) that is applicable for slow/stopping traffic. For example, a project requiring a PG 64-28 binder for standard highway traffic may benefit from specifying a PG 70-28 asphalt binder at the intersection. However, the grade bumping for slow/stopping traffic may not be practical or economical when the stiffer PG is not locally available or the asphalt tonnage is limited.
    - Increase the shear resistance of the AC mix through the following:
        - High quality aggregates: low abrasion and high sand equivalent.
        - 100% crushed aggregates.
        - Rut-resistant aggregate gradation.
        - Prevent moisture damage to the AC mix.
- Method of Repair: the only practical method of repair for shoving is to mill the top 2–3 inches of the AC layer and replace with high quality AC mix with the characteristics listed in the prevention section.

Figure 5. Typical shoving of flexible pavements.

Rutting: the source of rutting of a flexible pavement can be located within any of its layers, i.e., surface, base, sub-base, and/or subgrade. It consists of permanent depressions in the wheel-paths along the direction of travel. Typically, the narrower the rut the shallower the source, for example, the narrow ruts shown in figure 6 are caused by failure within the AC layer. While the wide rut shown in figure 7 is caused by failure in any of the supporting layers. Therefore, it is very common to classify rutting of flexible pavements in two categories: a) AC rutting and b) Total rutting. The methods of prevention and repair will be discussed under each category.

Figure 6. Typical narrow ruts formed in AC layer.

Figure 7. Typical wide ruts formed in the base, sub-base, or subgrade layers.

- Prevention of AC Rutting: the major causes of rutting in the AC layer are low shear strength and high in-place air voids of the AC mix. The structural section of the pavement has minimal contribution towards its resistance to rutting in the AC layer. The following steps can be implemented to prevent rutting in the AC layer.
    - Use the appropriate high temperature PG for the asphalt binder. It has been determined that polymer and terminal blend rubber modified asphalt binders with a high PG of 64°C are appropriate for North-Western Nevada.
    - Increase the shear resistance of the AC mix through the following:
        - High quality aggregates: low abrasion and high sand equivalent.
        - 100% crushed aggregates.
        - Rut-resistant aggregate gradation.
        - High resistance to moisture damage.
- Method of Repair of AC Rutting: there are two practical methods of repair for rutting in the AC layer.
    - Mill the top 2–3 inches of the AC layer and replace with high quality AC mix with the characteristics listed in the prevention section.
    - Fill the ruts and apply a structural AC overlay to accommodate the estimated future design traffic.
    - For pavements with rut depth less than or equal to 0.50 inch, micro-surfacing can be applied. However, this action will be a short-term fix of 3–5 years.
- Prevention of Total Rutting: the major cause of total rutting is low strength in any or all of the base, sub-base, or subgrade layers. The properties of granular materials with most significant impact on strength are moisture content and percent of passing No. 200 sieve. The structural section of the pavement has significant contribution towards its resistance to total rutting. The following steps can be implemented to prevent total rutting in flexible pavement.
    - Use the appropriate high temperature PG for the asphalt binder. It has been determined that polymer and terminal blend rubber modified asphalt binders with a high PG of 64°C are appropriate for North-Western Nevada.
    - Increase the strength of the base, sub-base, and subgrade through the following:
        - Clean gradation for base materials with adequate particle shape and low plasticity.
        - Good drainage to keep moisture out the structure.
        - High resistance to moisture damage.
        - Compaction at optimum moisture content and density.
- Method of Repair of Total Rutting: the only effective method of repair for total rutting is to identify the in-situ properties of the various pavement layers and design a structural AC overlay to accommodate estimated future design traffic. Otherwise, a full depth removal and replacement is needed.

<u>Fatigue Cracking</u>

Even-though the origin of fatigue cracking of a flexible pavement is the AC layer, the properties of the entire structure significantly contribute to this type of failure. As traffic loads are applied, the AC layer bends and generates tensile strains at its bottom face. After numerous repetitions of traffic loads, the AC mix cracks due to fatigue. The fatigue crack initiates at the bottom face of the

AC layer and quickly propagates to the surface as a single longitudinal (or transverse crack) in the wheel-path as shown in the left side of figure 8. Additional traffic loading, further deteriorates the AC mix and accelerates the formation of additional cracks as shown in the right side of figure 8. Cracking allows water intrusion that will accelerate the deterioration of the underlying pavement structure.

Figure 8. Typical fatigue cracking of flexible pavements.

- Prevention of Fatigue Cracking: several factors contribute to the fatigue cracking of flexible pavements, including: inadequate structural design, brittle AC mix, moisture damaged AC mix, low binder content in the AC mix, and inappropriate asphalt binder grade. The following represents methods to prevent early or excessive fatigue cracking of flexible pavements at the end of their design life.
  - Ensure adequate structural design for the anticipated design traffic.
  - Properly apply and use an adequate amount of tack coat between AC layers/lifts to promote bonding.
  - Select an asphalt binder with appropriate PG and minimal long-term aging potential.
  - Prevent moisture damage of the AC mix.
  - Ensure adequate asphalt binder content in the mix.
  - Polymer and terminal blend rubber modified asphalt binders have shown significant improvement in the AC mix resistance to fatigue cracking.
- Method of Repair of Fatigue Cracking: unfortunately, fatigue cracking is the most difficult distress to repair. The occurrence of fatigue cracking indicates a serious structural problem with the flexible pavement that must be extensively investigated prior to identifying an effective method of repair. Because of inadequate structural support, any existing fatigue cracking can propagate through an overlay very quickly if no remedial action is taken. It should be noted that sealing fatigue cracks prior to overlay is not considered an effective remedial action. The following steps are recommended in order to identify the effective method of repair.
  - Conduct an in-depth investigation to identify the source of the low structural support:
    - Weak subgrade.
    - Weak base.
    - Weak or brittle AC layer or de-bonding of AC layers.
  - If the low structural support is in the subgrade, full reconstruction will be needed.

- If the low structural support in the base, the following options must be evaluated:
    - Option 1: Roadbed modification (RBM) of the AC layer and aggregate base/sub-base course followed by a new AC layer.
    - Option 2: Removal and replacement of AC and base layers.
- If the low structural support is in the AC layer, the following options must be evaluated:
    - Option 1: Removal and replacement of the AC layer.
    - Option 2: Cold in-place recycling (CIR) of the AC layer followed by an AC overlay.
    - Option 3: Application of an ***effective*** stress relief course followed by an AC overlay. The stress relief course mixture should be designed to be highly flexible while staying stable under construction and in-service traffic loadings. Furthermore, the stress relief course layer is not anticipated to stop cracking from reflecting to the surface and should not be considered in the design of the required structural capacity of the AC overlay.
- An economic analysis should be conducted to select the best option.

Bleeding

The origin of bleeding of a flexible pavement is the AC mix in the top course. If the asphalt binder in the AC mix cannot properly resist the high stresses generated by heavy traffic under hot weather, the binder will flow to the surface. Figure 9 shows two levels of bleeding in flexible pavements; the left side of the figure shows a mid-stage bleeding while the right side shows a very severe level of bleeding. Bleeding of the asphalt binder to the surface of a flexible pavement significantly reduces its skid resistance. While both levels shown in figure 9 are considered safety issues, the severe level creates an extremely dangerous driving condition and should be immediately repaired.

Figure 9. Medium and severe bleeding failures of flexible pavements.

- Prevention of Bleeding: several factors contributes to the bleeding of the top AC layer, including; soft asphalt binder, high asphalt binder content, moisture damage, extremely heavy traffic, and extremely hot weather. Typically, it takes the combination of two or more of these factors in order to initiate a bleeding failure. Even though traffic plays a major role in bleeding, the structural design of the flexible pavement does not control this type of failure. The following represents methods to prevent bleeding of flexible pavements.

- Select the appropriate high temperature PG for the asphalt binder while taking into consideration traffic conditions in terms of stop-and-go and braking.
- Avoid placing an AC mix with high asphalt binder contents within the top 2–3 inches without jeopardizing the mix's resistance to moisture damage and thermal cracking.
- Ensure good resistance of the AC mix to moisture damage.

- Method of Repair of Bleeding: there are several practical alternatives to repair a bleeding flexible pavement as listed below.
    - Apply a thin AC overlay, 1–1.5 inch.
    - Mill and fill the top 1–2 inches of the AC layer.
    - Apply a surface treatment, i.e., chip seal, slurry seal, or micro-surfacing as a temporary fix for the loss of skid resistance.

Reflective Cracking

Reflective cracking is a common distress mode of AC overlays over old AC pavements exhibiting any type of surface cracks, i.e., fatigue, transverse, or block. The intersection of the new AC overlay with the existing cracks represents a point of high stress concentration. As stated earlier, there are two causes of reflective cracking: a) load related, and b) non-load related. As traffic loads are applied, the AC overlay experiences tensile strains and un-even vertical movements across the existing cracks generating high shear stresses within the AC mix and resulting in the formation of a crack mimicking the existing crack. Without the application of traffic loads, both the AC overlay and the existing AC layer experience contraction and expansion movements due to temperature variations. These movements cause high tensile stresses at the interface of the two layers which results in the formation of a crack mimicking the existing crack. Figure 10 presents the two mechanisms of reflective cracking and a typical AC overlay over an existing AC pavement experiencing transverse cracking.

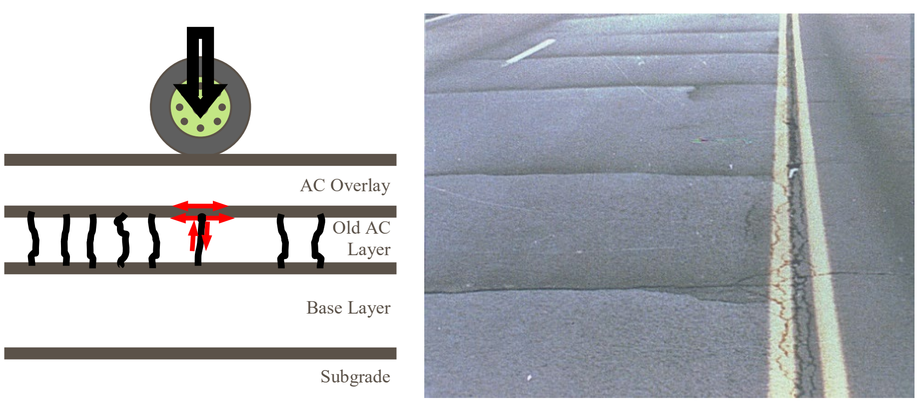

Figure 10. Mechanisms and typical reflective cracking in AC overlay.

- Prevention of Reflective Cracking: as stated earlier, reflective cracking can occur with and without the application of traffic loads. In general, traffic loads accelerate the generation of reflective cracking. The following represents methods to prevent reflective cracking of AC overlays. An economic analysis should be conducted to select the best option.
    - Treat the existing cracked AC surface in order to reduce the stress concentration at the point of intersection with the new AC overlay through any of the following options:
        - Option 1: Remove the existing cracked AC layer.

        - Option 2: Cold in-place recycle a portion of the existing cracked AC layer.
        - Option 3: Apply an ***effective*** stress relief course. The stress relief course mixture should be designed to be highly flexible while staying stable under construction and in-service traffic loadings. Furthermore, the stress relief course layer is not anticipated to stop cracking from reflecting to the surface and should not be considered in the design of the required structural capacity of the AC overlay.
    - Ensure adequate asphalt binder content in the mix.
    - Polymer and terminal blend rubber modified asphalt binders have shown significant improvement in the AC mix resistance to reflective cracking.
- Method of Repair of Reflective Cracking: unfortunately, reflective cracking is a difficult distress to repair. It should be noted that sealing the reflected cracks prior to overlay is not considered an effective remedial action. Once the reflective cracks appear on the surface of the AC overlay, the flexible pavement will be considered as a cracked pavement and must be rehabilitated with any of the following options. An economic analysis should be conducted to select the best option.
    - Option 1: Removal and replacement of the AC layer.
    - Option 2: Cold in-place recycling of the AC layer followed by an AC overlay.
    - Option 3: Application of an ***effective*** stress relief course followed by an AC overlay. The stress relief course mixture should be designed to be highly flexible while staying stable under construction and in-service traffic loadings. Furthermore, the stress relief course layer is not anticipated to stop cracking from reflecting to the surface and should not be considered in the design of the required structural capacity of the AC overlay.

2.2.b Non-Load Related Distresses

Non-load related distresses are defined as distresses caused or accelerated by the environment. This type of distresses typically occurs over the entire pavement surface.

Transverse Cracking

As the AC layer experiences contraction due to drop in air temperature, tensile stresses are generated within the AC mix. The magnitude of the thermal tensile stresses is directly related to the stiffness of the AC mix. Thus, an asphalt binder with low aging susceptibility can effectively reduce the magnitude of thermal stresses. Once the thermally generated tensile stresses exceed the tensile strength of the AC mix, a transverse crack is formed across the pavement surface as shown in figure 11 (cracks are perpendicular to the pavement's centerline or laydown direction). As the air temperature rises again, the AC mix does not expand and recover the contraction movement leading to excessive widening in the transverse crack as shown in the right side of figure 11.

- Prevention of Transverse Cracking: the main cause of transverse cracking is the generation of tensile thermal stresses due to the contraction of the AC layer. The following represents methods to prevent transverse cracking of flexible pavements.
    - Select an asphalt binder with appropriate low PG and reduced susceptibility to long-term aging.
    - Prevent moisture damage of the AC mix.
    - Ensure adequate asphalt binder content in the AC mix.
    - Use polymer or terminal blend rubber modified asphalt binders that have shown significant improvement in the AC mix resistance to transverse cracking.

Figure 11. Typical transverse cracks in flexible pavements in northern Nevada.

- Method of Repair of Transverse Cracking: unfortunately, transverse cracking is a difficult distress to repair. The following options may be used. An economic analysis should be conducted to select the best option
  - Option 1: Sealing of the cracks.
  - Option 2: Removal and replacement of the AC layer.
  - Option 3: Cold in-place recycling of the AC layer followed by an AC overlay.
  - Option 4: Application of an ***effective*** stress relief course followed by an AC overlay. The stress relief course mixture should be designed to be highly flexible while staying stable under construction and in-service traffic loadings. Furthermore, the stress relief course layer is not anticipated to stop cracking from reflecting to the surface and should not be considered in the design of the required structural capacity of the AC overlay.

Block Cracking

As the AC layer experiences contraction through the daily cycling of air temperature, tensile stresses are generated within the AC mix. The magnitude of the tensile stresses is directly related to the stiffness of the AC mix. Thus, an asphalt binder with low aging susceptibility can effectively reduce the magnitude of the stresses. Once the generated tensile stresses exceed the tensile strength of the AC mix, random cracks are formed over the pavement surface as shown in figure 12. As the air temperature cycles, the AC mix does not expand and recover the contraction movement leading to excessive widening in the random cracks as shown in the right side of figure 12.

Figure 12. Typical block cracks in flexible pavements in northern Nevada.

- Prevention of Block Cracking: the main cause of block cracking is the generation of tensile stresses due to the contraction of the AC layer and excessive aging of the asphalt binder. The following represents methods to prevent block cracking of flexible pavements.
  - Select an asphalt binder with appropriate PG and reduced susceptibility to long-term aging.
  - Seal the pavement surface within 3–5 years after construction in order to slow down the aging of the asphalt binder.
  - Prevent moisture damage of the AC mix.
  - Target lower design air voids (e.g., design air voids of 3% instead of 4%) or reduce the laboratory compaction effort (e.g., Marshall blows of 50 instead of 75) to allow more asphalt binder in the AC mix.
  - Increase the in-place density of the AC layer.
  - Use polymer or terminal blend rubber modified asphalt binders that have shown significant improvement in the AC mix resistance to block cracking.
- Method of Repair of Block Cracking: unfortunately, block cracking is a difficult distress to repair. The following options may be used. An economic analysis should be conducted to select the best option.
  - Option 1: Sealing of the cracks if the extent is less than 50 percent.
  - Option 2: Removal and replacement of the AC layer.
  - Option 3: Full or partial cold in-place recycling of the AC layer followed by an AC overlay.
  - Option 4: Application of an ***effective*** stress relief course followed by an AC overlay. The stress relief course mixture should be designed to be highly flexible while staying stable under construction and in-service traffic loadings. Furthermore, the stress relief course layer is not anticipated to stop cracking from reflecting to the surface and should not be considered in the design of the required structural capacity of the AC overlay.

Raveling

As the AC mix experiences moisture damage through the weakening of the bond at the aggregate–asphalt binder interface, the high stresses at the tire-pavement interface dislodge the aggregate particles. The strength of the bond at the aggregate-asphalt binder interface is directly related to the flexibility of the asphalt binder. Therefore, an asphalt binder with low aging susceptibility can effectively maintain a good bond. Once the bond is broken, aggregate particles are dislodge as shown in figure 13. Continuous attack by moisture accelerates the dislodging of aggregate particles as shown in the right side of figure 13.

- Prevention of Raveling: the main causes of raveling are moisture damage of the AC mix and poor compaction during construction (low in-place density). The following represents methods to prevent raveling of flexible pavements.
  - Treat moisture susceptible aggregates with hydrated lime or treat the asphalt binder with liquid anti-strip additive if allowed by specifications.
  - Assure adequate AC mix compaction during construction.
  - Select an asphalt binder with appropriate PG and minimal long-term aging potential.
  - Seal the pavement surface within 3–5 years after construction in order to slow down the aging of the asphalt binder and prevent moisture penetration into the AC mix.

- Ensure adequate asphalt binder content in the AC mix.
- Polymer and terminal blend rubber modified asphalt binders have shown significant improvement in the AC mix resistance to moisture damage.

- Method of Repair of Raveling: a flexible pavement experiencing raveling can be repaired with the following options.
  - Option 1: Application of a pavement preservation treatment: chip seal, slurry seal, or micro-surfacing.
  - Option 2: Removal and replacement of the affected portion of the AC layer.

Figure 13. Typical raveling of flexible pavements in northern Nevada.

2.2.c Combination Distresses

This type of distress is the result of a combination of multiple distresses caused by either traffic loads or the environment.

Potholes

Potholes are formed in the AC layer as a result of severe raveling and cracking. However, two types of potholes can be formed: a) Functional Pothole, and b) Structural Pothole. The following presents a guidance on the identification, possible cause, and recommended action for each type of pothole.

Functional Pothole: This type of pothole is characterized as being shallow and limited to the depth of the surface treatment (i.e., single or sequential micro-surfacing) as shown in figure 14. This type of pothole is classified as functional due to the absence of cracks deviating from its outer edges and the solid un-damaged appearance of the exposed AC layer (figure 14).

Figure 14. Typical functional pothole on surface treatment.

The main cause of this functional pothole is the deterioration of the single or sequential surface treatment (i.e., micro-surfacing or slurry seal). However, the cause of deterioration of the treatment may be dependent on the type of treatment, i.e., single or sequential. In the case of single treatment, early stage moisture damage can cause the localized failure. In the case of sequential treatments, stacking of more than three treatments over the AC surface may cause a weak plane at any of the interfaces that is vulnerable to moisture damage. It should be recognized that the role of the treatment is to seal the AC surface and protect the AC mixture from oxidative aging. While surface treatments do an excellent job in these two functions, they experience oxidative aging due to the relatively thin structure and the large exposed area of the asphalt binder within the treatment. Therefore, stacking more than three aged surface treatments may create a weak layer that is highly susceptible to moisture damage.

Based on these observations, it is recommended that the practice of sequential surface treatments should be limited to three applications. At the end of the performance life of the third surface treatment, the in-place surface layer should be micro-milled to the level of the in-place AC layer and the cycle of surface treatment re-started.

The most effective and practical method to repair the functional pothole is to fill the distressed area with a fresh surface treatment mix that is compatible with the existing material. This activity can be applied through the use of the rut-filling box on the treatment machine. The hole filling method is recommended to fully preserve the existing AC layer that is still very solid and un-damaged as shown in figure 14. It should be recognized that removing and patching the good and stable AC mix would lead to more problems than solutions.

Structural Pothole: This type of pothole is characterized as being deep and surrounded by multiple distresses. Figure 15 shows three types of structural potholes; the left side shows a low severity structural pothole in a surface sealed pavement, the middle shows a severe structural pothole on a residential road after heavy rains, and the right side is the end result of fatigue cracking (interconnected cracks create small chunks of pavement that can be dislodged as vehicles drive over them). These types of pothole are classified as structural due to the presence of cracks deviating from their outer edges and the deteriorated appearance of the exposed AC layer.

Figure 15. Typical structural potholes.

The main cause of this structural pothole is the lack of the timely repair of the initial distresses or functional pothole. A functional pothole that is left un-repaired will progress to an advanced damage stage leading to a structural pothole.

The most effective and practical method to repair a structural pothole is to cut and patch the deteriorated area following the standard procedures for full-depth patching of AC layer. It should be recognized that the existing AC mix within the structural pothole is already deteriorated and should be removed to avoid further deterioration of the surrounding area and the base layer underneath the AC surface. Even though the pothole and the surrounding pavement have already been damaged, a timely repair is recommended to reduce the potential damage to the base layer due to moisture penetration through the cracked AC layer.

Roughness

Roughness is defined as the deviation of the pavement surface from a planar surface. Figure 16 shows two types of pavement roughness; the left side shows a rough pavement without any other distresses and the right side shows a rough pavement with multiple associated distresses (e.g., fatigue cracking, potholes, etc.).

- Prevention of Roughness: the main cause of roughness without associated distresses is the construction of a non-smooth surface. Such a problem can be prevented through the implementation of a smoothness specification. The main cause of roughness with associated distresses is the lack of maintenance activities over the life of the pavement. An effective solution to this problem is the implementation of a pavement preservation program that can both prevent distresses from occurring and repair distresses immediately after appearing.
- Method of Repair of Roughness: the method of repair depends on the type of roughness.
    - Roughness without associated distresses: apply a functional AC overlay at a thickness of 1–2 inch.
    - Roughness with associated distresses: conduct a project level analysis to identify the causes of the distresses and select the most effective repair strategy.

Figure 16. Typical pavement roughness.

2.2.d Pavement Assessment

Surface Condition Surveys

The first step of pavement assessment is the evaluation of surface condition to identify and document the various exiting distresses. Most local agencies use commercial software to conduct this evaluation. Software allow agencies to identify the type, severity, and extent of the various distresses previously described. Using the identified distresses, the Pavement Condition Index (PCI) is calculated in accordance with *ASTM D6433 Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys* for each pavement section on a scale of 0 (worst possible condition—very poor) to 100 (best possible condition—very good).

The PCI value determined by the software based on the surface condition survey is a valuable tool to conduct network level pavement management. Using the PCI values, the agency can classify its pavement network into the major categories of: Do-Nothing, Preservation, Rehabilitation, or Reconstruction. This network level analysis allows the agency to make annual budget estimates and forecast future needs.

Identification of Applicable Treatments

The next step following the network level management is to identify the applicable treatment for every pavement section in each of the three categories: Preservation, Rehabilitation, and Reconstruction. This process is referred to as "Project Level Analysis." This Guide recommends that the Causes and Method of Repair presented in the previous sections be followed to identify the most applicable treatment for each pavement section. This will require the pavement engineer to investigate the individual distresses and not rely on the final calculated value of the PCI. In other words, the engineer must examine the cause of each distress separately and identify the treatment that can effectively repair the combination of distresses.

CHAPTER 3. MATERIAL TYPE SELECTION

3.1 Introduction

The objective of this chapter is to support pavement engineers in specifying, selecting, and approving material types and mix designs associated with pavement designs. The scope of the chapter includes AC mixtures, base courses, and in-place recycled materials. For each material type references are made to the appropriate RTC of Washoe County Orange Book Specifications.[2] In addition to specification references, additional information is provided for some materials. An example is the types of asphalt mixtures that should be specified for different traffic classifications along with the minimum thickness that should be specified for each asphalt mixture type.

Figure 17 illustrates examples of three pavement structures and the terminology used in this guide to identify materials. The pavement structure on the left is the most typical with aggregate base on subgrade and asphalt layer(s) on top of the base course. The middle pavement structure in the figure shows an existing pavement that is rehabilitated using the cold in-place recycling (CIR) technique. The CIR replaces the asphalt base course with a new asphalt surface course placed on top of it. Similarly, the pavement structure on the right portion of the figure illustrates Roadbed Modification (RBM) in-place recycling technique used to rehabilitate the existing pavement. The existing asphalt and aggregate base courses are recycled together, and in some cases with a portion of the subgrade, creating RBM on top of which a new layer of asphalt surface course is placed. With both the CIR and RBM rehabilitation techniques some of the old asphalt may be removed by coldmilling prior to in-place recycling and taken to an asphalt plant and used as reclaimed asphalt pavement (RAP) in new asphalt mixture. When using CIR and RBM techniques it is important that the designer give consideration to grade and drainage constraints.

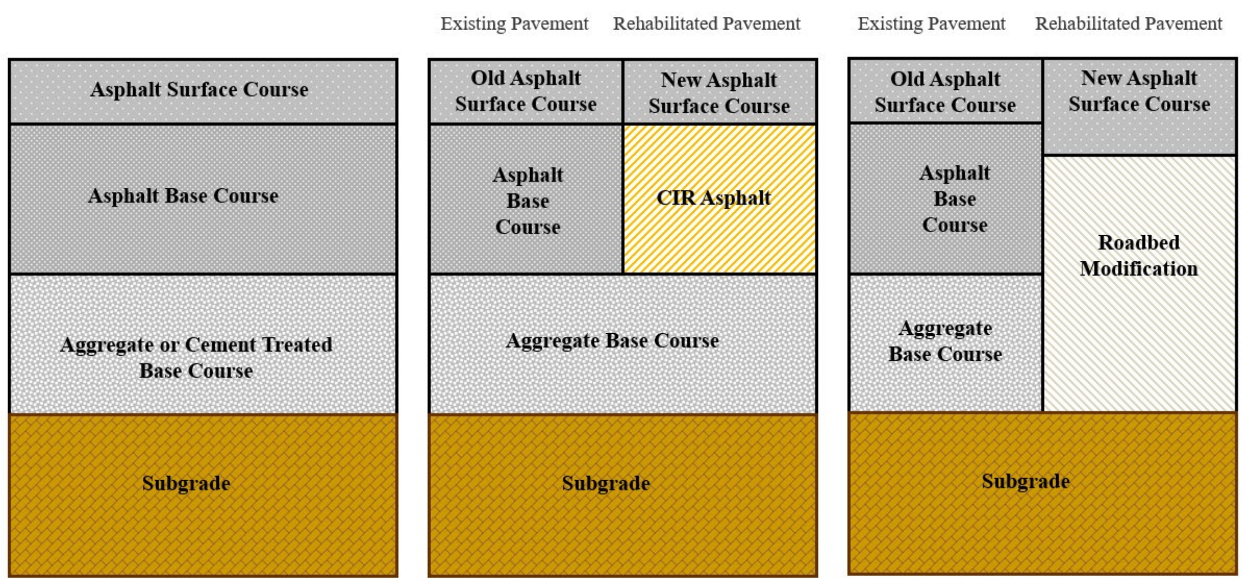

Figure 17. Example pavement structures.

[2]Standard Specifications for Public Works Construction, Orange Book (2012). Sponsored and Distributed by: Regional Transportation Commission of Washoe County, Carson City, Churchill County, City of Reno, City of Sparks, City of Yerington, and Washoe County.

3.2 Functional Classifications

Three functional classifications are used to categorize arterial, industrial, collector, local and residential roads as shown in table 1. Arterial and industrial are together in one classification. Similarly, local and residential are together in another classification. The pavement engineer needs to know the functional classification in order to select some materials. For example, arterial and industrial roads typically require the greatest structural capacity and are likely to be subjected to heavier truck traffic and loads. This would require a different asphalt surface mix type than a local or residential road. Identification of a functional classification is pertinent to all construction activities because it directs the pavement engineer towards the appropriate materials.

Table 1. Functional Classifications.

Functional Classification
Arterial/Industrial[1]
Collector
Local/Residential

[1]Industrial carries heavier loads and at least 6 percent trucks.

3.3 Construction Activities

Pavement designs and selection of specified materials for them are dependent on the type of planned construction activity. There are three basic roadway construction types: new construction, rehabilitation, or reconstruction. Table 2 breaks down the construction types, processes within each type, material selection options for each, and lists specification references for each material.

Table 2. Construction Activities, Material Selection Options, and Specification References.

Construction Activity	Construction Process	Material Selection		Location of Orange Book Specification References	
New Construction	Asphalt Concrete Roadway	Aggregate for Base Course	Asphalt Mixtures	Table 5	Figure 20, Table 3
		Cement Treated Base		Table 6	
Rehabilitation	Mill and Fill	Asphalt Mixtures		Figure 20, Table 3	
	Overlay	Asphalt Mixtures		Figure 18, Table 3	
	Cold In-Place Recycling (CIR)	Recycled Material	Asphalt Mixtures	Table 4	Figure 20, Table 3
Reconstruction	Remove and Replace	See New Construction		See New Construction	
	Roadbed Modification (RBM)	Recycled Material	Asphalt Mixtures	Table 4	Figure 20, Table 3

By working from left to right in table 2, material selection options can be identified for each construction process. The last two columns of the table list tables with details and options for each material identified. As an example, for new construction of an asphalt concrete roadway, aggregate base course and cement treated base (CTB) are options and asphalt mixture is required. Table 5 and table 6 contain aggregate base course and CTB information and Orange Book Specification section references. Figure 20 and table 3 have asphalt mixture type options and recommendations for different traffic classifications along with related Orange Book Specification section references.

3.4 Construction Materials

A brief description of each construction material identified in table 2 that could be considered when performing a pavement design follows.

3.4.a Asphalt Mixtures

Asphalt mixtures typically make up the layer(s) between the aggregate base and pavement surface. The uppermost layer of asphalt mixture is the asphalt surface course (see figure 17), which is directly exposed to the surface and in direct contact with vehicles. Asphalt layers below the surface course are referred to as asphalt base course(s). When selecting asphalt mixture types for surface layers the engineer should consider gradation type and nominal maximum aggregate size (NMAS). For residential pavements finer gradation types (Type 2 and Type 3) and smaller NMAS are desirable to provide smoother and more aesthetically pleasing surfaces for lower traffic roads. Conversely, for arterials coarser gradation types (Type 2C and Type 3C) and larger NMAS can be more appropriate for heavier traffic.

The Orange Book Specification includes four dense-graded asphalt mixture types and two open-graded types. The dense-graded types are Type 2, Type 2C, Type 3, and Type 3C. A “C” in the gradation type description indicates a coarser gradation. The open-graded types are open-graded friction course (OGFC) and asphalt treated permeable base (ATPB). The material types and specification references are shown in table 3. The table also includes NMAS and recommended lift thicknesses. Minimum lift thicknesses are very important and established to ensure the lift thickness is at least 3–4 times the NMAS to allow for adequate compaction, which is critical to long-term performance. For Type C asphalt mixtures 4 times the NMAS is recommended. The recommended minimum lift thicknesses in table 3 should not be compromised.

Table 3. Asphalt Mixture Materials and Specification References.

	Material Type	Gradation	Gradation Reference Table (2012 Orange Book)	Aggregate Requirements Table (2012 Orange Book)	NMAS (inch)	Lift Thickness (inch)
Asphalt Mixtures	OGFC	Type 1	200.02.04-I	200.02.04-II	1/2	1.25
		Type 2	200.02.04-I	200.02.04-II	3/8	0.75
	ATPB	ATPB	200.02.04-I	200.02.04-II	3/4	2.5–4
	Dense-Grade Plantmix and Roadmix (Marshall)	Type 2C	200.02.03-I	200.02.03-II	3/4, 1	3–4
		Type 2	200.02.03-I	200.02.03-II	3/4	2.5–4
		Type 3C	200.02.03-I	200.02.03-II	1/2	2–4
		Type 3	200.02.03-I	200.02.03-II	3/8, 1/2	1.5–4

It is important to note that if minimum lift thicknesses cannot be accommodated when selecting surface and base asphalt mixtures for a given pavement design, then a single mixture should be specified that it be an appropriate surface mixture for the functional classification. Similarly, if an agency elects to use a single mixture type for surface and base mixtures, then it should be an appropriate surface mixture for the functional classification.

The Orange Book Specifications relies on the Marshall mix design method for asphalt mixtures. It has provisions for two levels of compaction effort (50 and 75 blows), as well as two levels of air voids at which optimum asphalt binder content is selected. Both options influence the optimum asphalt binder content of a mixture design. As compaction level increases from 50 to 75 blows, optimum asphalt binder content decreases as illustrated in figure 18. As the air void level at which the optimum asphalt binder content is selected is reduced from 4.0 to 3.0 percent, the optimum asphalt binder content increases as illustrated in figure 19.

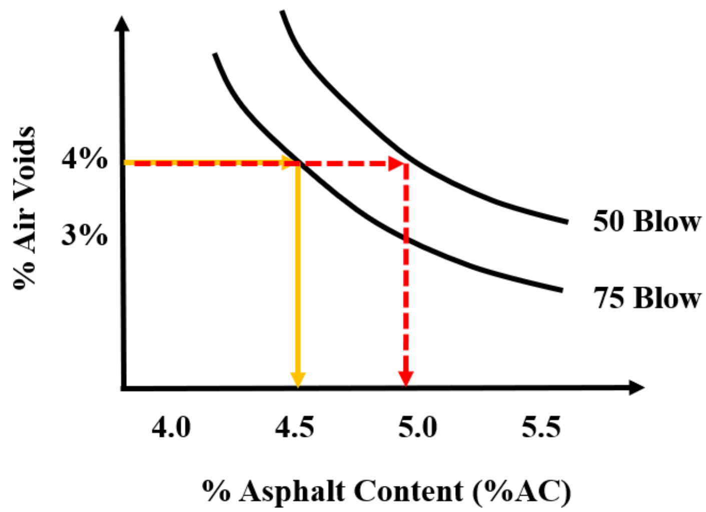

Figure 18. Influence of mix design compaction level (blows) on optimum asphalt binder content.

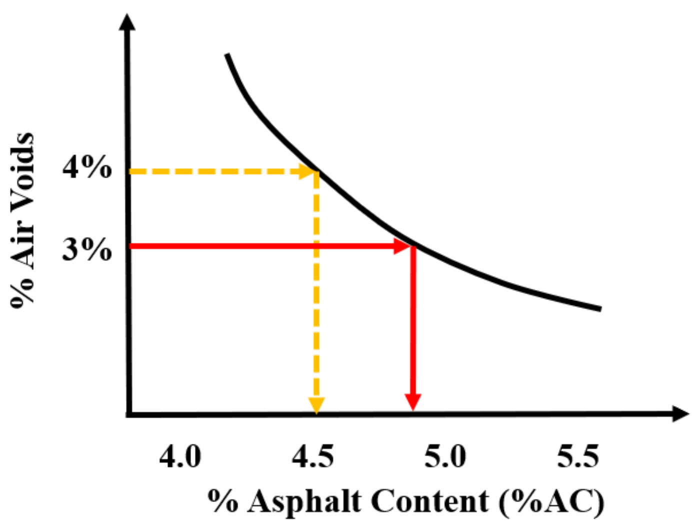

Figure 19. Influence of mix design air void level on optimum asphalt binder content.

As asphalt binder content increases, rutting resistance decreases and durability increases. The goal of a mixture design process is to balance the asphalt binder content of a mixture to assure adequate rutting resistance or stability and durability or cracking resistance. In general, the higher the traffic level the higher the compaction effort and design air void content should be. And vice versa, the lower the traffic level, the lower the compaction effort and design air void content should be. When this is coupled with gradation type and NMAS options, as well as allowable RAP levels, the pavement engineer is faced with up to 32 different mixtures to select from when specifying the right asphalt mixtures for a given pavement design.

To assist with this decision process, figure 20 was developed to provide guidance on the more applicable asphalt mixtures for different traffic classifications, mixture types, and locations of the mixtures in a pavement structure. The figure includes numbering and color-coding in the "Applicability" portion. The lower the number (0 through 3) the less applicable the mixture design. The color-coding in the cells also indicates applicability (0 = red, 1 = yellow, 2 = light green, and 3 = dark green). As an example, for the surface course of a residential street the most applicable mixture designs would be dense-graded Type 3 or Type 3C, incorporating up to a maximum of 15 percent RAP, at 3 or 4 percent design air voids, and 50 or 75 blows of compaction. A lower traffic residential street would likely have a Type 3, 50-blow mix.

Note that OGFC is a high air void, high permeability, high friction mixture which is only applicable for surface courses on arterials with high traffic speeds. OGFC also helps reduce noise and can reduce splash and spray (improving visibility) during rainy weather. OGFC layers are non-structural layers that do not contribute to a pavement structural capacity and should not be considered in the structural design of a flexible pavement.

Functional Classification	**Layer**	**Applicability**								
Local / Residential	Surface Course *(0-15% RAP)*	3	3	3	3	2	1	0	0	0
	Base Course (*0-30% RAP*)	3	3	3	3	2	1	0	0	0
Collector	Surface Course *(0-15% RAP)*	0	0	2	2	3	3	0	0	0
	Base Course (*0-30% RAP*)	0	0	2	2	3	3	1	1	0
Arterial / Industrial	Surface Course (*0-15% RAP*)	0	0	0	0	2	3	2	2	2
	Base Course (*0-30% RAP*)	0	0	0	0	2	3	3	3	0
Mixture Design Types Options	Mixture Type	Dense Graded Type 3		Dense Graded Type 3C		Dense Graded Type 2		Dense Graded Type 2C		OGF (non-structural layer)
	NMAS (inch)	3/8, 1/2		1/2		3/4		3/4, 1		3/8, 1/2
	Percent Lab Voids	3, 4		3, 4		4		4		n/a
	Marshall Blows	50	75	50	75	50	75	50	75	n/a
Note: The lower the number (0 through 3) the less applicable the mixture design. The color-coding in the cells also indicates applicability: 0 = red, 1 = yellow, 2 = light green, and 3 = dark green.										

Figure 20. Mix design type options based on functional classification and applicability.

3.4.b In-place Recycled Materials

There are two in-place recycling techniques that have successfully been used in northern Nevada and nationally. They are cold in-place recycle (CIR) and roadbed modification (RBM) that is also commonly referred to as full depth reclamation (FDR). CIR relies on a recycling train to mill out a portion of existing pavement while staying in the AC layer and recycling it in place removing the bulk of existing cracks. The depth of CIR is typically 2 to 5 inches and it is normally surfaced with at least 2 inches of AC mixture. Once the material has been milled, the recycled asphalt gets mixed with an emulsion or foamed asphalt. Note that some of the existing AC may be removed by coldmilling prior to the CIR, especially in situations where grade controls such as curb and gutter or drainage constraints exist. Active fillers such as lime or cement can also be added to strengthen the recycled mixture. Once the recycled materials have been paved, the operation becomes very similar to a normal AC paving operation. The CIR is then covered by an AC mixture to create the final product. Cold center plant recycling (CCPR) is similar, but instead of paving the material directly after milling and mixing, the milled material is sent to a plant to be processed. One benefit of this process is that the materials can be more closely monitored for quality and consistency. This technique has not been used in northern Nevada, but is becoming popular nationally. Table 4 provides a list of in-place recycled material types, and typical thicknesses.

Table 4. In-Place Recycled Materials Information.

<table>
<tr><th colspan="2">Material Type</th><th>Binder Application</th><th>Additives</th><th>Typical Thickness (Minimum Inches)</th><th>Specification Reference[1]</th></tr>
<tr><td rowspan="5">Recycled Material</td><td rowspan="2">Cold In-place Recycle (CIR)</td><td>Emulsion</td><td>Lime
Cement</td><td>2-5</td><td>Structural Design Guide for Flexible Pavements (RTC)</td></tr>
<tr><td>Foam</td><td>Lime
Cement</td><td>2-5</td><td>Structural Design Guide for Flexible Pavements (RTC)</td></tr>
<tr><td>Roadbed Modification (RBM)</td><td>Cement</td><td></td><td>6-12</td><td>200.01.04-I, 200.01.04-II</td></tr>
</table>

[1]The RTC of Washoe County Orange Book does not contain a provision for CIR or RBM using asphalt emulsions.

RBM is another recycling technique that has been successfully used in northern Nevada. With RBM a pulverizer is used to break up and blend an existing AC layer, as well as a portion of or all the underlying aggregate base layer and commonly a portion of the subgrade. The materials are blended together, with cement or lime, and then the material is bladed and compacted. Similar to CIR, the RBM material is surfaced with AC mixture for a final product. With both CIR and RBM it is not uncommon to remove a portion of the existing AC surface prior to conduction the in-place recycling. The depth of removal is a function of geometric constraints and overall new pavement design thickness. The RTC of Washoe County Orange Book does not contain a provision for CIR using foamed emulsion or RBM using emulsions. A special provision has been used in the area for CIR. RBM is commonly performed with an asphalt emulsion in other parts of the U.S.

3.4.c Aggregate for Base Courses

Base course is placed above the subgrade and below the AC layer. Material used for the base course layer needs to meet certain aggregate requirements and density when being placed. The base course is important to the overall structure of the roadway because it provides the asphalt layer support. Table 5 provides a list of aggregate base material types, gradations, NMAS, minimum lift thicknesses, and Orange Book Specification sections.

Table 5. Aggregates for Base Course Materials and Specification References.

	Material Type	Gradation	Gradation Reference Table (2012 Orange Book)	Aggregate Requirements Table (2012 Orange Book)	NMAS (inch)	Maximum Lift Thickness (inch)
Aggregates for Base Courses	Crushed Aggregate Base	Type 1, Class A	200.01.03-I	200.01.03-II	2	6
		Type 2, Class B	200.01.03-I	200.01.03-II	1	6
	Recycled Aggregate Base	Type 1 (Imported)	200.01.04-I	200.01.04-II	2	6
		Type 2 (On-site)	200.01.04-I	200.01.04-II	2	6[1]
	Select Natural Base	Type 1 (Coarse)	200.01.07-I	200.01.07-II	1.5	6
		Type 2 (Fine)	200.01.07-I	200.01.07-II	1	6
	Pit Run Subbase		200.01.08-I	200.01.08-II	4	6
	Structural Fill		200.01.09-I	200.01.09-II	4	6

[1]May be greater than 6 inches if used for RBM.

3.4.d Cement Treated Base

Cement treated base (CTB) is used in situations that require a more supportive base material than what can be obtained with aggregate base course. In some cases where very week subgrade soils are encountered this can be a good alternative. The cement bonds with aggregate base material and creates a high modulus material. This reduces the design thickness and allows for more structural support. CTBs require a mixture design and approval from an engineer prior to placing and compaction. Table 6 provides a list of CTB material types, gradations, NMAS, minimum lift thicknesses and Orange Book Specification sections.

Table 6. Cement Treated Base Materials and Specification References.

	Material Type	Gradation	Reference Table (2012 Orange Book)	Aggregate Requirements Table (2012 Orange Book)	NMAS (inch)	Maximum Lift Thickness (inch)
Cement Treated Base	Crushed Aggregate Base	Type 2, Class B	200.01.03-I	200.01.03-II	1	6
	Recycled Aggregate Base	Type 1 (Imported)	200.01.04-I	200.01.04-II	2	6
		Type 2 (On-site)	200.01.04-I	200.01.04-II	2	6

3.5 Using the Materials Selection Guide

The material selection guide allows the pavement engineer to select material based on traffic classification and planned construction activities. The goal is to make it possible to consistently and easily select the right materials for the right project specific conditions. When a pavement design is performed it is done based on a project specific site investigation that includes determination of the existing pavement condition, layer thicknesses, and determination of underlying material properties for existing pavements. For new construction the investigation is limited to determining the in-situ subgrade soil conditions.

Table 7 is included below to illustrate an example of using the materials selection guide. It shows common construction processes used to address different pavement distresses. The first step of the material selection guide is to identify the pavement distresses associated with the pavement being evaluated. This will help select the proper construction activity needed.

Table 7. Pavement Distresses and Associated Construction Processes.

<table>
<tr><th></th><th colspan="6">Construction Process</th></tr>
<tr><th></th><th>New Construction</th><th>Remove and Replace</th><th>CIR</th><th>RBM</th><th>Mill and Fill</th><th>Overlay</th></tr>
<tr><td rowspan="4">Pavement Distresses</td><td></td><td colspan="3">Fatigue Cracking</td><td>Raveling</td><td rowspan="2"></td></tr>
<tr><td colspan="2">Total Rutting</td><td colspan="3">Reflective Cracking</td></tr>
<tr><td rowspan="2"></td><td colspan="3">Transverse Cracking</td><td colspan="2">AC Rutting</td></tr>
<tr><td colspan="3">Block Cracking</td><td colspan="2">Bleeding</td></tr>
</table>

Example 1: New Construction, arterial roadway, good quality subgrade soil.

- *Step 1*: Reference Table 1 to determine the functional classification: Arterial.
- *Step 2*: Reference Table 2 to identify the type of construction activity: New Construction.
- *Step 3*: Reference Table 2 to identify base course material options: Aggregate Base and Cement Treated Base (CTB). Since the subgrade soil is of good quality select aggregate base.
- *Step 4*: Reference Table 5 and select Type 2 Class B crushed aggregate base with NMAS of 1 inch and minimum thickness of 6 inches.
- *Step 5*: Reference Table 2 and review Figure 20 to select asphalt surface course mixture type: Dense Grade Type 2, ¾ inch NMAS, 4 percent air void design, and 75 blow compaction with up to 15% RAP surface course mixture. Reference Table 3 and determine the minimum lift thickness for Type 2 that is 2.5 inches.
- *Step 6*: reference Table 2 and review Figure 20 to select asphalt base course mixture types: Dense Grade Type 2C, ¾ inch NMAS, 4 percent air void design, and 75 blow compaction with up to 30% RAP base course mixture. Reference Table 3 and determine the minimum lift thickness for Type 2C that is 3 inches.
- *Step 7*: perform the pavement structural design and if the total asphalt mixture thickness is less than the sum of the minimum thickness for the identified surface and base mixtures (2.5 inches of surface plus 3.0 inches of base AC mixtures = 5.5 inches), then only specify Dense Grade Type 2, ¾" NMAS, 4 percent air void design, and 75 blow compaction with

up to 15% RAP for surface and base course mixture. The reason for this is that if the total thickness is less than 5.5 inches the minimum thickness of 3 inches for the Type 2C mix and 2.5 inches for the Type 2 mix cannot be satisfied.

CHAPTER 4. TRAFFIC CONSIDERATION FOR PAVEMENT DESIGN

Traffic loads play a major role in the structural design of flexible pavements. The majority of the distresses encountered on flexible pavements are caused by traffic loads. It can be hypothesized that the environment plays the role of the conditioner while traffic plays the role of the executor. For example, the environment causes moisture damage of the various pavement layers while traffic loads impose the stresses that lead to the eventual failure in terms of various forms of rutting, shoving, or cracking.

The method of considering traffic loads in the pavement design process depends on the design approach; empirical or mechanistic-empirical. Currently, there are two commonly used pavement design approaches by the American Association of State Highway and Transportation officials (AASHTO); Empirical–"AASHTO 1993 Pavement Design Guide" and Mechanistic-Empirical (M-E)–"AASHTO Mechanistic-Empirical Pavement Design Guide." This chapter presents traffic considerations for the two pavement structural design approaches.

4.1 Traffic Consideration for the AASHTO 1993 Guide

The AASHTO 1993 Pavement Design Guide uses the concept of Equivalent Single Axle Load (ESAL) to incorporate traffic loads into the structural design process. The ESAL concept is based on the following hypothesis: *"The amount of damage on a given pavement structure caused by any type of axle and any level of axle load can be expressed in terms of the amount of damage on the same pavement structure caused by a single axle loaded with 18,000 lb."*

Using the ESAL concept, the AASHTO 1993 Guide implements the following process to determine the cumulative traffic over the design life of the pavement. An example traffic calculation follows to illustrate the process.

1. Estimate the number of axles in each axle type of: single, tandem, and triple.
2. Estimate the number of repetitions of each axle type in each axle load range.
3. Estimate the structural number (SN) of the designed pavement structure.
4. Define the terminal serviceability (P_t) of the designed pavement structure.
5. Identify the Load Equivalency Factor (LEF) for each axle type and axle load range from the appropriate table in the AASHTO 1993 Guide.
6. Multiply the number of repetitions of each axle type and every axle load range by its corresponding LEF.
7. Sum all the values from step 6 to determine the total ESALs for the first year of design.
8. Define the design life (Y) of the pavement in terms of years.
9. Determine a representative traffic annual growth rate (r) over the design life. The r value should not be less than +0.5%.
10. Calculate the traffic growth factor (G) over the design life.
11. Multiply the first year ESALs by the determined traffic growth factor to estimate the cumulative ESALs over the design life.

4.1.a Example of Design ESALs Calculations

This sample calculation presents the actual method for determining the design traffic following the process prescribed in AASHTO 1993 Guide. The following information are used in the calculations:

- Estimated structural number (SN) of the designed pavement: 4.0
- Terminal serviceability (P_t) of the designed pavement: 2.5
- Traffic annual growth rate (r): 3%
- Design life (Y): 20 years

Based on the SN of 4.0 and P_t of 2.5, the LEFs can be obtained from Tables D.4, D.5, and D.6 of the AASHTO 1993 Guide for single, tandem, and triple axles, respectively. Table 8 summarizes the calculated ESALs for each of the axle type.

Table 8. Summary of Design ESALs Calculations.

Axle Type	**Axle Load (kips)**	**Number of Repetitions During First Year**	**AASHTO 1993 LEF**	**ESALs**
Single	16	1,000	0.645	645
	18	3,000	1.00	3,000
	20	2,500	1.47	3,675
	22	8,000	2.09	16,720
	24	4,000	2.89	11,560
	26	1,000	3.91	3,910
Tandem	26	1,000	0.401	401
	28	2,500	0.534	1,335
	30	4,500	0.695	3,128
	32	5,000	.887	4,435
	34	10,000	1.11	11,100
	36	8,500	1.38	11,730
Triple	40	500	0.533	267
	42	750	0.644	483
	44	2,000	0.769	1,538
	46	4,500	0.911	4,100
	48	5,500	1.07	5,885
	50	4,000	1.25	5,000
First Year ESALs	88,912			

Based on the traffic annual growth rate and design life, the traffic growth factor (G) over the design life can be calculated from Equation 1:

$$G = \frac{[(1+r)^{Y}-1]}{r} \qquad [1]$$

$$G = \frac{[(1+0.3)^{20}-1]}{0.3} = 27$$

The cumulative ESALs over the entire design life of Y-years can be calculated from Equation 2:

$$\text{Design ESALs} = \text{First Year ESALs} \times G \qquad [2]$$

$$\text{Design ESALs} = 88{,}912 \times 27 = 2{,}400{,}600$$

Looking at the sample calculations above, it can be observed that following the process recommended by the AASHTO 1993 Guide to determine design ESALs require extensive traffic data in terms of number of repetitions of each axle type at every axle load level. Most likely this type of traffic information will not be available for the majority of state highway agencies and very highly unlikely to be available for local government agencies. The following section presents a recommended simpler method to determine the design ESALs to be incorporated into the structural pavement design process.

4.1.b Recommended Method for Design ESALs Calculations

This method for design ESALs calculations is based on information that are commonly available to most agencies involved in the design of pavement structures. The design ESALs are estimated from the relationship shown in Equation 3:

$$\text{Design ESALs} = AADT_1 \times 365 \times T \times T_f \times G \qquad [3]$$

Where;

$AADT_1$: annual average daily traffic for the first year
T: percent of trucks in the traffic volume (decimal)
T_f: truck Factor
G: Traffic growth factor (Eq. 1)

The annual average daily traffic and the percent of trucks in the traffic volume can be measured from traffic studies using automatic vehicle classification (AVC). The determination of the truck factor requires the implementations of the following process:

- Measure the number of repetitions in every axle type and at each load level.
- Conduct the measurement over at least a 1-week period.
- Calculate the total ESALs generated by the total number of trucks (N_T) following the AASHTO 1993 Guide process as outlined in Section 4.1.a.
- Calculate the Truck Factor (T_f) as the ratio of the total ESALs over the number of trucks weighed.
- For example:
    - A total of 1,650 trucks were weighed over the 1-week period
    - The calculated total ESALs was 2,000
    - $T_f = 2{,}000/1{,}650 = 1.21$

A sample calculation of the Design ESALs is shown below:

- $AADT_1 = 1{,}350$
- Truck Percent: T = 15%
- $T_f = 1.21$
- Design Life = 20 years
- Traffic growth rate = 3%, G = 27 (Eq. 1)

The design ESALs for a 20-year pavement life can be calculated as follows:

$$\text{Design ESALs} = 1{,}350 \times 365 \times 0.15 \times 1.21 \times 27 = 2{,}414{,}722$$

Typically, state highway agencies establish truck factors for the various regions within the state through annual portable weigh in-motion studies conducted by the traffic division. Some state highway agencies publish the truck factors on their website. It is highly recommended that local agencies obtain the appropriate truck factor for their region from the state highway agency. Table 9 summarizes the truck factors published by Nevada Department of Transportation (NDOT) in 2007.[3] A sample calculation of the truck factor for an assumed vehicle distribution is shown in table 10.

Table 9. Flexible Pavement T_f as a Function of Vehicle Classes (Based on 2007 Annual Traffic Report by NDOT).

Vehicle Classification		Urban	Rural	
		Arterial	**Arterial**	**Collector**
Buses		0.839	0.669	0.686
Buses (Catalyst E2)	2 Axle	3.150	3.150	3.750
Bus Rapid Transit (BRT)*	3 Axle	5.197	–	–
Single-Unit Trucks	2 Axle, 6 Tire	0.239	0.203	0.377
	3 Axle or More	0.938	0.679	0.923
Single-Trailer Units	4 Axle or Less	0.680	0.482	0.808
	5 Axle	1.285	1.094	1.158
	6 Axle or More	1.259	1.236	0.803
Multi-Trailer Trucks	5 Axle or Less	2.099	3.308	1.663
	6 Axle	0.593	1.449	0.000
	7 Axle or More	1.844	2.450	1.426

*New Flyer

Table 10. Sample Calculation of Weighted Average T_f for an Arterial in Urban Area.

Vehicle Classification		Vehicle Class Distribution (VCD)	Vehicle T_f	Weighted T_f[(b)]
Buses		0.48%	0.839	0.0040
Single-Unit Trucks	2 Axle, 6 Tire	2.40%	0.239	0.0057
	3 Axle or More	0.41%	0.938	0.0038
Single-Trailer Units	4 Axle or Less	0.41%	0.680	0.0028
	5 Axle	0.69%	1.285	0.0089
	6 Axle or More	0.05%	1.259	0.0006
Multi-Trailer Trucks	5 Axle or Less	0.11%	2.099	0.0023
	6 Axle	0.06%	0.593	0.0004
	7 Axle or More	0.23%	1.844	0.0042
Bus Rapid Transit (BRT)	3 Axle	0.35%	5.197	0.0182
	Truck Percent[(a)]	**5.19%**	**Weighted Average T_f**[(c)]	**0.9824**

[(a)]Truck Percent is equal to the sum of VCD.
[(b)]Weighted T_f is equal to VCD × Vehicle T_f.
[(c)]Weighted Average T_f is equal to the sum of Weighted T_f divided by Truck Percent.

[3] *The Annual Traffic Report*, Nevada Department of Transportation, Traffic Information Division in cooperation with the U.S. Department of Transportation, Federal Highway Administration (2007).

For AASHTO 1993 Guide empirical flexible pavement designs use of the PaveXpress free online pavement design tool (http://www.pavexpressdesign.com/) is recommended for traffic calculations.

4.2 Traffic Consideration for the AASHTO M-E Guide

The AASHTO M-E Guide requires traffic distributions for each of the standard Federal Highway Administration (FHWA) vehicle classes listed in table 11. Furthermore, the AASHTO M-E utilizes the axle load spectra (i.e., daily, monthly, and seasonal distributions of the traffic with respect to axle type/load of various vehicle classes) as an input to the analysis process. This represents a major departure from the ESAL concept that is used in the AASHTO 1993 Guide.

Table 11. FHWA Vehicle Classes.

Vehicle Class Number	Standard FHWA Vehicle Classes
1	Motorcycles
2	Passenger Cars
3	Other 2-Axle, 4-Tire Single Unit Vehicles
4	Buses
5	2-Axle, 6-Tire, Single-Unit Trucks
6	3-Axle Single-Unit Trucks
7	4+ Axle Single-Unit Trucks
8	3 or 4 Axle Single-Trailer Trucks
9	5-Axle Single-Trailer Trucks
10	6+ Axle Single-Trailer Trucks
11	5 or fewer Axle Multi-Trailer Trucks
12	6-Axle Multi-Trailer Trucks
13	7+ Axle Multi-Trailer Trucks

Consequently, four groups of traffic data are required by the AASHTO M-E Guide for pavement structural design as listed in table 12: 1) Traffic-related Design Properties, 2) Traffic Volume Adjustment Factors, 3) Axle Load Distribution Factors, and 4) General Traffic Input. A brief definition of the various data elements within each group is also provided in table 12.

The traffic data elements listed in table 12 are eventually needed for any pavement design conducted using the AASHTO M-E Guide. However, the degree of details of the various data elements depends on the design levels as described below:

Table 12. Traffic Data Inputs Required for the AASHTO M-E Guide.

Description	Variable	Brief Definition
Traffic-Related Design Properties	Initial two-way average annual daily truck traffic ($AADTT_0$)	Total volume of truck (classes 4-13) passing a point or segment of a road facility to be designed in both directions during a 24-hour period
	Number of lanes in design direction	Total number of lanes in one direction
	Trucks in the design direction (%)	Quantify any difference in the overall volume of trucks in two directions referred to as direction distribution factor (DDF)
	Trucks in the design lane (%)	Accounts for the distribution of truck traffic between the lanes in one direction referred to as truck lane distribution factor (LDF)
	Operational speed	Posted truck speed limit or the average travel speed of trucks
Traffic Volume Adjustment Factors	Monthly/seasonal adjustment factors (MAF)	Proportion of the annual truck traffic for a given truck class that occurs in a specific month
	Vehicle class distribution (VCD, %)	Percentage of each truck class (classes 4 through 13) within the AADTT for the base year
	Traffic growth factors (%)	Account for growth or decay in truck traffic over time
Axle Load Distribution Factors	Axle load distribution factors by axle type (ALDF)	Percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (classes 4 through 13)
General Traffic Inputs	Mean wheel location (inches from lane marking)	Distance from the outer edge of the wheel to the pavement marking
	Traffic wander standard deviation (in)	Standard deviation of the lateral traffic wander
	Design lane width (in)	Distance between the lane markings on either side of the design lane.
	Number of axle types per truck class	Average number of axles for each truck class (class 4 to 13) for each axle type (single, tandem, tridem, and quad)
	Axle configuration: • Axle width • Dual tire spacing • Tire pressure • Axle spacing	 • Distance between two outside edges of an axle • Distance between centers of a dual tire • Median value for hot inflation tire pressure • Distance between the two consecutive axles of a tandem, tridem, or quad
	Wheel base distribution • Average axle spacing • Percent of trucks	 • Average longitudinal distance between two consecutive axles that fall under the short, medium and long axle spacing (class 8 to 13) • Percentage of trucks in class 8 through 13 with the short, medium and long axle spacing

- **Level 1 Inputs–Site Specific Vehicle Classification and Axle Weight Data.** Level 1 is considered the most accurate and extensive design level as it uses the actual axle weights and vehicle class spectrum measured over or near the project site. Hence, Level 1 involves a straightforward analysis of the traffic data measured at the site including: a) counting and classifying the number of vehicles traveling over the roadway, along with the breakdown by lane and direction, and b) measuring the axle loads for each vehicle class over a sufficient period of time to reliably determine the design traffic. When such detailed data are available, their incorporation into the AASHTO M-E is not a complicated process. This level is recommended for use in designing high-volume and very important roads. However, the designer may not be able to use Level 1 under certain conditions, for example, in the development of new routes where roadways do not currently exist. For these conditions, Level 2 would need to be used.
- **Level 2 Inputs–Regional Vehicle Classification and Axle Weight Data.** Level 2 is similar to Level 1 but does not require site-specific data other than Average Annual Daily Traffic (AADT) and percent trucks information. Regional vehicle classification and axle weight data for similar road classifications are needed and used to develop axle load spectra or distributions for each vehicle class that can be used for a specific project. Level 2 has an intermediate accuracy and would be used for designing new pavements where routes do not currently exist.
- **Level 3 Inputs–Site Specific Vehicle Count Data, AADT.** Traffic data analysis for Level 3 is the simplest and requires only the vehicle counts and percent trucks information, but is the least accurate. For this level, default axle load distribution and vehicle classification distribution parameters are used with the AADT and percent trucks information to estimate the traffic data required for the AASHTO M-E design procedure. The default values to describe the vehicle classification distribution and axle load spectra that are currently incorporated in the AASHTO M-E Guide were determined from analyzing the Long-Term Pavement Performance (LTPP) traffic database for selected test sections in the US and Canada. Level 3 inputs are typically recommended for use in designing local and low-volume roads.

In summary, at input Level 1, a very thorough knowledge of traffic characteristics based on historical and site-specific traffic counts and load spectra measurements is required. At input Level 2, regional load axle spectra can be used. Finally, at input Level 3, when a very limited knowledge of traffic data is available, default values can be used. Table 13 provides additional details regarding the data collection or measurement requirements for each of the AASHTO M-E input levels.

Table 13. Traffic Data Estimation.

Variable	Level	How to acquire and/or measure
Initial two-way average annual daily truck traffic (AADTT)	1	Site-specific weigh-in-motion (WIM), static weighing stations, AVC, vehicle count data, or site calibrated traffic forecasting and trip generation models
	2	Regional WIM, static weighing stations, AVC, vehicle count data, or regionally calibrated traffic forecasting and trip generation models
	3	AADT from mostly traffic counts and an estimate of the percentage of trucks expected in the traffic stream
Number lanes in design direction	N/A	From design specifications
Trucks in the design direction (%)	1	Site-specific WIM, static weighing stations, AVC, or vehicle counts
	2	Regional WIM, static weighing stations, AVC, or vehicle counts
	3	State/regional average value or local vehicle counts/experience
Trucks in the design lane (%)	1	Site-specific WIM, static weighing stations, AVC, or vehicle counts
	2	Regional WIM, static weighing stations, AVC, or vehicle counts
	3	State/regional average value, traffic forecasting and trip generation models, or local vehicle counts/experience
Operational speed	N/A	Direct measurement of site-specific segment or calculate based on Highway Capacity Manual.
Monthly/seasonal adjustment factors (MAF)	1	Site-specific WIM, static weighing stations, AVC, or vehicle count data.
	2	Regional WIM, static weighing stations, AVC, or vehicle count data
	3	State/regional average values or local vehicle counts/experience
Vehicle class distribution (VCD, %)	1	Site-specific WIM, static weighing stations, AVC, or vehicle counts
	2	Regional WIM, static weighing stations, AVC, or vehicle counts
	3	State/regional average values or local vehicle counts/experience
Traffic growth factors (%)	N/A	Continuous or short duration AADTT counts
Axle load distribution factors by axle type (ALDF)	1	Site-specific WIM or static weighing stations
	2	Regional WIM or static weighing stations
	3	State/regional average values
Mean wheel location (inches from lane marking)	1	Direct measurement of site-specific segments
	2	Regional/statewide average
	3	State/regional average or local experience
Traffic wander standard deviation (in)	1	Direct measurement of site-specific segments
	2	Regional/statewide average
	3	State/regional average or local experience
Design lane width (in)	N/A	Direct measurement of site specific segment
Number of axle types per truck class	1	Site-specific WIM, static weighing stations, AVC, or vehicle counts
	2	Regional WIM, static weighing stations, AVC, or vehicle counts
	3	State/regional average values
Axle configuration:	N/A	Measure directly, obtain information from manufacturers, state/regional average or local experience
Wheel base distribution	N/A	Measure directly or obtain information from manufacturers

CHAPTER 5. STRUCTURAL PAVEMENT DESIGN

5.1 Introduction

The overall objective of the structural design process is to identify the required thicknesses of the various layers (surface, base, and sub-base) above the subgrade to successfully carry the combined actions of environment and traffic loads over the specified design life. One of the major limitations placed on the structural design process is the use of locally available materials in the various layers. In other words, the structural design must be optimized to generate the most reliable and cost effective (i.e., local materials) pavement that can accommodate the design traffic under the prevailing environmental conditions.

The main goal of the structural design process is to reduce the stresses generated by traffic loads to the levels that can be accommodated by the various layers while maintaining sufficient resistance to potential damages caused by the environment. This goal has been achieved through two structural design concepts: Empirical and Mechanistic-Empirical.

The empirical pavement design concept relies on statistical correlations between pavement performance and traffic loads while incorporating limited materials properties and environmental factors. In this case, the entire structural design process is based on empirical correlations that were established through actual field testing at a specific location (i.e., single environment) with limited variations in traffic and materials properties. The AASHTO 1993 Pavement Design Guide is the most commonly used empirical pavement design method for flexible pavements.

The mechanistic-empirical (M-E) pavement design concept combines a two-part analysis. The mechanistic part deals with the determination of the mechanical responses of the pavement structure in terms of stresses, strains, and displacements under the combined action of environment and traffic loads. The empirical part deals with the conversion of the determined pavement mechanical responses into long-term performance through statistical field-calibrated relationships. The mechanical responses of the pavement due to traffic loads are determined through numerical modeling while the environment is incorporated through temperature and its impacts on the aging of the AC mixture and moisture-related conditions of the unbound materials. The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) is the most recent development in the M-E pavement design process.

5.2 AASHTO 1993 Pavement Design Guide

The AASHTO 1993 Pavement Design Guide is based on empirical relationships between traffic loads, pavement structure, and pavement performance that were established based on the results of the AASHTO Road Test experiment. The AASHTO 1993 guide covers the structural design process for both flexible and rigid pavements. However, since this guide is only applicable to flexible pavements, only this type of pavement design will be presented.

The AASHTO Road Test experiment was conducted in Ottawa, Illinois during the period of 1959–1962. Multiple flexible pavement sections with varying layer thicknesses were constructed and loaded with actual truck traffic. The following limitations are applicable to the AASHTO 1993 Pavement Design Guide:

- Single type of subgrade.
- Single environment.

- Single type of AC mixture and unbound materials.
- Traffic with single and tandem axles.
- Limited axles load levels.

Over the past several decades, the AASHTO Guide has been revised to accommodate wider ranges of materials and traffic loads where the latest update was done in 1993, hence the name AASHTO 1993 Guide. Figure 21 shows the overall process used in the AASHTO 1993 Pavement Design Guide for flexible pavements. The following sections present the various parts of the design process.

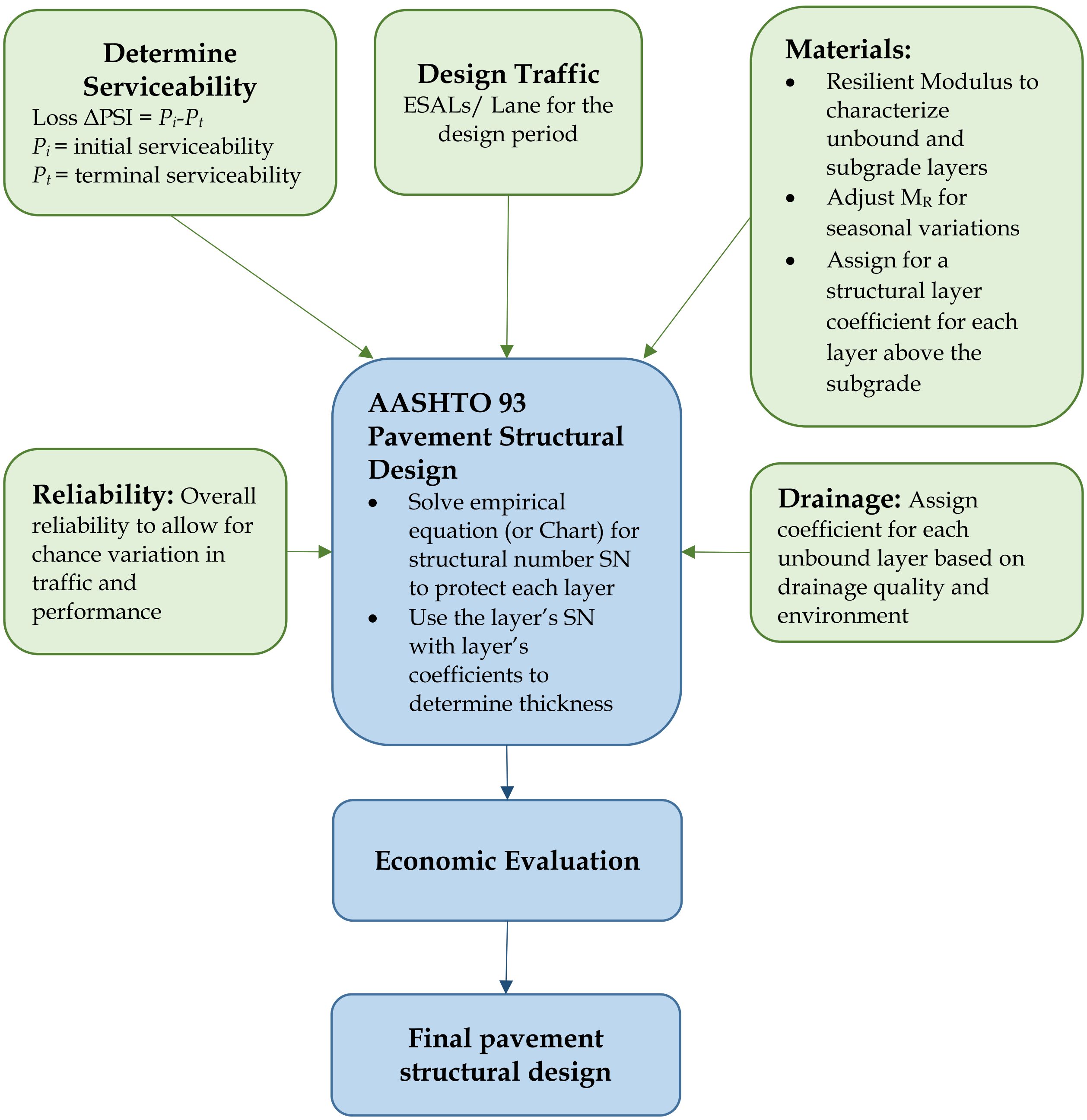

Figure 21. Overall process of the AASHTO 1993 Pavement Design Guide.

5.2.a Present Serviceability Index

The concept of the present serviceability index (PSI) was developed during the AASHTO Road Test to assign a numerical value for the quality of the pavement based on the observed surface distresses, including; roughness, rutting, cracking, and patching. The PSI ranges between 0 and 5. Figure 22 shows the anticipated relationship between PSI and traffic indicating a range of minimum PSI where action is recommended to be taken in order to avoid the need for re-construction.

The relationship shown in figure 22 is not fixed for all flexible pavements since it is highly influenced by the properties of the materials used in the various layers and the environmental conditions at the specific location. The AASHTO 1993 Guide refers to the PSI of a new pavement as the "initial serviceability, P_i" and sets it at a fixed value of 4.2 for flexible pavements. The traffic loads served by the pavement vary as a function of the serviceability level that can be tolerated by the road users. The relations between PSI and traffic loads clearly shows that there is a minimum acceptable PSI level where action is recommended. The AASHTO 1993 Guide refers to the level of PSI that can be tolerated as the "terminal serviceability, P_t". Therefore, it is the range of PSI that controls the pavement design process which is defined as; $\Delta PSI = P_i - P_t$. Since the P_i for a flexible pavement is fixed at 4.2, the pavement design process can only select the P_t as shown in table 14.

Table 14. Recommended Levels for Terminal Serviceability, P_t.

Functional Classification	Terminal Serviceability, P_t	Change in PSI, ΔPSI
Arterial/Industrial	2.5	1.7
Collector	2.0	2.2
Local/Residential	2.0	2.2

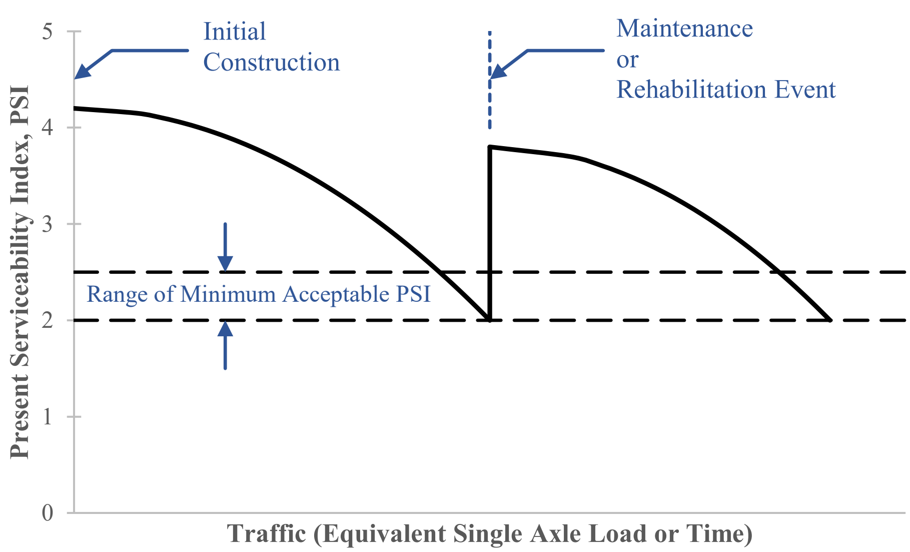

Figure 22. Typical relationship between PSI and traffic loads for flexible pavements.

5.2.b Reliability of Design

The AASHTO 1993 Guide recognizes that design parameters cannot be represented by fixed values and each parameter may have a certain level of associated variability. The two major variable design parameters are traffic loads and predicted performance. As discussed in Chapter 3, the determination of design traffic loads involves numerous assumptions and future predictions that make it extremely difficult to arrive at a precise value. In the case of performance prediction, variations are compounded from materials, natural soil, construction, and environment.

The AASHTO 1993 Guide uses two factors to estimate the reliability level of the designed flexible pavement: reliability factor (R) and overall standard deviation (S_0).

The reliability level represents the probability that the designed pavement section will perform satisfactory under the traffic and environmental conditions for the design period. The selection of the reliability level for the design of a particular pavement mainly depends on the risks associated with the possibility of the design not surviving the full design life. For example, a pavement for a high traffic volume facility must be designed with a high reliability since the consequences of its failure could be significant delays and serious interruption to the economy. Table 15 summarizes the reliability levels recommended based on the AASHTO 1993 Guide.

Table 15. Recommended Levels for Reliability, *R*.

Functional Classification	Reliability, *R*
Arterial/Industrial	90
Collector	80
Local/Residential	70

The reliability level takes into consideration the uncertainty in the traffic estimation process and performance prediction model. In order to incorporate the impact of variations in the various factors used in the traffic estimation and performance prediction model, the AASHTO 1993 Guide introduced the overall standard deviation, S_0. The S_0 is a parameter to describe the variations associated with actual traffic data, materials properties, and construction techniques. The AASHTO 1993 Guide presents a range for the S_0 of 0.3–0.5 with a recommended representative value for flexible pavements of 0.45.

5.2.c Design Lane Traffic

Chapter 3 presented the process for estimating the Design ESALs representing the traffic over the design life of the pavement structure. The next step is to derive the traffic for the design lane to be used in the structural design process. The AASHTO 1993 Guide referred to the design lane traffic as "W_{18}" to be determined using Equation 4:

$$W_{18} = \text{Design ESALs} \times D \times L \qquad [4]$$

Where;

Design ESALs: determined from Equation 3 in Chapter 3
D: directional distribution factor of 0.5 is recommended for a 50% traffic split
L: design lane distribution factor as recommended in Table 4

Table 16. Recommended Lane Distribution Factor, L.

Number of Lanes per Direction of Travel	Lane Distribution Factor, L
1	1.0
2	0.9
3	0.7
4	0.6

For example, if the facility used in the sample calculation presented in Section 3.1.b has two lanes per direction of travel, the design lane traffic will be calculated as follows:

$$W_{18} = 2{,}414{,}722 \times 0.5 \times 0.9 = 1{,}100{,}000^*$$

* *Note that design lane traffic is typically rounded-up to the nearest 100k, in this case the design lane traffic used will be 1,100,00 ESALs instead of the actual calculated number of 1,086,625.*

The structural pavement design process shall use the 1,100,000 ESALs to determine the required thicknesses of the various pavement layers in order for the pavement to survive its intended design life (i.e., 20 years in this example) with the selected levels of reliability and serviceability loss.

5.2.d Materials Characterization

Materials characterization is the most critical part of the flexible pavement design process. Since each layer is a structural layer, the structural value of the material used in each layer must be carefully determined. The following sections present the process recommended to characterize each layer in the flexible pavement structure. The recommended structural layer coefficients for the various materials are also summarized in table 17.

Table 17. Structural Layer Coefficients for Various Materials.

Layer Type	Material Type	Layer Coefficient (a_i)
Asphalt Concrete (AC) Layer	Hot Mix or Warm Mix Asphalt Mixture	0.35
Unbound Layer	Aggregate Base – Type 2, Class B	0.12
	Aggregate Base – Type 1, Class A	0.10
	Aggregate Sub-base (Pit Run)	0.07
Stabilized Layer	Cement-treated crushed aggregate base	0.18
	Cement-treated recycled aggregate base (i.e., roadbed modification—RBM)	0.18
Cold In-Place Recycling (CIR) Layer	CIR Mixture	0.25

A critical component of a pavement structural design is exploration to determine the existing structural section components (type and thickness) and the subgrade soil characteristics including soil classification, development of a soil profile along the roadway alignment, soil strengths (R-values), and potential adverse soil properties including shrink/swell potential.

Exploratory methods may include records (e.g., as-build documentations) review, coring, borings, test pits, and nondestructive testing in unique situations. Coring and borings are commonly used due to the cost of exploration and schedule. Test pits are more costly and scheduling them is dependent on a contractor's availability. Nonetheless, the test pits provide a better visual of existing pavement section profile, especially the base/subgrade soil layer interface; allow for a continuous soil profiling of subgrade soil layer, generally at least 3 feet in thickness; and allow for sampling of the subgrade soils for laboratory testing. In the case of boring, at least two borings are needed at each sampling location to provide sufficient material for testing. Continuous soil sampling by standard penetration test (SPT) methods need to be completed starting at the subgrade soil/base interface to a depth of at least 3 feet in all borings. In some rehabilitation scenarios, nondestructive testing (e.g., falling weight deflectometer) can be used along with laboratory test results to determine feasible rehabilitation alternatives. The exploratory methods used and frequency of sampling and testing are project specific (e.g., functional classification, type and frequency of existing distresses, etc.).

Asphalt Concrete (AC) Layer

The AC layer represents the surface layer of the flexible pavement. The AASHTO 1993 Guide requires only one property for the AC layer, which is the structural layer coefficient "a_1." The a_1 can be determined from a typical relationship between a_1 and the resilient modulus of the AC mixture at 68°F presented in the AASHTO 1993 Guide. However, since most agencies do not measure the resilient modulus of the AC mixture, it has been a common practice to assign a constant a_1 for all AC mixtures used by a specific agency. In the case of the North-Western Nevada, it is recommended the use of the NDOT selected value of $\boldsymbol{a_1 = 0.35}$.

Unbound Layers

Unbound materials are used in the construction of the base and sub-base (Pit Run) layers. In order to determine the correct protection for the base and sub-base layers, their engineering properties and layer coefficients must be known.

Engineering Property: The AASHTO 1993 Guide defines the engineering property of unbound materials in terms of the resilient modulus (*Mr*). The *Mr* can be determined through direct laboratory measurement per AASHTO T 307 under repeated load triaxial conditions. This approach for determining the *Mr* property of unbound materials has proven to be highly complicated and expensive for highway agencies to conduct on a routine basis. Therefore, the majority of highway agencies, including Nevada DOT, rely on empirical relationships to determine the *Mr* of unbound materials from less complicated properties. For the design of flexible pavements in North-Western Nevada, it is recommended to determine the *Mr* property of unbound materials from the following relationships:[4]

- For base layer:

$$Mr = e^{(7.1217 + 0.0366 \times R\text{-}value)} \qquad [5a]$$

- Sub-base layer:

$$Mr = e^{(8.5849 + 0.0102 \times R\text{-}value)} \qquad [5b]$$

Where;

Mr: resilient modulus of unbound material, psi

R-value: resistance value of unbound material determined in accordance with ASTM D2844

[4]Sebaaly, P. E., Thavathurairaja, J., and Hajj, E. Y. (2018). "Characterization of Unbound Materials (Soils/Aggregates) for Mechanistic-Empirical Pavement Design Guide (MEPDG)," Final Report No. P361-16-803, Nevada Department of Transportation, Carson City, NV.

It should be noted that the *R-value* test is highly variable and may lead to unrealistically high values for the unbound materials typically used in the base and sub-base layers. In order to avoid assigning high *Mr* values based on the direct results of the R-value test, it is recommended to use the measured R-value of the unbound materials to confirm that the specification is met for the minimum value of 70 and 45 for the base and sub-base layers, respectively. Once it is confirmed that the unbound materials meet the specifications, the minimum specification values should be used in Equation 5 to determine the *Mr* properties as shown below:

$$\textit{For base layer, } Mr = e^{(7.1217 + 0.0366 \times 70)} = 16{,}054 \textit{ psi use } 16{,}100 \textit{ psi}$$

$$\textit{For sub-base layer, } Mr = e^{(8.5849 + 0.0366 \times 45)} = 8{,}467 \textit{ psi use } 8{,}500 \textit{ psi}$$

Layer Coefficient: The AASHTO 1993 Guide uses the layer coefficient to represent the relative structural value of a 1-inch thickness of a given material. The total structural value provided by a given layer can be evaluated as the product of its coefficient times its thickness. The layer coefficient of unbound materials for base and sub-base can be established based on its strength property such as *Mr*, *R-value*, or California bearing ratio (*CBR*). However, since most agencies do not measure these properties on a routine basis, it has been a common practice to assign a constant layer coefficient for all unbound materials used in base and sub-base layers. In the case of the North-Western region of Nevada, it is recommended to use Nevada DOT's selected layer coefficients for base and sub-base as $a_2 = 0.10$ and $a_3 = 0.07$, respectively. For Type 2, Class B aggregate base an $a_2 = 0.12$ can be used.

Using the recommended values for layer coefficients, it can be seen that structural value of the base layer is estimated as 43% (or 71% in the case of Type 2, Class B) higher than the structural value of the sub-base layer. In other words, every 1-inch of the base layer will be equivalent to 1.43 inches (or 1.71 inches in the case of Type 2, Class B) of the sub-base layer. This relationship in the structural value of the two materials should be taken into consideration in the selection of the final pavement structure.

Stabilized Layer

Stabilized materials such as cement-treated crushed aggregate base or cement-treated recycled aggregate base (also referred to as roadbed modification—RBM) are typically used in flexible pavements to improve its overall structural capacity. The cement-treated crushed aggregate base, consisting of a mixture of aggregates, cementitious materials, and water, is typically used in new pavements. The cement-treated recycled aggregate base, consisting of a mixture of recycled aggregates, cementitious materials, and water, is mainly used in rehabilitated flexible pavements. This type of base involves the following construction process: (1) pulverizing the existing AC layer with a portion or the entire base layer; (2) blending the pulverized materials with cement at a predetermined dosage; and (3) compacting the blended pulverized materials to form a stabilized base layer.

The mix design for the stabilized materials is determined utilizing samples compacted at the optimum moisture content for the mixture in accordance with ASTM D1633, Method A. The optimum moisture content for the cement-treated crushed aggregate base and the cement-treated recycled aggregate base is determined in accordance with ASTM D558 and ASTM D698, respectively with the exception that a maximum drying temperature of 140°F is used.

The cement content is selected to achieve an unconfined compressive strength (ASTM D1633) at 7 days curing within the range of 250 to 400 psi. Typically, a cement content between 2 and 4%

has been determined necessary through mix designs to meet the unconfined compressive strength requirement for stabilized materials from North-Western Nevada. The *Mr* of the stabilized materials is determined using the relationship developed under the NCHRP Report 789:[5]

- For stabilized base layer:

$$Mr = 120 \times UCS + 9{,}980 \qquad [5c]$$

Where;

Mr: resilient modulus of stabilized material, psi

UCS: unconfined compressive strength of stabilized material at the 7 days curing determined in accordance with ASTM D1632 and ASTM D1633, psi

Based on the recommended range for the unconfined compressive strength, the *Mr* property for the stabilized materials ranges between the following two values:

For UCS of 250 psi, Mr* = 120×*250* + 9,980= *39,980 psi

For UCS of 400 psi, Mr* = 120×*400* + 9,980 = *57,980 psi

Layer Coefficient: for the North-Western region of Nevada, it is recommended to use the Nevada DOT's layer coefficient for roadbed modification as $\boldsymbol{a_2 = 0.18}$ for both the cement-treated crushed aggregate base and the cement-treated recycled aggregate base (i.e., RBM). The same layer coefficient is used for both types of stabilized layer as both are being treated with a similar amount of cement ranging between 2 and 4%.[6]

Using the recommended value for the layer coefficient, it can be seen that the structural value of the stabilized layer is estimated as 80% higher than the structural value of the unbound base layer. In other words, every 1 inch of the stabilized layer will be equivalent to 1.8 inch of the unbound base layer. This relationship in the structural value of the two materials should be taken into consideration in the selection of the final pavement structure.

Cold In-Place Recycling (CIR) Layer

CIR is an on-site 100 percent pavement recycling process to a typical treatment depth of 2–5 inches. An additive or combination of additives (asphalt emulsions, rejuvenating agents, foamed asphalt, lime, fly ash, or cement) may be used. The CIR construction process can be summarized by the following steps:

1. Milling and crushing of the existing pavement
2. Mixing and addition of a recycling agent
3. Lay down
4. In-place compaction
5. Placement of the overlay/wearing course

Figure 23 shows the CIR process using a train of equipment (tankers, trucks, milling machines, crushing and screening units, and mixer). The water tanker supplies the optimum water content to achieve the maximum density of the compacted CIR material. The optimum water content and

[5] Wen, H., Muhunthan, B., Wang, J., and Li, X. (2014). "Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis," NCHRP Report 789, Transportation Research Board, Washington, D.C.

[6] Gaspard, K.J. (2000). "Evaluation of Cement Treated Base Courses," Report No. 00-1TA, Louisiana Transportation Research Center, Louisiana Department of Transportation and Development, Baton Rouge, LA.

maximum density are determined in the laboratory mix design through the modified proctor compaction method as specified in AASHTO T 180.

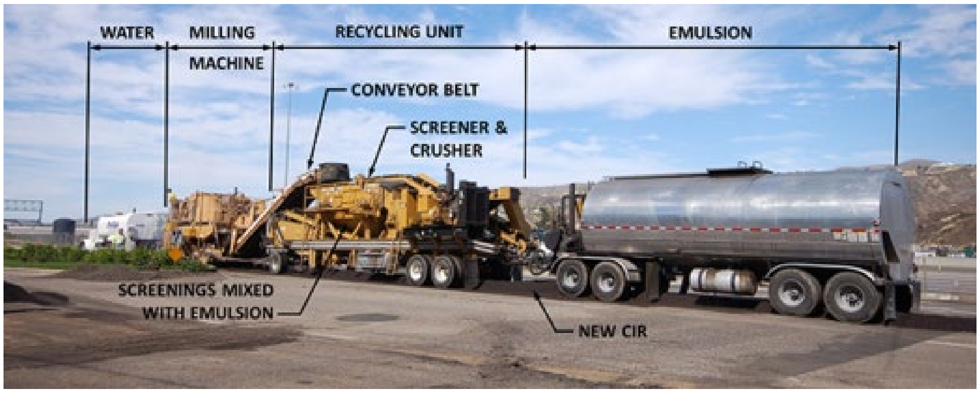

Figure 23. Cold in-place recycling train components.

The milling machine pulverizes the existing AC layer to the specified depth of the CIR process. The milled material is referred to as reclaimed asphalt pavement (RAP). The recycling unit crushes the RAP material to meet the gradation specification of the project and mixes the RAP material with the asphalt emulsion. Some projects apply full gradation specification while other projects may only apply a maximum size specification. The emulsion tank is connected to the recycling unit and delivers the amount of asphalt emulsion specified per the job mix formula (JMF). The final CIR mix is laid-down in a windrow behind the recycling unit.

Figure 24 shows the final two steps of the construction process where the CIR mix is picked up by the paver, laid-down to the specified thickness, and compacted by the rollers. The degree of compaction achieved in the CIR layer plays a major role in its long-term performance as a structural layer in the AC pavement. Recent studies indicate that in-place air voids between 12 and 15% are desired for good long-term CIR performance.[7]

Figure 24. The CIR mix is picked up by the paver and roller compacted.

[7]Sebaaly, P.E., Castro, J.A., Ayal, F., Carvajal, M., and Hajj, E.Y. (2018). "Development of Mix Design and Structural Design Procedures for Cold In-Place Recycling," Final Report No. P361-16-803, Nevada Department of Transportation, Carson City.

Materials specifications and mix design for the CIR layer are beyond the scope of this Guide. Table 18 summarizes the recommended compositions of the CIR mixtures to be used in North-Western Nevada. These recommendations have been established based on extensive research efforts conducted by the Pavement Engineering and Science Program at the University of Nevada, Reno.[8]

Table 18. Recommended Compositions of CIR Mixtures.

Asphalt Emulsion Type	Design Emulsion Content by Dry Weight of RAP	Lime Slurry by Dry Weight of RAP	In-Place Air Voids
CMS-2s Latex Modified Polymer Modified Rubber Modified	3.0–3.5%	4.5% Lime: 1.5% Water: 3.0%	Max. 15%

<u>*Layer Coefficient:*</u> for the North-Western region of Nevada, it is recommended to use $\boldsymbol{a_2 = 0.25}$. This layer coefficient is for a properly designed and constructed CIR layer (in-place air voids less than 15%).

<u>Subgrade Layer</u>

The property required for the subgrade layer is its engineering property. The AASHTO 1993 Guide defines the engineering property of subgrade material in terms of the *Mr*. As in the case of unbound materials, the *Mr* can be determined through direct laboratory measurement per AASHTO T 307 under repeated load triaxial conditions. This approach for determining the *Mr* property of subgrade material has proven to be highly complicated and expensive for highway agencies to conduct on a routine basis. Therefore, the majority of highway agencies, including Nevada DOT, rely on empirical relationships to determine the *Mr* of subgrade material from less complicated properties. For the design of flexible pavements in North-Western Nevada, it is recommended to determine the *Mr* property of subgrade materials from the following relationships:[9]

- For Subgrade layer with R-value > 20

$$Mr = e^{(3.1784+0.018\times R-value+0.0136\times P40+0.0315\times \gamma_d+0.0433\times PI)} \quad [5d]$$

- For Subgrade layer with R-value ≤ 20

$$Mr = 145\times 10^{(0.0147\times R\text{-}value\ +\ 1.23)} \quad [5e]$$

Where;

Mr: resilient modulus of subgrade material, psi
R-value: resistance value of subgrade determined in accordance with ASTM D2844
P40: percent passing sieve No. 40 (0.425 mm) determined in accordance with AASHTO T 88 and ASTM C136, %
γ_d: maximum dry density determined in accordance with ASTM D1557, pcf
PI: plasticity index determined in accordance with ASTM D4318

[8]Sebaaly, P.E., Castro, J.A., Ayal, F., Carvajal, M., and Hajj, E.Y. (2018). "Development of Mix Design and Structural Design Procedures for Cold In-Place Recycling," Final Report No. P361-16-803, Nevada Department of Transportation, Carson City.
[9]Sebaaly, P. E., Thavathurairaja, J., and Hajj, E.Y. (2018). "Characterization of Unbound Materials (Soils/Aggregates) for Mechanistic-Empirical Pavement Design Guide (MEPDG)," Final Report No. P361-16-803, Nevada Department of Transportation, Carson City, NV.

The design *R-value* for subgrade material should be based on laboratory testing of field samples obtained from various locations along the length of the project. Due to the embedded variability in R-value testing, it is strongly recommended that two R-value tests be conducted per sampling location at least every 1,000 feet (or less) along the project with no less than three sampling locations per project. Sampling for soil index properties and moisture-density relationships should be conducted at a higher frequency to determine the subgrade uniformity along the project length. Such information is used to select the soil samples for R-value testing.

The two R-value tests per location should be compared for reasonableness given the ASTM D2844 precision statement for R-value testing. If the test results from the same sampling location meet the ASTM precision, the two R-values should be averaged to determine the average R-value for that particular location. If the test results do not meet the ASTM precision, two new R-value tests from the same or a new adjacent location should be conducted.

It is recommended that the 50th percentile of the measured *R-values* be used as the design *R-value*. The 50th percentile defines the *R-value* that is exceeded for 50% of the measured values along the length of the project.

Careful consideration should be given to areas with poor subgrade quality as indicated by low *R-values*, high plasticity index (PI), high fines content, expansive clays, high moisture content, frost susceptibility, high organic content, collapsible soil, etc. In such cases, the following remedies should be considered:[10]

- Soils that are excessively expansive should receive special consideration. Generally, expansive soils have high plasticity indices, high percentages passing the #200 sieve, low *R-values*, and are A-6 and A-7 soils according to the AASHTO Soil Classification System. The expansion index determined in accordance with ASTM D4829-21 can be used as an indicator of the potential expansion in a soil. One solution may be to cover these soils with a sufficient layer of selected material to overcome the detrimental effects of expansion. In some cases, it may be more economical to treat expansive soils by stabilizing with a suitable admixture such as lime or cement. Another solution would be to over-excavate at least 3 feet, unless otherwise suggested by the geotechnical investigation, and replacing existing soils.
- Weak subgrade soils generally have *R-values* less than 20. The marginally poor subgrade soils (i.e., *R-value* between 10 and 20) may be made acceptable by using additional base layers. Poor subgrade soils may be treated with suitable stabilizer such as lime, cement, or asphalt. Figure 25 presents a stabilizer selection system based on the subgrade PI and the percent passing the #200 sieve. Based on figure 25, more than one stabilizer may be suitable for stabilization. The best stabilizer is always listed at the top. Once a stabilizer is selected for a particular subgrade, detailed tests should be performed to determine the appropriate percent of the stabilizer to be added. Additional considerations should be given to the climatic limitations that may restrict the use of a stabilizer. Table 19 summarizes general climatic limitations as well as construction safety precautions. In some instances, the use of geosynthetics like geogrid or geotextile may also be a suitable alternative. Additionally, the quality of the subgrade may be improved by blending the existing soil with a granular soil. If the soil with poor quality is in limited areas, it may be most

[10]Flexible Pavement Design Manual (February 2007). Regional Transportation Commission of Washoe County, Prepared by Sierra Transportation Engineers.

economically treated by over-excavating (at least 3 feet) and replacing with a selected material (e.g., sub-base material).

- In areas that have frost heave soils, additional non-frost susceptible base material will need to be placed. For example, a granular base with less than 5% passing #200 sieve with good drainage properties or permeable bituminous treated base could be used. The frost line depth at the project site should be obtained from Appendix Table R301.2(1) of the 2018 Northern Nevada Code Amendment document. Table 20 summarizes the frost line depths for various locations in northern Nevada. The frost line depth should be calculated from the top of the pavement structure. For example, the pavement design of a 5-inch AC surface layer on top of an 8-inch base layer in Washoe County needs to have 11 inches of additional non-frost susceptible base material in order to protect the existing frost heave soil (5 inches of AC + 8 inches of base + 11 inches of non-frost susceptible base = 24 inches).
- Problems with highly organic soils are related to their extremely compressible nature and are accentuated when deposits are extremely nonuniform. Local deposits, or those of relatively shallow depth, may be more economically excavated and replaced with suitable selected material. Problems associated with deeper and more extensive deposits may be alleviated by placing surcharge embankments for pre-consolidation that can include special provisions for rapid removal of water to hasten consolidation.
- Certain roadbed soils pose difficult problems during construction. These are primarily the cohesionless soils, which are readily displaced under construction equipment; and wet clay soils, which cannot be compacted at high water contents because of displacement under rolling construction equipment and require long periods of time to dry to a suitable water content. Remedies include blending with other soils or adding suitable stabilizer to sands to provide cohesion, or to clays to fasten drying or increasing shear strength; covering with a layer of more suitable selected material to act as a working platform for construction of the pavement; or use of a geosynthetic to provide additional stability.

Table 19. Climatic Limitations and Construction Safety Precautions.

Type of Stabilizer	Climatic Limitations	Construction Safety Precautions
Lime and Lime-Fly Ash	• Do not use with frozen soils. • Air temp. should be 40°F (5°C) and rising. • Complete stabilized base construction one month before first hard freeze. • Two weeks of warm to hot weather are desirable prior to fall and winter temperatures.	• Quicklime should not come in contact with moist skin. • Hydrated lime [Ca(OH)2] should not come in contact with moist skin for prolonged periods of time. • Safety glasses and proper protective clothing should be worn at all times.
Cement and Cement-Fly Ash	• Do not use with frozen soils • Air temperature should be 40°F (5°C) and rising. • Complete stabilized layer one week before first hard freeze.	• Cement should not come in contact with skin for prolonged periods of time. • Safety glasses and proper protective clothing should be worn at all times.
Asphalt	• Air temperature should be above 50°F (10°C) when using emulsions. • Air temp. should be 40°F (5°C) and rising when placing thin lifts of hot mixed asphalt concrete. • Hot, dry weather is preferred for all types of asphalt stabilization.	• Some cutbacks have flash fire points below 100°F (40°C). • Hot mix asphalt concrete temperatures may be as high as 325°F (175°C).

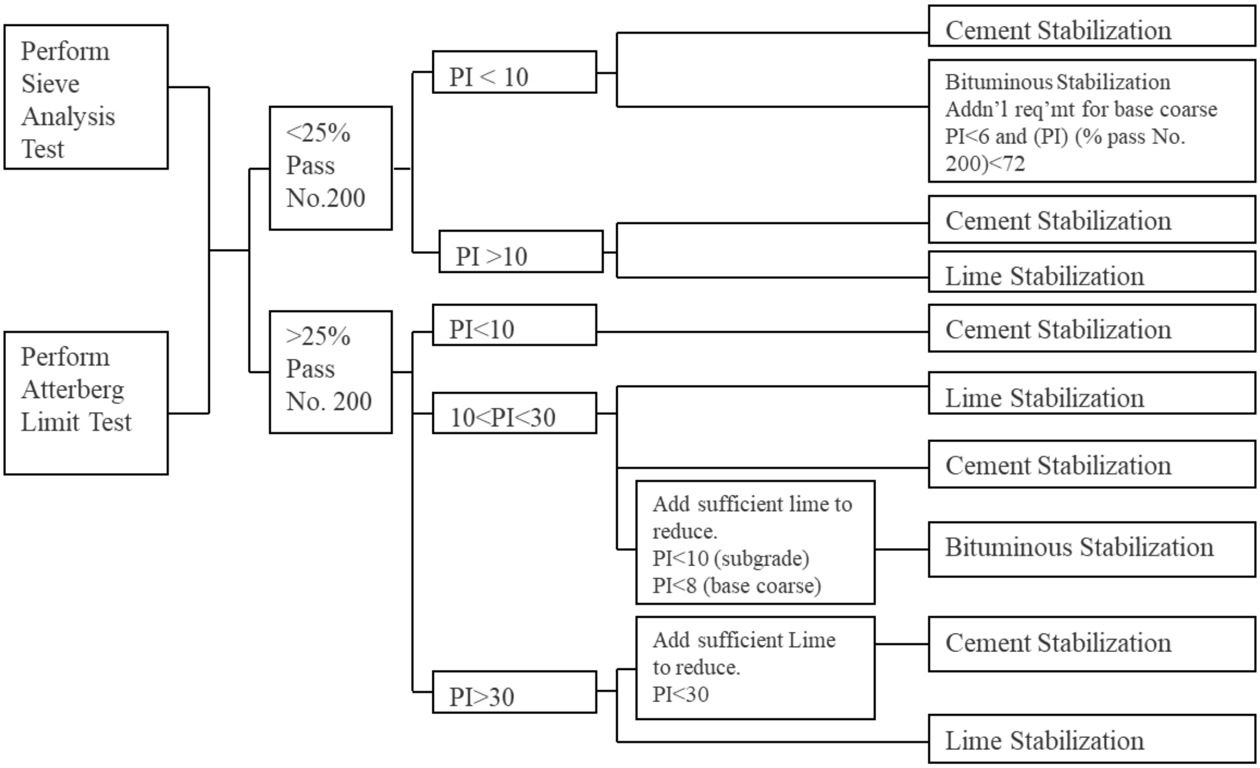

Figure 25. Selection of stabilizer.[11]

Table 20. Frost Line Depths.

Location	Frost Line Depth*
Carson City	24 inches
City of Fernley	18 inches
City of Reno	24 inches
City of Sparks	24 inches
Douglas County	18 inches for elevation < 6,000 ft 24 inches for elevation ≥ 6,000 ft
Washoe County	24 inches

*The frost line depth should be calculated from the top of the pavement structure.

5.2.e Environmental Impact

The AASHTO 1993 Guide incorporates the impact of the environment on the structural design of flexible pavements in terms of the effect of moisture on the engineering properties of unbound materials in the base and sub-base layers and subgrade material.

[11]Little, D.N., Thompson, M.R., Terrel, R.L., Epps, J.A., and Barenberg, E.J. (1987). "Soil Stabilization for Roadways and Airfileds," Final Report, Engineering Research Division, Engineering and Services Laboratory, Headquarters Air Force Engineering and Services Center (AFESC).

Unbound Layers

The impact of moisture on the engineering properties of unbound materials is incorporated into the structural design through the drainage coefficient "m." The basic concept of the drainage coefficient is to adjust the structural layer coefficient of the unbound base and sub-base layers based on the quality of the drainage system and the percent of time the pavement structure is exposed to moisture level approaching saturation. In summary, pavements in dry environment with good to excellent drainage systems will be assigned a m-value above 1.0 while pavements in wet environments with fair to poor drainage systems will be assigned a m-value below 1.0. Since the m-value is a direct multiplier to the structural layer coefficient, a value above 1.0 will increase the structural value of the unbound materials (i.e., reduce thickness) while a value below 1.0 will decrease the structural value (i.e., increase thickness) of the unbound layer.

The environment throughout North-Western Nevada can be classified as dry. Therefore, for pavements with expected drainage system ratings of good or better, the m-value should be ***1.0*** and for pavements with expected drainage systems rating of fair or worse, it should be **0.85**. The information presented in table 21 can be used to select the quality of the drainage system.

Table 21. Drainage System Quality and Related m-value Based on AASHTO 1993 Guide.

Quality of Drainage	Water Removed within	m-value
Excellent	2 hours	1.0
Good	1 day	1.0
Fair	2 week	0.85
Poor	1 month	0.85
Very Poor	Water will not drain	0.85

Subgrade Layer

The impact of moisture on the engineering properties of subgrade material is incorporated into the structural design through the effective roadbed modulus. The basic concept of the effective roadbed modulus is to estimate the relative pavement damage caused by the seasonal variations of the *Mr* property of the subgrade material. In summary, the higher moisture content in the subgrade during the wet season is expected to cause greater reduction in the *Mr* and generate more pavement damage. An effective *Mr* for the subgrade is determined while taking into consideration the anticipated relative pavement damage during each season. The AASHTO 1993 Guide uses the relationship in Equation 6 to estimate the pavement relative damage:

$$U_r = \frac{1.18 \times 10^8}{Mr^{2.32}} \qquad [6]$$

Where;

U_r: seasonal pavement relative damage

Mr: seasonal resilient modulus of subgrade material, psi

It is recommended to subdivide the year into the 4 seasons: spring, summer, fall, and winter; and use the seasonal adjustment factors developed for the local subgrade conditions in North-Western Nevada through a Nevada DOT research at the University of Nevada as summarized in table 22. The same study concluded that the *R-value* measured according to Nevada DOT T115D represents the subgrade layer condition during the spring season.

Table 22. Seasonal Adjustment Factors for Subgrade Material.[12]

Season	Seasonal Adjustment Factor
Spring	1.00
Summer	1.43
Fall	1.46
Winter	1.16

The following process can be followed to identify the effective *Mr* for the subgrade layer to be used in the structural design process:

1. Determine the 50th percentile *R-value* and other subgrade material properties (i.e., *P40*, γ_d, and *PI*)
2. Use the R-value determined in step 1 in Equation 5d to determine the *Mr* property.
3. Use the value from step 2 as the spring season *Mr* for the subgrade layer.
4. Adjust the *Mr* property for the summer, fall, and winter seasons using the factors in table 22 and the following relationship.

   Mr(season) = *Mr*(spring) × Seasonal Adjustment Factor (table 22)
5. For each season, use the determined *Mr* value in Equation 6 to calculate the seasonal pavement relative damage (U_r).
6. Determine the average damage, $U_{r(average)}$, by dividing the sum of U_r by four.
7. Use the average pavement relative $U_{r(average)}$ in equation 7 to determine the effective *Mr* of the subgrade layer ($Mr_{effective}$).

$$Mr_{effective} = 10^{\left[\left[log_{10}\left(\frac{U_{r(average)}}{1.18\times10^{8}}\right)\right]/(-2.32)\right]} \quad [7]$$

8. The $Mr_{effective}$ shall be used in the structural design process

Sample Calculation: The following is a sample calculation of the effective subgrade modulus:

1. The 50th percentile *R-value*: 66; *P40* = 63%; γ_d = 112.9 pcf; *PI* = 0
2. The *Mr* value from Equation 5d: 6,500 psi
3. The seasonal Mr properties of the subgrade layer: *Mr* × seasonal adjustment factor

   Mr(spring): 6,500 psi
   Mr(summer): 9,300 psi
   Mr(fall): 9,500 psi
   Mr(winter): 7,550 psi
4. The seasonal relative damage from Equation 6

   $U_{r(spring)}$: 0.17

[12] Hajj, E.Y., Sebaaly, P.E., Piratheepan, M., and Nabhan, P. (2019). "Manual for Designing Flexible Pavements in Nevada Using the AASHTOWare™ Pavement ME," Report No. WRSC-201504, Nevada Department of Transportation, Carson City.

$U_{r(summer)}$: 0.07
$U_{r(fall)}$: 0.07
$U_{r(winter)}$: 0.12

5. The average relative damage:

$$U_{r(average)} = (0.17+0.07+0.07+0.12)/4 = 0.11$$

6. The effective *Mr* for the subgrade layer from Equation 7:

$$Mr_{effective} = 10^{\left(log_{10}\left(\frac{0.11}{1.18\times10^{8}}\right)/(-2.32)\right)} = 7{,}810 \text{ psi}$$

5.2.f Design of New Pavement

A new flexible pavement is defined as a pavement structure to be designed over the natural subgrade layer with either a 3-layer or 4-layer system. A 3-layer flexible pavement consists of: subgrade, base, and surface layers while a 4-layer system consist of: subgrade, base, sub-base, and surface layers.

The AASHTO 1993 Guide is based on the Structural Number (SN) concept to determine the amount of protection required for each layer starting with the subgrade layer and working up through the base layer. Figure 26 presents the SN concept for the design of new flexible pavements.

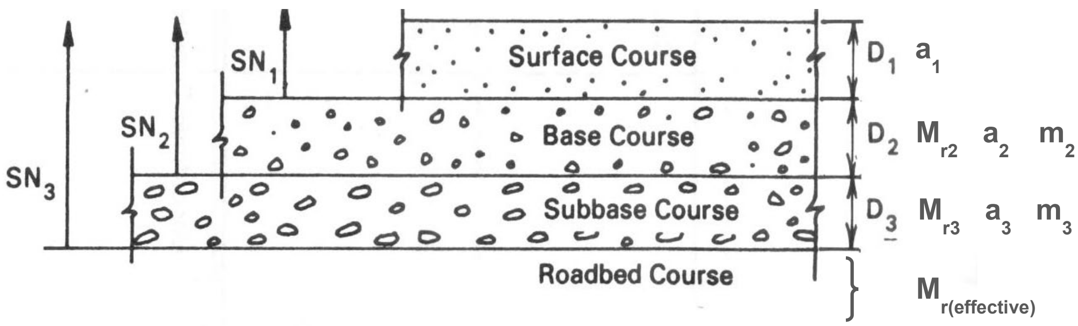

Figure 26. Structural Number concept for the design of flexible pavements.

The SN_1, SN_2, and SN_3 represent the structural capacity required to protect the base, sub-base, and subgrade layers, respectively. The SN's are related to the properties of the various layers as shown in Equations 8, 9, and 10:

$$SN_1 = a_1D_1 \quad [8]$$

$$SN_2 = a_1D_1 + a_2m_2D_2 \quad [9]$$

$$SN_3 = a_1D_1 + a_2m_2D_2 + a_3m_3D_3 \quad [10]$$

Where;

a_1: structural layer coefficient of surface AC layer = 0.35
a_2: structural layer coefficient of unbound base layer = 0.10 (Type 1, Class A) or 0.12 (Type 2, Class B)
a_3: structural layer coefficient of unbound sub-base layer = 0.07
m_2: drainage coefficient of unbound base layer = 1.0 or 0.85
m_3: drainage coefficient of unbound sub-base layer = 1.0 or 0.85
D_1, D_2, D_3: thickness of surface, base, and sub-base layers, respectively (inch)

At this stage of the structural design, all factors are known expect the SN_1, SN_2, and SN_3. The SN values can be determined from Equation 11 through an iterative process, or through by using the nomograph presented in figure 27.

$$\log_{10}(W_{18}) = Z_R S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{2.7}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(Mr) - 8.07 \qquad [11]$$

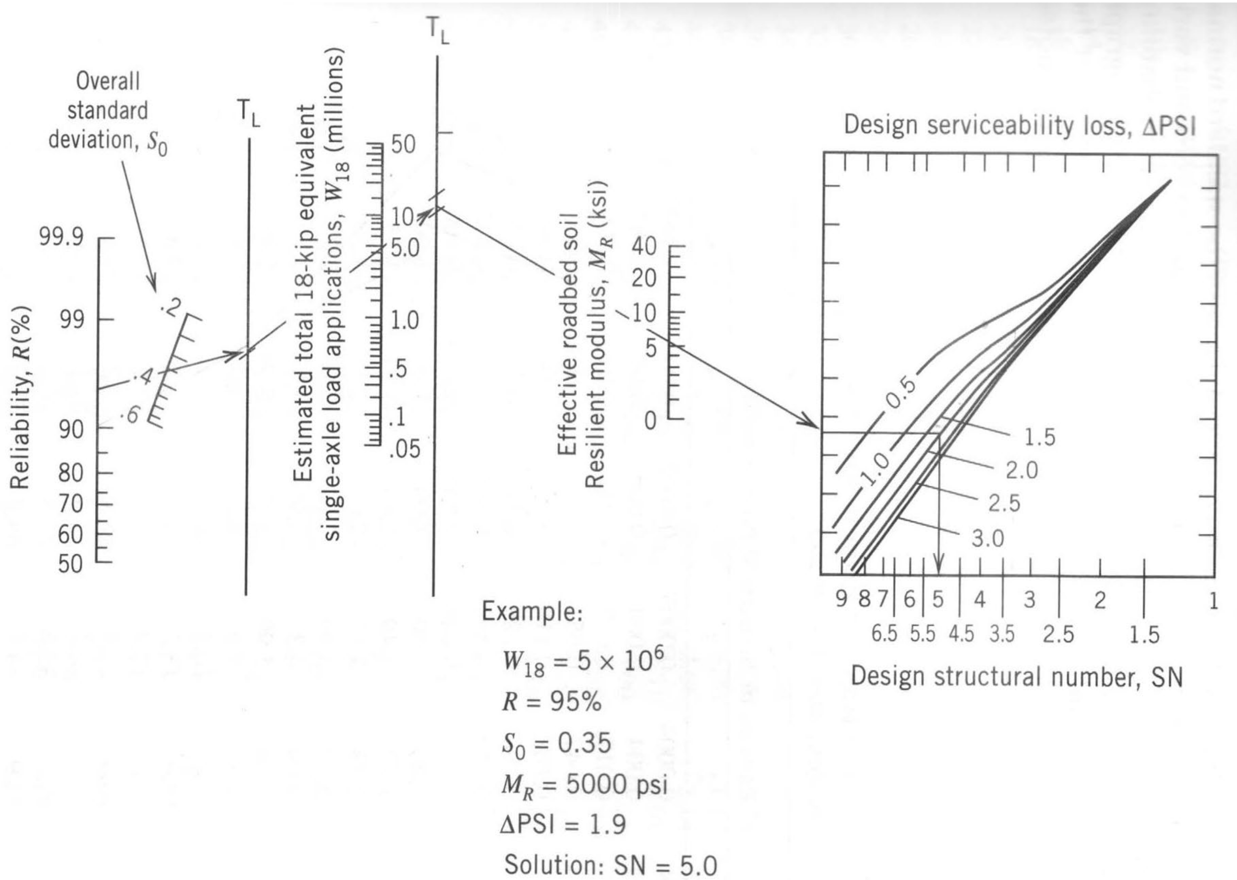

Figure 27. AASHTO 1993 Design Guide nomograph.

The following process is used to determine the SN values:

1. Determine the change in PSI (ΔPSI) from Table 14.
2. Select the desirable level for Reliability from table 15 and use an Overall Standard Deviation of 0.45.
3. Determine the design traffic loads (W_{18}) following the process outlined in Chapter 3 and Equation 4.
4. Determine the *Mr* properties for the unbound layers, for example;
   a. *Mr* for Base = 16,100 psi
   b. *Mr* for Sub-base = 8,500 psi

5. Determine the effective *Mr* property of the subgrade layer as described in Section 4.2.c
6. Enter the nomograph with the following information:
   a. Reliability, *R*
   b. Overall Standard Deviation, S_0
   c. Design Traffic, W_{18}
7. On the scale labeled "Effective Roadbed Soil Resilient Modulus":
   a. Enter the value of the effective *Mr* of the subgrade layer (i.e., $Mr_{effective}$) in ksi to obtain SN_3
   b. Enter the value of the *Mr* for the sub-base layer of 8.5 ksi to obtain SN_2
   c. Enter the value of the *Mr* for the base layer of 16.1 ksi to obtain SN_1
8. Move horizontally to the appropriate curve for the selected change in PSI (ΔPSI) and drop vertically to identify the SN value.
9. Once the SN values are determined, use Equations 8, 9, and 10 to solve for the thicknesses of the surface, base, and sub-base layers:
   a. Start with the SN_1 equation; solve for D_1 and round-up to nearest 0.5 inch
   b. Use the actual value of D_1 in the SN_2 equation to solve for D_2 and round-up to nearest 0.5
   c. Use the actual values of D_1 and D_2 in the SN_3 equation to solve for D_3 and round-up to the nearest 0.5 inch

Design Example

Develop the structural design of a new flexible pavement with the following characteristics:

- Facility classification: Arterial with 2-lanes/direction
- Design life: 20 years
- $AADT_1$: 1,350 vehicle per day (vpd)
- Truck percentage: 15%
- Truck Factor: 1.21
- Traffic Growth Rate: 3%
- Subgrade 50th percentile R-Value: 66; *P40* = 63%; γ_d = 112.9 pcf; *PI* = 0
- Good drainage system

The following steps are followed to determine the required pavement structure:

1. ΔPSI: 1.7 (table 14)
2. Reliability, *R*: 90% and S_0: 0.45 (table 15)
3. Determine Design Lane Traffic, W_{18}:
   a. Traffic Growth Factor: $G = [(1+r)^Y - 1]/r = [(1+0.3)^{20} - 1]/0.3 = 27$
   b. Design ESALs = $AADT_1 \times 365 \times T \times T_f \times G$
      Design ESALs = $1{,}350 \times 365 \times 0.15 \times 1.21 \times 27 = 2{,}414{,}722$
   c. W_{18} = Design ESALs × D × L = 2,414,722 x 0.5 x 0.9 = 1,100,000 (rounded-up to nearest 100k)
4. *Mr* for base: 16,100 psi

5. Effective *Mr* property of the subgrade layer is determined as described in Section 4.2.c: 7,810 psi

6. Enter the Nomograph with the following information:

    a. *R*: 90%
    b. S_o: 0.45
    c. W_{18}: 1.1 million

    NOTE: the use of the Nomograph is not very repeatable due to approximations involved in the various scales and at the turning lines, it is highly recommended the PaveXpress software be incorporated with this Guide to determine SN values. PaveXpress can be accessed free of charge at http://www.pavexpressdesign.com/.

7. On the scale labeled "Effective Roadbed Soil Resilient Modulus":

    a. Enter the value of the effective *Mr* of the subgrade layer as 7.8 ksi to obtain SN_2
    b. Enter the value of the *Mr* for the base layer of 16.1 ksi to obtain SN_1

8. After each step of 7.a and 7.b move horizontally from the vertical axis until the curve of ΔPSI = 1.7 *(interpolate between curves for $\Delta PSI = 1.5$ and 2.0)*:

    a. Using the Effective *Mr* of the subgrade layer of 7.8 ksi, the determined $SN_2 = 3.47$
    b. Using the *Mr* of the base of 16.1 ksi, the determined $SN_1 = 2.63$

9. ***Design a 3-layer Pavement Structure.*** Solve for the thicknesses of the surface, base, and sub-base layers:

<u>Thickness of AC layer</u>

$$SN_1 = a_1 D_1$$

$$D_1 = SN_1/a_1 = 2.63/0.35 = 7.5 \text{ inch}$$

<u>Thickness of Base layer</u>

$$SN_2 = a_1 D_1 + a_2 m_2 D_2$$

$$D_2 = (SN_2 - a_1 D_1)/(a_2 m_2) = (3.47 - 0.35\times 7.5)/(0.1\times 1.0) = 8.5 \text{ inch}$$

NOTE: the thickness of the layers used in the calculations of the subsequent layers are the round-up values.

Assuming that this new pavement design was required for an existing flexible pavement, thus allowing for the use of a cement-treated recycled aggregate base layer instead of importing a new base layer. Supposing the existing pavement consisted of a 4-inch AC layer on top of a 6-inch base layer on top of the subgrade. The existing AC layer with the full base layer are pulverized and blended with cement at the dosage determined from the mix design. An unconfined compressive strength of 300 psi was measured at the select cement content. The following steps are followed to determine the required pavement structure for the same project presented above:

1. Same as above.
2. Same as above.
3. Same as above.
4. *Mr* for stabilized base: 120×300 + 9980 = 45,980 psi assume 46,000 psi

5. Same as above.
6. Same as above.
7. On the scale labeled “Effective Roadbed Soil Resilient Modulus”:
    a. Enter the value of the effective *Mr* of the subgrade layer as 7.8 ksi to obtain SN_2
    b. Enter the value of the *Mr* for the stabilized base layer of 46 ksi to obtain SN_1
8. After each step of 7.a and 7.b move horizontally from the vertical axis until the curve of $\Delta PSI = 1.7$ *(interpolate between curves for $\Delta PSI = 1.5$ and 2.0)*:
    a. Using the Effective *Mr* of the subgrade layer of 7.8 ksi, the determined $SN_2 = 3.47$
    b. Using the *Mr* of the stabilized base of 46 ksi, the determined $SN_1 = 1.75$
9. ***Design a 3-layer Pavement Structure.*** Solve for the thicknesses of the surface, base, and sub-base layers:

Thickness of AC layer

$$SN_1 = a_1D_1$$

$$D_1 = SN_1/a_1 = 1.75/0.35 = 5.0 \text{ inch}$$

Thickness of Stabilized Base layer

$$SN_2 = a_1D_1 + a_2m_2D_2$$

$$D_2 = (SN_2 - a_1D_1)/(a_2m_2) = (3.47 – 0.35\times5.0)/(0.18\times1.0) = 9.6 \text{ inch use 10 inches}$$

NOTE: the thickness of the layers used in the calculations of the subsequent layers are the round-up values.

PaveXpress is a free web-based pavement design tool based on the AASHTO 93 and 98 design standards for flexible and rigid pavements. The software includes modules for both new construction and rehabilitation (overlay as well as coldmill and overlay) flexible pavement designs. Since its release in 2015, it has been enhanced to include the AASHTO layered design analysis to ensure proper layer support for varying base materials; AASHTO-recommended defaults for design parameters, such as reliability and serviceability to streamline the design process; enhanced traffic input methods; and enhanced printable reports with additional detail, as well as updated and expanded help and FAQs. It also includes a Life-Cycle Cost Analysis (LCCA) module for comparison of design options. PaveXpress can be accessed on any internet-connected device at ***http://www.pavexpressdesign.com/*** and requires no software downloads or licenses. Developed pavement designs can also be saved for later reference. ***It is highly recommended that all flexible pavement designs be performed using the PaveXpress software, rather than hand calculations or Excel spreadsheets.***

5.2.g Design of Pavement Rehabilitation

Pavement rehabilitation consists of two major categories; I. Structural Overlay and II. Cold In-place Recycling. Both categories are used to correct major structural deficiencies in existing flexible pavements. Structural Overlay may also integrate cold milling prior to overlay, which must be accounted for in the overlay thickness determination.

Pavement Evaluations

The design of the two major rehabilitations requires the evaluation of the structural capacity of the existing pavement. The AASHTO 1993 Guide uses the concept of effective structural number, SN_{eff}, to define the structural capacity of an existing pavement, which can be determined through three different approaches described below.

SN_{eff} based on Visual Survey: this approach uses pavement condition survey data to estimate the structural value of the existing pavement in terms of its SN_{eff} as follows:

1. Use the thicknesses of the various layers in the existing pavement.
2. Adjust the coefficients for the AC, base, and sub-base layers based on the visual condition survey data and following the recommendations listed in table 23.
3. Calculate the SN_{eff} of the existing pavement from Equation 12:

$$SNeff = a_1^*D_1 + a_2^*m_2D_2 + a_3^*m_3D_3 \qquad (12)$$

Where;

a_1^*, a_2^*, a_3^*: coefficients for AC, base, and sub-base layers adjusted based on table 23
D_1, D_2, D_2: thickness of AC, base, and sub-base layers
m_2, m_3: drainage coefficient for base and sub-base layers

The thickness of the existing layers can be obtained from field coring or construction records. The drainage coefficients of the base and sub-base layers may be adjusted to reflect any damage in the drainage system that has occurred during the life of the existing pavement.

Table 23. Adjusted Layer Coefficients Based on Condition Surveys.

Layer	Surface Condition	Coefficient
AC Surface	Little or no alligator cracking and/or only low severity transverse cracking	$a_1^* = 0.30$
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	$a_1^* = 0.20$
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5 percent medium- and high-severity transverse cracking	$a_1^* = 0.15$
	>10 percent medium-severity alligator cracking and/or <10 percent high-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	$a_1^* = 0.10$
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	$a_1^* = 0.05$
Base	No evidence of pumping, degradation, or contamination by fines	$a_2^* = 0.10$
	Some evidence of pumping, degradation, or contamination by fines	$a_2^* = 0.05$
Sub-base	No evidence of pumping, degradation, or contamination by fines	$a_3^* = 0.07$
	Some evidence of pumping, degradation, or contamination by fines	$a_3^* = 0.04$

SN_{eff} based on Remaining Life: this approach assumes that the existing flexible pavement has a certain level of remaining life (RL) which is directly related to its original structural number (SN_o). The SN_o is calculated per Equations 9 or 10 using the thicknesses and layers coefficients of the original pavement structure. The RL factor is determined from Equation 13:

$$RL = 100\,[1 - (N_p/N_{1.5})] \qquad [13]$$

Where;

RL: Remaining life, %

N_p: Total applied ESALs to date

$N_{1.5}$: Total ESALs to a terminal PSI (P_t) of 1.5

The N_p is obtained from actual traffic loads applied to the pavement up to date and the $N_{1.5}$ is determined from the AASHTO 1993 Design with $\Delta PSI = 4.2 - 1.5 = 2.7$ using the SN_o of the pavement with all other design data remaining constant. The SN_{eff} is calculated from Equation 14:

$$SN_{eff} = CF \times SN_o \qquad [14]$$

Where, CF: Correction factor obtained from figure 28.

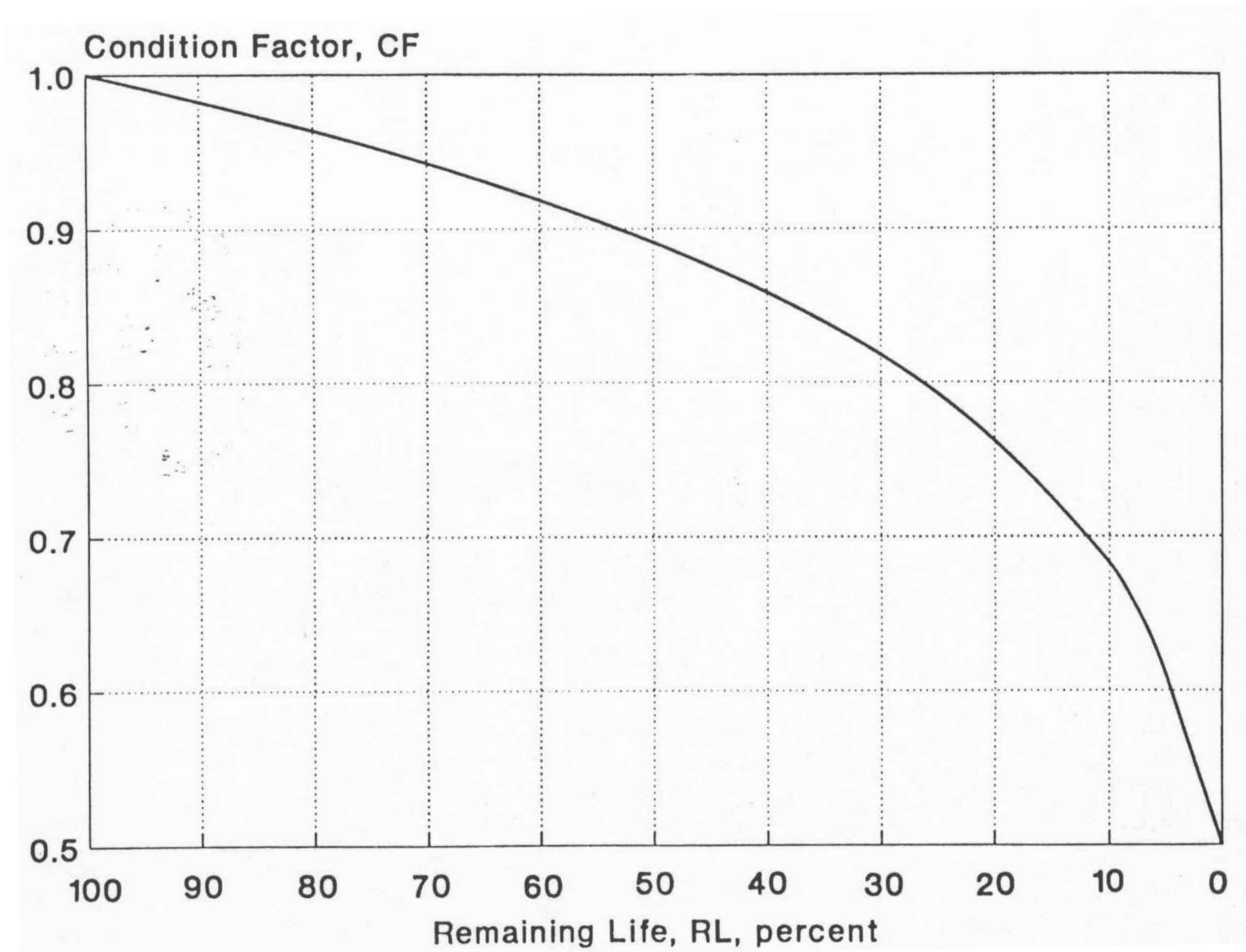

Figure 28. Correction factor for original pavement structural capacity based on remaining life.

SN_{eff} based on Non-Destructive Testing (NDT): this approach requires the conduct of NDT to evaluate the in-situ properties of the existing pavement structure. The use of the falling weight deflectometer (FWD) is recommended for the NDT testing. Once the NDT is conducted, the effective modulus of the existing pavement structure, E_p, is determined from the relationship presented in figure 31 as follows:

1. Evaluate the R-value (laboratory testing) of the subgrade materials throughout the project length and determine the 50th percentile R-value.
2. Determine the effective resilient modulus of the subgrade layer as described in Section 4.2.d and label it as M_R.
3. Conduct FWD testing throughout the project length using a loading plate with a radius of 5.9 inch and determine the average surface deflection over the length of project under the center of the FWD plate as d_0. It is highly recommended to conduct the FWD testing when the ambient air temperature is in the range of 55 to 80°F.
4. Determine the value of $M_R \times d_0/P$, where the M_R is the effective resilient modulus of the subgrade in psi determined in step 2, the d_0 is the average vertical surface deflection under the center of the FWD loading plate in mils (0.001 inch) adjusted to 68°F (figure 29 and figure 30), and P is the FWD load in lbs.
5. Determine the total pavement thickness (D) as the sum of the layer thicknesses above the subgrade in inches.
6. Enter the chart in figure 31 with the determined values from steps 4 and 5 to identify the appropriate ratio of E_p/M_R.
7. Knowing the M_R value from step 2, calculate the E_p of the existing pavement structure as: $E_p = (E_p/M_R) \times M_R$ in psi.
8. The SN_{eff} of the existing pavement structure is calculated per Equation 15:

$$SN_{eff} = 0.0045\ D\ (E_p)^{1/3} \qquad [15]$$

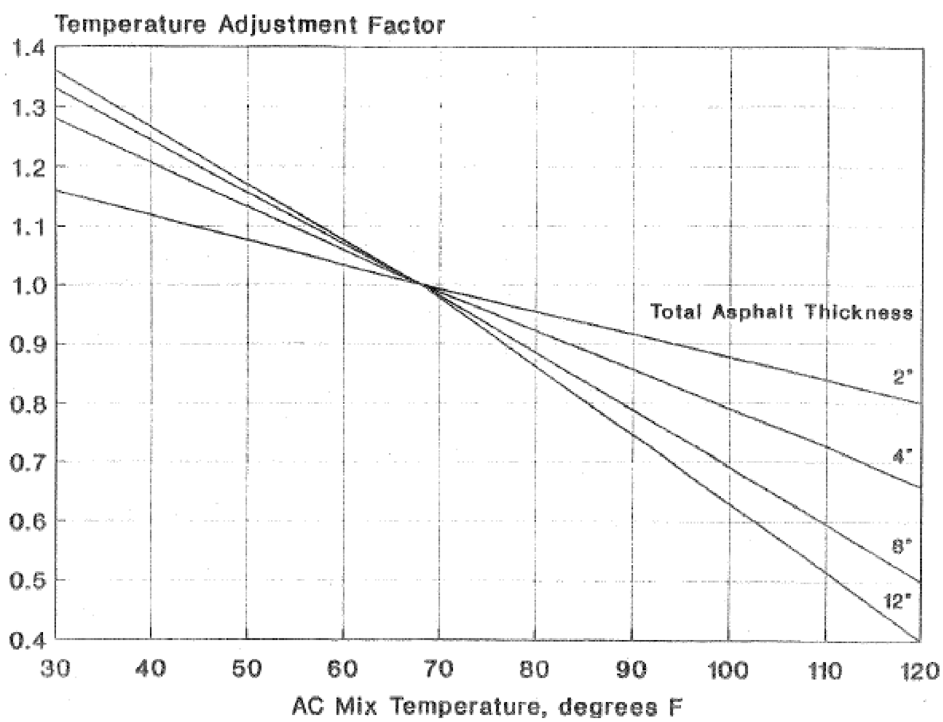

Figure 29. d_0 adjustment for AC mixture temperature for aggregate base and asphalt-treated base pavements.

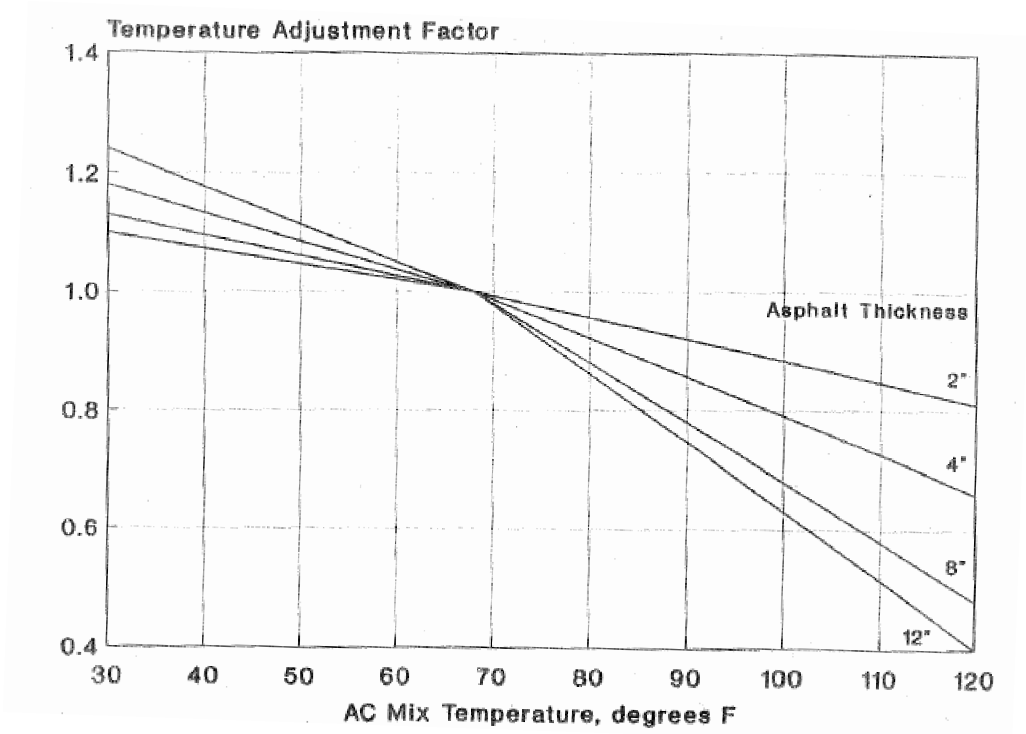

Figure 30. d_0 adjustment for AC mixture temperature for cement-treated base pavements.

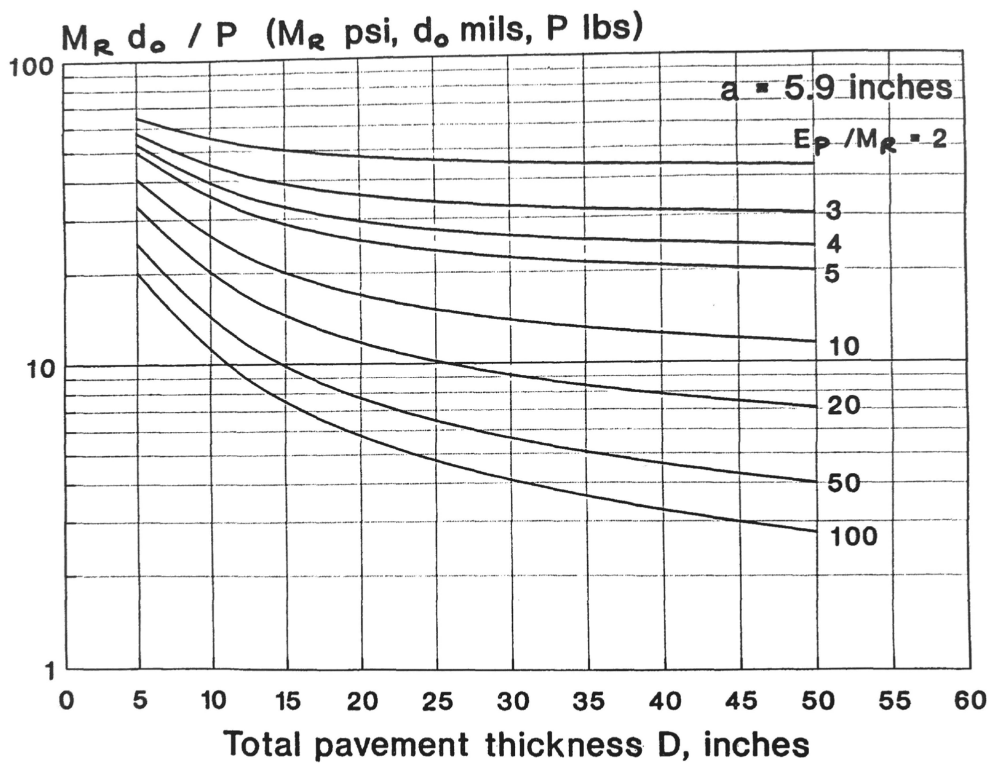

Figure 31. Determination of the effective modulus for the existing pavement structure.

Design of Asphalt Concrete Overlay

As discussed in Chapter 2, the application of AC overlay over AC pavement is most effective when the existing pavement is not showing any type of cracking above the low severity level. All other distresses can be effectively addressed by a structural AC overlay, i.e.; bleeding, raveling, rutting, and shoving. In cases where the AC overlay is desired over cracked AC pavement, special attention must be taken in order to reduce the potential for reflective cracking as discussed in Section 2.2.a.

The AASHTO 1993 Guide uses the concept of SN_{eff} to determine the required thickness of the structural AC overlay as presented in Equations 16 and 17:

$$SN_{OL} = SN_f - SN_{eff} \qquad [16]$$

$$D_{OL} = SN_{OL}/a_{OL} \qquad [17]$$

Where;
- SN_{OL}: Structural number of the AC overlay
- SN_f: Structural number for design future traffic
- SN_{eff}: Effective structural number of the existing pavement
- D_{OL}: thickness of the AC overlay, inch (round-up to the nearest 0.5 inch)
- a_{OL}: layer coefficient of the AC overlay; 0.35

The SN_f is determined from the AASHTO Guide using the applicable design parameters along with design traffic (Section 3.1.b) and effective subgrade modulus (Section 4.2.d). The SN_{eff} is determined from any one of the three methods presented in the previous section (Equations 12, 14, 15). The layer coefficient of the AC overlay is assumed to be the same as for the AC layer in the design of new flexible pavements (i.e., 0.35).

As it can be observed from the design example presented in the next section, the three methods of determining SN_{eff} lead to AC overlay thickness ranging between 2.0 and 3.0 inches. This level of variation is expected since all three methods are empirical and involve numerous assumptions. It is recommended that the users of this Guide determine the SN_{eff} in the following order of priority:

- Option 1: Based on NDT
- Option 2: Based on Remaining Life
- Option 3: Based on Visual Survey

Design Example: the following calculations present an example design of AC overlay over existing flexible pavement.

- Facility classification: Arterial with 2-lanes/direction
- Existing Pavement:
  - AC layer: 6 inch
  - Base Layer: 8 inch
  - Subbase: 10 inch
  - Average FWD center deflection, d_0 at 68°F: 55 mils and FWD Load, P: 10,000 lb
  - Little or no alligator cracking and/or only low severity transverse cracking
  - Evidence of degradation in the base and subbase layers
- Overlay Design life: 20 years
- AADT: 1,350 vpd

- Truck percentage: 15%
- Truck Factor: 1.21
- Traffic Growth Rate: 3%
- Subgrade 50th percentile R-Value: 66; $P40$ = 63%; γ_d = 112.9 pcf; PI = 0
- Good drainage system

The following steps are followed to determine the required AC overlay thickness:

1. ΔPSI: 1.7 (table 14)
2. Reliability, R: 90% and S_0: 0.45 (table 15)
3. Determine Design Lane Traffic, W_{18}:
   a. Traffic Growth Factor: $G = [(1+r)^Y - 1]/r = [(1+0.3)^{20} - 1]/0.3 = 27$
   b. Design ESALs = $AADT_1 \times 365 \times T \times T_f \times G$
      Design ESALs = 1,350 × 365 × 0.15 × 1.21 × 27 = 2,414,722
   c. W_{18} = Design ESALs × D × L = 2,414,722 × 0.5 × 0.9 = 1,100,000 (rounded-up to nearest 100k)
4. The effective Mr property of the subgrade layer is determined as described in Section 4.2.d: 7,810 psi
5. Enter the Nomograph with the following information:
   a. R: 90%
   b. S_0: 0.45
   c. W_{18}: 1.1 million
   d. On the scale labeled "Effective Roadbed Soil Resilient Modulus" enter the value of the effective Mr of the subgrade layer as 7.8 ksi to obtain SN_f = 3.50
   e. Determine the SN_{eff} of the existing pavement:
      i. Based on Visual Survey:
         $SN_{eff} = a_1^* D_1 + a_2^* m_2 D_2 + a_3^* m_3 D_3$
         From table 23; a_1^* = 0.30, a_2^* = 0.05, a_3^* = 0.04
         Drainage is Good: $m_2 = m_3$ = 1.0
         SN_{eff} = 0.30 × 6.0 + 0.05 × 1.0 × 8.0 + 0.04 × 1.0 × 10.0 = 2.60
      ii. Based on Remaining Life:
         SNeff = CF × SN_o
         $SN_o = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3$
         For original pavement design; a_1 = 0.35, a_2 = 0.1, a_3 = 0.07, $m_2 = m_3$ = 1.0
         SN_o = 0.35 × 6.0 + 0.1 × 1.0 × 8.0 + 0.07 × 1.0 × 10.0 = 3.60
         Total applied traffic to date (N_p) = 1.7 million
         Determine $N_{1.5}$ from the AASHTO Guide (ΔPSI = 2.7); $N_{1.5}$ = 2.5 million
         RL = 100 [1 – ($N_p/N_{1.5}$)] = 100 [1 – (1.7/2.5)] = 32%
         From Figure 21, CF = 0.81
         SNeff = CF × SN_o = 0.81 × 3.60 = 2.92
      iii. Based on NDT:
         Determine $M_R \times d_0/P$ = 7,810 × 55/10,000 = 43
         Total pavement thickness, $D = D_1 + D_2 + D_3$ = 6.0 + 8.0 + 10.0 = 24.0 inch

From Figure 31, $E_p/M_R = 2.3$
$E_p = (E_p/M_R) \times M_R = 2.3 \times 7{,}810 = 17{,}963$
$SN_{eff} = 0.0045 \times D \times (E_p)^{1/3} = 0.0045 \times 24 \times (17{,}963)^{1/3} = 2.83$

f. Determine the required thickness of the AC overlay:

i. Based on Visual Survey:

$SN_{OL} = SN_f - SN_{eff}$

$SN_{OL} = 3.50 – 2.60 = 0.90$

$D_{OL} = SN_{OL}/a_{OL} = 0.90/0.35 = 2.57$ inch

After rounding-up to the nearest 0.5 inch, $D_{OL} = 3.0$ inch

ii. Based on Remaining Life:

$SN_{OL} = SN_f - SN_{eff}$

$SN_{OL} = 3.50 – 2.92 = 0.58$

$D_{OL} = SN_{OL}/a_{OL} = 0.58/0.35 = 1.66$ inch

After rounding-up to the nearest 0.5 inch, $D_{OL} = 2.0$ inch

iii. Based on NDT:

$SN_{OL} = SN_f - SN_{eff}$

$SN_{OL} = 3.50 – 2.83 = 0.67$

$D_{OL} = SN_{OL}/a_{OL} = 0.67/0.35 = 1.91$ inch

After rounding-up to the nearest 0.5 inch, $D_{OL} = 2.0$ inch

Design of AC Mill and Overlay

It is common to coldmill an existing flexible pavement prior to overlay to eliminate near surface distress and so existing geometric constraints can be matched. For example, with coldmilling prior to overlay, bridge clearances are maintained. Similarly, where curb and gutter or other draining inlet structures exist, coldmilling prior to overlay allows for matching to the original construction grades. Typical coldmilling depths are 2–4 inch.

The structural design of an AC mill and overlay layer follows a similar process to the design of AC overlay over flexible pavement with the following exceptions:

1. No adjustment need be made to SN_{eff} values determined by NDT if the depth of milling does not exceed the minimum necessary, to remove surface ruts.
2. If a greater depth is milled, the NDT-determined SN_{eff}, may be reduced by an amount equal to the depth milled times a structural coefficient for the AC surface based on the visual condition survey (table 23).

The structural design of an AC mill and overlay follows a similar process to the design of AC overlay over flexible pavement with the following exceptions.

a. ***Based on Visual Survey:***

The SN_{eff} is calculated from Equation 18:

$$SN_{eff} = a_1^*(D_1 - D_{AC\text{-}mill}) + a_2^* m_2 D_2 + a_3^* m_3 D_3 \qquad [18]$$

Where, $D_{AC\text{-}mill}$ is the thickness of the milled AC layer, inch.

b. ***Based on Remaining Life:***

The SN_o used for the determination of $N_{1.5}$ considers the entire thickness of the existing AC layer along with the thicknesses of the other layers

The RL factor is determined from Equation 13

The SN_o used in the calculation of the SN_{eff} considers the thickness of the existing AC layer less the depth of the milled AC layer as calculated per Equation 19:

$$SN_0 = a_1 \times (D1 - D_{AC\text{-}mill}) + a_2 m_2 D_2 + a_3 m_3 D_3 \qquad [19]$$

Where; $D_{AC\text{-}mill}$ is the thickness of the milled AC layer, inch.

c. ***Based on NDT:***

The SN_{eff} is calculated from Equation 20:

$$SN_{eff} = 0.0045 \times (D) \times (E_p)^{1/3} - (a_1^* D_{AC\text{-}mill}) \qquad [20]$$

Where, a_1^* is determined from table 23, and $D_{AC\text{-}mill}$ is the thickness of the milled existing AC layer

Design Example: the following calculations present a design of AC mill and overlay.

All the information from the design example of AC overlay over AC pavement remains the same with the exception that the top 3 inches of the existing AC layer will be milled.

1. The depth of the AC milling layer, $D_{AC\text{-}mill}$ = 3.0 inch
2. Establishing the SN_{eff} of the existing pavement based on visual survey:

   From table 23; a_1^*= 0.30, a_2^*= 0.05, a_3^*= 0.04

   Drainage is Good: m_2=m_3= 1.0

   Per Equation 18,

   $$SN_{eff} = a_1^* \times (D_1 - D_{AC\text{-}mill}) + a_2^* m_2 D_2 + a_3^* m_3 D_3$$

   $$SN_{eff} = 0.30 \times (6.0 - 3.0) + 0.05 \times 1.0 \times 8.0 + 0.04 \times 1.0 \times 10.0 = 1.70$$

3. Establishing the SN_{eff} of the existing pavement based on Remaining Life:

   The RL factor is determined from Equation 13: 32%

   The CF is obtained from Figure 21: 0.81

   The SN_o is calculated per Equation 19:

   $$SN_o = a_1 \times (D1 - D_{AC\text{-}mill}) + a_2 m_2 D_2 + a_3 m_3 D_{33}$$

   $$SN_o = 0.35 \times (6.0 - 3.0) + 0.1 \times 1.0 \times 8.0 + 0.07 \times 1.0 \times 10.0 = 2.55$$

The SN_{eff} is calculated:

$$SN_{eff} = CF \times SN_o$$

$$SN_{eff} = 0.81 \times 2.55 = 2.07$$

4. Establishing the SN_{eff} of the existing pavement based on NDT:

   From table 23; $a_1^* = 0.30$

   The SN_{eff} is calculated from Equation 20:

$$SN_{eff} = 0.0045 \times (D) \times (E_p)^{1/3} - a_1^* D_{AC\text{-}mill}$$

$$SN_{eff} = 0.0045 \times (24.0)\ (17{,}963)^{1/3} - 0.3 \times 3.0 = 1.93$$

5. Determining the required thickness of the AC overly:

   i. Based on Visual Survey:

$$SN_{OL} = SN_f - SN_{eff}$$

$$SN_{OL} = 3.50 - 1.70 = 1.80$$

$$D_{OL} = SN_{OL}/a_{OL} = 1.80/0.35 = 5.14 \text{ inch}$$

   After rounding-up to the nearest 0.5 inch, $D_{OL} = 5.5$ inch

   ii. Based on Remaining Life:

$$SN_{OL} = SN_f - SN_{eff}$$

$$SN_{OL} = 3.50 - 2.07 = 1.43$$

$$D_{OL} = SN_{OL}/a_{OL} = 1.43/0.35 = 4.09 \text{ inch}$$

   After rounding-up to the nearest 0.5 inch, $D_{OL} = 4.5$ inch

   iii. Based on NDT:

$$SN_{OL} = SN_f - SN_{eff}$$

$$SN_{OL} = 3.5 - 1.93 = 1.57$$

$$D_{OL} = SN_{OL}/a_{OL} = 1.57/0.35 = 4.49 \text{ inch}$$

   After rounding-up to the nearest 0.5 inch, $D_{OL} = 4.5$ inch

<u>Design of AC Overlay over Cold In-Place Recycling</u>

The structural design of an AC overlay over the CIR layer follows a similar process to the design of AC overlay over flexible pavement with the following exceptions.

1. The coefficient for the CIR layer is: 0.25
2. Establishing the SN_{eff} of the existing pavement:
   a. ***Based on Visual Survey:*** this method is not applicable since the existing AC layer will be recycled and the majority of the distresses will be removed

*b. **Based on Remaining Life:***

The SN_o used for the determination of $N_{1.5}$ considers the entire thickness of the existing AC layer along with the thicknesses of the other layers

The RL factor is determined from Equation 13

The SN_o used in the calculation of the SN_{eff} considers the thickness of the existing AC layer less the depth of the CIR layer as calculated per Equation 21:

$$SN_o = a_1 \times (D_1 - D_{CIR}) + a_2 m_2 D_2 + a_3 m_3 D_3 \qquad [21]$$

Where; D_{CIR} is the depth of the CIR layer, inch

The SN_{eff} is calculated from Equation 22:

$$SN_{eff} = CF \times SN_o + a_{CIR}\, D_{CIR} \qquad [22]$$

Where; a_{CIR}: 0.25 and CF: Correction factor obtained from figure 28.

*c. **Based on NDT:***

The SN_{eff} is calculated from Equation 23:

$$SN_{eff} = 0.0045 \times (D) \times (E_p)^{1/3} - a_1^* \times D_{CIR} \qquad [23]$$

3. Final design:

Based on Remaining Life

$$SN_{OL} = SN_f - SN_{eff} \qquad [24]$$

Based on NDT

$$SN_{OL} = SN_f - (SN_{eff} + a_{CIR} D_{CIR}) \qquad [25]$$

Then AC overlay is calculated from Equation 26

$$D_{OL} = SN_{OL}/a_{OL} \qquad [26]$$

Design Example: the following calculations present a design of AC overlay over flexible pavement with a CIR layer.

All the information from the design example of AC overlay over AC pavement remains the same with the exception that the top 3 inches of the existing AC layer will be subjected due to the CIR process.

1. The depth of the CIR layer, D_{CIR} = 3.0 inch
2. The coefficient for the CIR layer, a_{CIR} = 0.25
3. The coefficient for the existing AC layer, a_1^* = 0.30 (from table 23)
4. Establishing the SN_{eff} of the existing pavement based on Remaining Life:

The RL factor is determined from Equation 13: 32%

The CF is obtained from figure 28: 0.81

The SN_o is calculated per Equation 21:

$$SN_o = a_1 \times (D_1 - D_{CIR}) + a_2 m_2 D_2 + a_3 m_3 D_3$$

$$SN_o = 0.35\times(6.0 - 3.0) + 0.1\times1.0\times8.0 + 0.07\times1.0\times10.0 = 2.55$$

The SN_{eff} is calculated from Equation 22:

$$SN_{eff} = CF \text{ x } SN_o + a_{CIR} D_{CIR}$$

$$SN_{eff} = 0.81\times2.55 + 0.25\times3.0 = 2.82$$

5. Establishing the SN_{eff} of the existing pavement based on NDT:

The SN_{eff} is calculated from Equation 23:

$$SN_{eff} = 0.0045 \times (D) \times (E_p)^{1/3} - a_1{}^* D_{CIR}$$

$$SN_{eff} = 0.0045 \times (24.0) \times (17{,}963)^{1/3} - 0.3\times3.0 = 1.93$$

6. Determining the required thickness of the AC overly:

   i. Based on Remaining Life:

$$SN_{OL} = SN_f - SN_{eff}$$

$$SN_{OL} = 3.50 - 2.82 = 0.68$$

$$D_{OL} = SN_{OL}/a_{OL} = 0.68/0.35 = 1.94 \text{ inch}$$

After rounding-up to the nearest 0.5 inch, D_{OL} = 2.0 inch

   ii. Based on NDT:

$$SN_{OL} = SN_f - (SN_{eff} + a_{CIR} D_{CIR})$$

$$SN_{OL} = 3.5 - (1.93 + 0.25\times3.0) = 0.82$$

$$D_{OL} = SN_{OL}/a_{OL} = 0.82/0.35 = 2.34 \text{ inch}$$

After rounding-up to the nearest 0.5 inch, D_{OL} = 2.5 inch

Comparison of Rehabilitation Options

The previous sections presented the three options for rehabilitating an existing flexible pavement; AC overlay over existing AC layer (with and without coldmilling) and AC overlay over CIR layer. The results are summarized in Table 24. An economic analysis should be conducted in order to select the optimum rehabilitation approach while taking into consideration the following factors:

- The CIR option should only be evaluated in the cases where the existing pavement exhibits surface cracking distresses.
- The cost of the CIR process.
- The ability of the CIR layer to reduce the occurrence of reflective cracking based on previous research that evaluated CIR projects throughout Nevada.
- The additional cost associated with installation of a reflective cracking mitigation technique for the option of AC overlay over existing AC layer.

Table 24. Comparison of Rehabilitation Options for Design Examples.

Rehabilitation Options	AC Overlay Thickness (inch)		
	Based on Visual Survey	Based on Remaining Life	Based on NDT
AC Overlay	3.0	2.0	2.0
Coldmilling (3 inch) and AC Overlay	5.5	4.5	4.5
CIR (3 inch) and AC Overlay	Not Applicable	2.0	2.5

5.3 Minimum AC and Base Layer Thicknesses

An acceptable performance of a flexible pavement structure requires the implementation of minimum design requirements. Figure 32 depicts a summary of the analysis process for recommended minimum pavement design layer thicknesses. Minimum SN values of 3.30, 2.55, and 2.00 are recommended for arterial/industrial, collector, and local/residential functional classification, respectively (table 25). Note that minimum pavement design requirements may vary among agencies. The analysis process shown in Figure 32 leads to one of two possible options based on SN or thickness. The path to each outcome is as follows:

- **Calculated design SN value is less than the recommended minimum SN value for the associated functional classification.** The minimum pavement structural thickness design in table 25 is used:
  - Local/residential streets: 4-inch AC surface on 6-inch base layer.
  - Collector streets: 5-inch AC surface on 8-inch base layer.
  - Arterial streets: 6-inch AC surface on 12-inch base layer.
- **Calculated design SN value is greater than or equal to the minimum SN value, but the design AC layer thickness is less than the recommended minimum AC thickness value for the associated functional classification.** The minimum AC layer thickness design in table 25 is used; and the base layer thickness is re-calculated using the minimum AC layer thickness and the design SN value:
  - Local/residential streets: 4-inch AC surface on re-calculated base layer thickness.
  - Collector streets: 5-inch AC surface on re-calculated base layer thickness.
  - Arterial streets: 6-inch AC surface on re-calculated base layer thickness.
- **Calculated design SN value is greater than or equal to the minimum SN value with the design AC layer thickness being greater than or equal to the recommended minimum AC thickness value for the associated functional classification.** The calculated pavement structural design layer thicknesses are used.

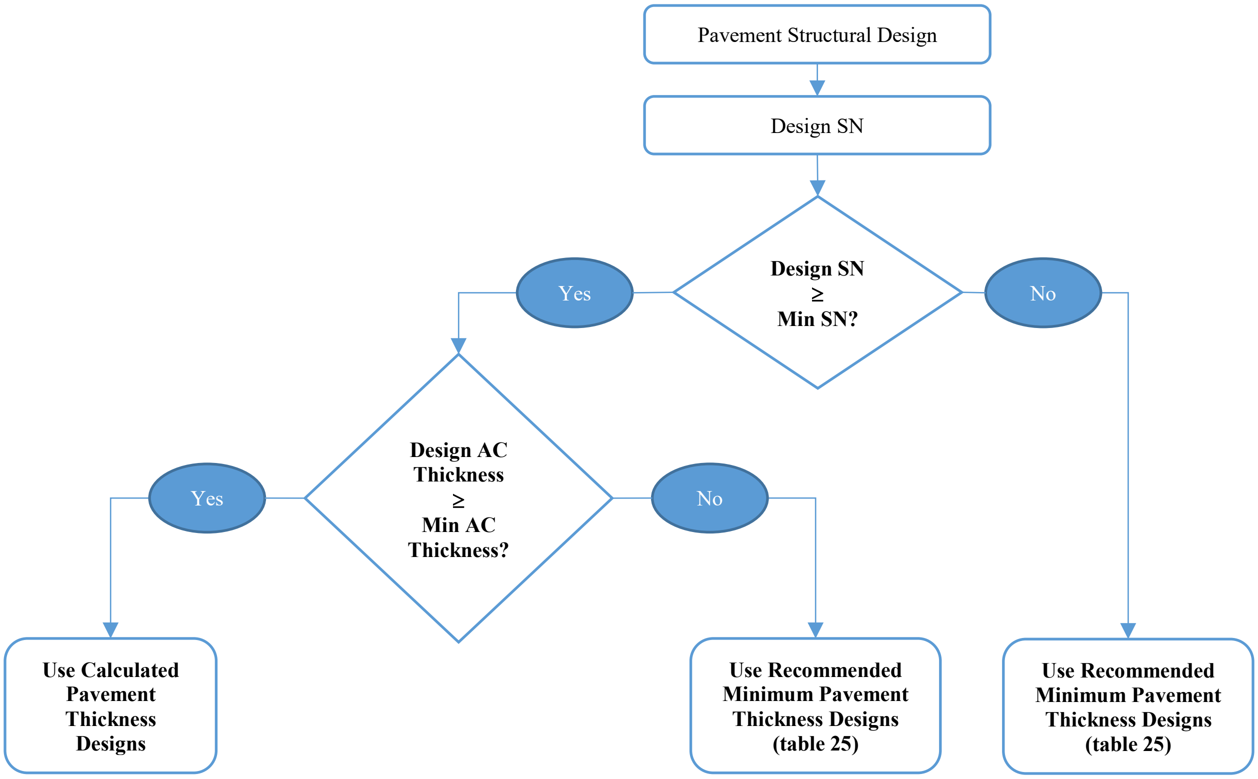

Figure 32. Flowchart for Recommended Minimum Pavement Thickness Designs.

Table 25. Recommended Minimum Pavement Designs Based on Roadway Functional Classification.

Functional Classification	Minimum SN	Minimum Thickness (inch)	
		AC Surface Layer	Base Layer
Arterial/Industrial[1]	3.30	6	12
Collector	2.55	5	8
Local/Residential	2.00	4	6

[1]Industrial carries heavier loads and at least 6 percent trucks.

5.4 AASHTO Mechanistic-Empirical Design Guide

The overall objective of the Mechanistic-Empirical Pavement Design Guide (MEPDG) is to provide highway agencies with a state-of-the-practice tool for the design of new and rehabilitated pavement structures, based on mechanistic-empirical (M-E) principles. Accordingly, pavement responses (stresses, strains, and deflections) are calculated and used to compute the incremental damage over time. The cumulative damage is then empirically related to observed pavement distresses (e.g., rutting, fatigue).

Consequently, pavement distress prediction models, and/or transfer functions, are the key components of the MEPDG procedure. The accuracy of the performance prediction models highly depends on an effective process of calibration and subsequent validation with independent data sets. It is essential that distress prediction models be properly calibrated and validated prior to adopting and using them for design purposes. By definition, the term calibration refers to the mathematical process through which the difference between observed and predicted values of distress is minimized. On the other hand, the term validation refers to the process of confirming that the calibrated model can produce robust and accurate predictions for cases other than those used for model calibration.

All current performance models in the MEPDG and associated AASHTOWare® Pavement ME software were calibrated on a global level to observed field performance over a representative sample of pavement test sites throughout North America. However, policies on pavement preservation and maintenance, construction and material specifications, construction practices, and material types vary among highway agencies and are not considered directly in the MEPDG. These factors can and should be considered indirectly through the local calibration parameters included in the MEPDG. The calibration process requires the use of data from the pavement management system (PMS) after converting the measured PMS distresses to a measurement consistent with the Pavement ME Design definitions.

Hence, this calibration-validation process is critical and necessary before a full implementation of the MEPDG by an agency. It should be noted that the first step toward a locally calibrated MEPDG is the development of the laboratory performance models for the locally used materials.

The following is a brief description of the MEPDG models used to predict performance of new and rehabilitated pavements and are considered of interest to North-Western Nevada.

Alligator Cracking in Asphalt Concrete Layer

Alligator cracking initiates at the bottom of the AC layers and propagates to the surface with repeated application of heavy truck axles. Alligator cracking prediction in the MEPDG begins with the computation incrementally of asphalt bottom up fatigue damage. This is done using a grid pattern throughout the AC layers at critical depths to determine the location within the AC layer subjected to the highest amount of horizontal tensile strain—the mechanistic parameters used to relate applied loading to fatigue damage. An incremental damage index, ΔDI, is calculated by dividing the actual number of axle loads by the allowable number of axle loads (note that computation of damage is based on Miner's hypothesis) within a specific time increment and axle load interval for each axle type. The cumulative damage index for each critical location is determined by summing the incremental damage over time and traffic using Equation 27.

$$DI = \sum(\Delta DI)_{j,m,l,p,T} = \sum\left(\frac{n}{N_{f-AC}}\right)_{j,m,l,p,T} \qquad [27]$$

Where,
n = Actual number of axle load applications within a specific time period
j = Axle load interval
m = Axle load type (single, tandem, tridem, quad, or special axle configuration)
l = Truck type using the truck classification groups included in the MEPDG
p = Month
T = Median temperature for the five temperature intervals or quintiles used to subdivide each month, °F
$N_{f\text{-}AC}$ = Allowable number of axle load applications to fatigue cracking in AC layer

The allowable number of axle load applications needed for the incremental damage index computation is shown in Equation 28.

$$N_{f-AC} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{AC})^{k_{f3}\beta_{f3}} \quad [28a]$$

$$C = 10^M \quad [28b]$$

$$M = 4.84\left(\frac{V_{be}}{V_a+V_{be}} - 0.69\right) \quad [28c]$$

$$C_H = \frac{1}{0.000398+\frac{0.003602}{1+e^{(11.02-3.49H_{AC})}}} \quad [28d]$$

Where,
ε_t = Tensile strain at critical locations and calculated by the structural response model, inch/inch
E_{AC} = Dynamic modulus of AC mix, psi
k_{f1}, k_{f2}, k_{f3} = Global field calibration parameters
β_{f1}, β_{f2}, β_{f3} = Local or mixture specific field calibration constants
V_{be} = Effective asphalt content by volume, percent
V_a = Percent air voids in the asphalt mixture
C_H = Thickness correction term
H_{AC} = Total AC layer thickness, inch

Alligator cracking is then calculated from the cumulative damage over time (Equation 27) using the relationship presented as Equation 29.

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1+e^{\left(C_1C_1^*+C_2C_2^*Log(DI_{Bottom})\right)}}\right) \quad [29a]$$

$$C_1^* = -2C_2^* \quad [29b]$$

$$C_2^* = -2.40874 - 39.748(1+H_{AC})^{-2.856} \quad [29c]$$

Where,
FC_{Bottom} = Area of alligator cracking that initiates at the bottom of AC layer, percent of total lane area
DI_{Bottom} = Cumulative damage index at the bottom of AC layer
$C_{1,2,4}$ = Transfer function regression constants

<u>Rutting in Various Pavement Layers</u>

Rutting is caused by the plastic or permanent vertical deformation in the AC layer, unbound base/sub-base layers, and subgrade/foundation soil. For the MEPDG, rutting is predicted by calculating incrementally the plastic vertical strain accumulated in each pavement layer due to applied axle loading. In other words, rutting is the sum of all plastic vertical strain at the mid-depth of each pavement layer within the pavement structure, accumulated over a given analysis period. The rate of pavement layer plastic deformation could vary significantly over a given time increment since: (1) the pavement layer properties (asphalt mixture and unbound aggregate material and subgrade) do change with temperature (summer versus winter months) and moisture (wet versus dry); and (2) applied traffic could also be very different.

The MEPDG model for calculating total rutting is based on the universal “strain hardening” relationship developed from data obtained from repeated load permanent deformation triaxial tests of both asphalt mixtures and unbound aggregate materials and subgrade soils in the laboratory. The laboratory-derived relationship was then calibrated to match field measured rut depth. It should be noted that no rutting is assumed to occur in the cement bound material (CBM) in the MEPDG.

For the asphalt mixtures, the MEPDG field calibrated form of the laboratory-derived relationship from repeated load permanent deformation tests is shown in Equation 30.

$$\Delta_{p(AC)} = \varepsilon_{p(AC)} h_{AC} = \beta_{r1} k_z \varepsilon_{r(AC)} 10^{k_{r1}} n^{k_{re}\beta_{r2}} T^{k_{r3}\beta_{r3}} \quad [30a]$$

$$k_z = (C_1 + C_2 D) \times 0.328196^D \quad [30b]$$

$$C_1 = -0.1039(H_{AC})^2 + 2.4868 H_{AC} - 17.342 \quad [30c]$$

$$C_2 = +0.0172(H_{AC})^2 - 1.7331 H_{AC} + 27.428 \quad [30d]$$

Where,

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

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

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

$h_{(AC)}$ = Thickness of the AC layer/sublayer, inch

n = Number of axle load repetitions

T = Mix or pavement temperature, °F

k_z = Depth confinement factor

k_{r1}, k_{r2} , k_{r3} = Global field calibration parameters

β_{r1}, β_{r2}, β_{r3}, = Local or mixture field calibration constants

D = Depth below the surface, inch

H_{AC} = Total AC layer thickness, inch

Equation 31 shows the field-calibrated mathematical equation used to calculate plastic vertical deformation within all unbound pavement sublayers and the foundation soil.

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \quad [31a]$$

$$Log\beta = -0.61119 - 0.017638(W_c) \quad [31b]$$

$$\rho = 10^9 \left(\frac{C_0}{(1-(10^9)^{\beta})}\right)^{\frac{1}{\beta}} \quad [31c]$$

$$C_0 = Ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}}\right) = 0.0075 \quad [31d]$$

Where,

$\Delta_{p(soil)}$ = Permanent or plastic deformation for the unbound layer/sublayer, inch

n = Number of axle load applications

ε_o = Intercept determined from laboratory repeated load permanent deformation tests, inch/inch

ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_o, β, and ρ, inch/inch

ε_v = Average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, inch/inch

h_{soil} = Thickness of the unbound layer/sublayer, inch

k_{s1} = Global calibration coefficients

β_{s1} = Local calibration constant for the rutting in the unbound layers (base, sub-base, or subgrade)

W_c = Water content, percent
M_r = Resilient modulus of the unbound layer or sublayer, psi
$a_{1,9}$ = Regression constants; a_1=0.15 and a_9=20.0
$b_{1,9}$ = Regression constants; b_1=0.0 and b_9=0.0.

Reflective Cracking in Asphalt Concrete Layer

In August of 2015, the NCHRP 1-41 reflection cracking model was integrated in the AASHTOWare® Pavement ME software which included enhanced capabilities for the AC Overlay of Existing AC Pavements design. The enhancement resulted in a large impact on the total predicted cracking for AC overlays and existing AC pavements.

The prediction of reflection cracks in the previous version of the software was based on an empirical regression equation and only applicable to load-related cracks. On the other hand, the NCHRP 1-41 reflection cracking model is a M-E based fracture mechanics model for predicting reflection cracks. The model is applicable to load and non-load related cracks of flexible, semi-rigid, intact PCC, and fractured PCC pavements. A calibration of the new reflection cracking model for the North-Western Nevada is needed, especially that Nevada DOT mandates the use of polymer-modified asphalt binder.

CHAPTER 6. PAVEMENT PRESERVATION

6.1 Introduction

FHWA defines pavement preservation as: "A program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety and meet motorist expectations."

An effective pavement preservation program will address pavements while they are still in good condition and before the onset of irreversible damage. By applying a cost-effective treatment at the right time, the pavement can be restored almost to its original condition. The cumulative effect of systematic, successive preservation treatments is to postpone costly rehabilitation and reconstruction. During the life span of a pavement, the cumulative discount value of the series of pavement preservation treatments is substantially less than the discount value of the more extensive, higher cost of rehabilitation/reconstruction and, therefore, is generally a more effective approach to achieve good long-term performance.

6.2 Preventive Treatments

AASHTO defines preventive treatments as: “A planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without significantly increasing the structural capacity.”

Preventive treatments are typically applied to pavements in good condition having significant remaining service life. As a major component of pavement preservation, preventive treatments represent a strategy of extending the service life by applying cost-effective treatments to the surface or near-surface of structurally sound pavements. Examples of preventive treatments for flexible pavements include the following:

- Chip seal
- Slurry seal
- Micro-surfacing
- Cape seal
- Thin AC overlay ≤ 1 inch
- Crack seal (not true preventive)

As long as pavements are subjected to traffic loads and environmental actions, they will experience deterioration and eventual failure. A preventive treatment program puts pavement engineers and managers in full control of the system’s long-term behavior: to prevent failures from occurring. Through preventive treatments, the agency can decide on the level of service provided by the facility and the length of time prior to a major rehabilitation activity.

Figure 33 compares the two concepts: preventive treatment and rehabilitation in terms of the PSI of a flexible pavement. Preventive treatment provides good pavement condition over an extended time period, while rehabilitation offers excellent conditions for a short time period. Typical pavement users have short memory span: no one will remember the excellent conditions that prevailed during the first few years while they are stuck using a poor (rough) facility in years 7-10. On the other hand, the boundary between excellent and good conditions (year 10) is very narrow and most users would not be able to differentiate between the two levels. The good level of service coupled with the significant savings offered by a preventive program makes it a wise choice for most agencies.

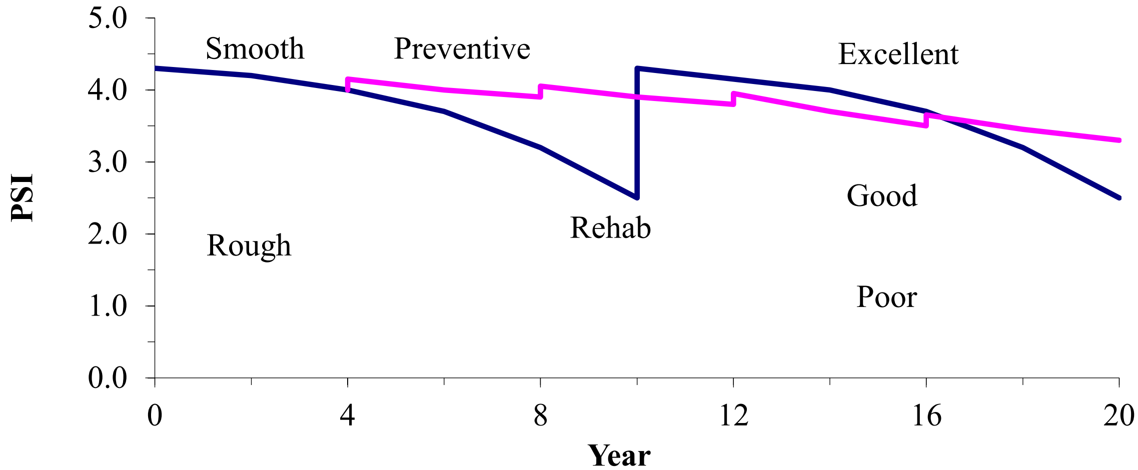

Figure 33. Impact of preventive treatment and rehabilitation on pavement performance.

6.2.a Optimum Application Time

Previous studies conducted by the PES Program at the University of Nevada, for the Washoe RTC indicated that the optimum time to apply the first slurry seal is when the pavement is 3–5 years old followed by the second slurry seal when the pavement is 7–9 years old.[13,14] Accordingly, for a newly constructed pavement, it is recommended that a slurry seal be first applied when the PCI reaches 90 followed by a second slurry seal when PCI reaches 86. For overlaid pavement, it is recommended that the agency apply the first slurry seal when PCI reaches 87 and the second slurry seal when PCI reaches 77. In the case of micro-surfacing and Cape seal, it was determined that the optimum application period is a function of the PCI.[15] It was concluded that micro-surfacing and Cape seal are highly effective treatments for flexible pavement at any age, as long as its PCI value is equal or greater than 70. Accordingly, the effective performance life of micro-surfacing cape seals is 7 years in the Truckee Meadows and 5 years in Incline Village; while the effective performance life of slurry seal cape seals is 3.5 years in the Truckee Meadows and 3 years in Incline Village.

6.3 Corrective Treatments

Corrective treatments are performed in response to the development of a deficiency or deficiencies that negatively impact the safe/efficient operations of the facility and future integrity of the pavement section. Corrective activities are generally reactive, not proactive, and performed to restore a pavement to an acceptable level of service due to unforeseen conditions. Activities such

[13]Hajj, E.Y., Loría, L., Sebaaly, P.E., Borroel, C.M., and Leiva, P. (2011). "Optimum Time for Application of Slurry Seal to Asphalt Concrete Pavements," *Transportation Research Record*, 2235(1):66-81. doi:10.3141/2235-08
[14]Hajj, E.Y., Loria L., Sebaaly, P.E., Cortez, E., and Gibson, S. (2013). "Effective Timing for Two Sequential Applications of Slurry Seal on Asphalt Pavement," *ASCE Journal of Transportation Engineering*, Vol. 139, Issue 5.
[15]Sebaaly, P.E., Hajj, E.Y., Weitzel, D., and Belancio, G. (2016). "Effectiveness of Cape Seal Pavement Preservation Technique in Northern Nevada," Final Report, WRSC-201602-03, UNR.

as pothole repair, patching of localized pavement deterioration, e.g. edge failures and/or grade separations along the shoulders, are considered examples of corrective treatments of flexible pavements.

6.4 Routine Maintenance

AASHTO defines routine maintenance as: "Work that is planned and performed on a routine basis to maintain and preserve the condition of the highway system or to respond to specific conditions and events that restore the highway system to an adequate level of service."

Routine maintenance consists of day-to-day activities that are scheduled by maintenance personnel to maintain and preserve the condition of the highway system at a satisfactory level of serviceability. Examples of pavement-related routine maintenance activities include cleaning of roadside ditches and structures, maintenance of pavement markings and crack filling, pothole patching and isolated overlays. Crack sealing is another routine maintenance activity, which consists of placing asphalt-based bituminous materials into "non-working" cracks to substantially reduce water infiltration; this activity may be considered a preventive action.