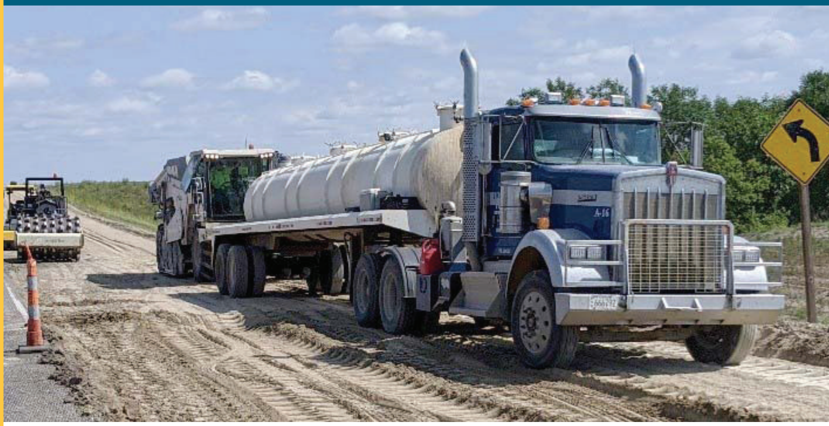
Nebraska Department of Transportation

Pavement Design Guide



NEBRASKA

Good Life. Great Journey.

DEPARTMENT OF TRANSPORTATION

Materials & Research - Pavement Design Section

2026

This document was developed as part of the Nebraska Department of Transportation's ongoing effort to provide guidance in fulfilling its mission to deliver the best possible statewide transportation system for the movement of people and goods. This mission is supported through dedicated teamwork and responsible leadership that emphasizes safety, economic development, and environmental stewardship.

This document is intended as a guidance resource and does not establish official Department policy. Instead, it is designed to assist in complying with existing NDOT policies.

Comments, suggestions, and ideas for improving this guide are welcome.

Please send comments to:

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Acknowledgments and Contributions

This Pavement Design Guide reflects the collective effort of many individuals who generously shared their time, knowledge, and expertise throughout its development. We sincerely thank everyone who participated in the preparation, review, and comment process. Their valuable feedback and continued support greatly contributed to the quality, clarity, and success of this document.

Special acknowledgment is extended to the Nebraska Department of Transportation – Materials and Research Division staff for their technical expertise, guidance, and continued support throughout the development of this guide. Their contributions and commitment to improving pavement design practices were instrumental to the completion of this document.

Chapter 1: Introduction

1.1 Purpose

The purpose of this guide is to provide the Nebraska Department of Transportation (NDOT), consultant firms, and contractors with a detailed pavement overview covering materials, design, construction, and maintenance considerations for traditionally let NDOT projects. This guide:

- Serves as an extensive pavement reference that users can browse to look up typical design values, methods, practices, and resources.
- Provides approved policies and procedures for pavement design for use on NDOT projects.
- Provides pavement designers with a uniform, streamlined process for designing pavements.
- Serve as a guide to pavement designers for selecting pavement rehabilitation strategies.
- Addresses related topics that a pavement designer needs such as procedures for load analysis and pavement forensics.

1.2 Organization

This Pavement Design Guide is organized into nine chapters:

- **Chapter 1 – Introduction**
  This chapter provides an overview of the Pavement Design Guide, policy resources, and a review of topics related to pavements and highway materials.

- **Chapter 2 – Pavement Design Process**
  This chapter provides definitions of pavement types, pavement type selection, the approved methods for pavement design, pavement design categories, pavement design process, information needed for pavement design, and pavement design reports.

- **Chapter 3 – Pavement Evaluation**
  This chapter provides definitions of pavement distresses, typical causes of pavement distresses, and guidance for selecting treatments for pavement rehabilitation.

- **Chapter 4 – Subgrade Design**
  This chapter provides an overview of Nebraska's soil characteristics, design parameters for subgrade stabilizations, and drainage layer information.

- **Chapter 5 – Gravel Road Design**
  This chapter provides an overview of Nebraska's gravel roads design and focuses on optimizing material selection, gradation, and stabilization to ensure long-term performance under varying traffic loads and environmental conditions.

- **Chapter 6 – Flexible Pavement Design**
  This chapter provides an overview of the types of flexible pavements NDOT uses, design parameters and typical ranges, backcalculation methodology, pavement detours and widening, and perpetual flexible pavement design.

- **Chapter 7 – Rigid Pavement Design**
  This chapter covers the approved design method for rigid pavements, rigid pavement design process, recommended typical values, thickness determination, concrete paving standards, bonded and unbonded concrete overlays.

- **Chapter 8 – Hot-Pour Sealants**
  This chapter provides an overview of the Hot-Pour Sealants that the NDOT utilizes and general guidance for applying Hot-Pour Sealants to bituminous and concrete pavements.

- **Chapter 9 – Material Cost and Quantity Estimates**
  This chapter provides information on the Departments project pay items and the associated estimated cost for different construction practices.

1.3 Pavement Design Reference Materials

Specifications and Design Manuals

- Standard Specifications for Highway Construction (2017)
  https://dot.nebraska.gov/media/g4qp4y0d/2017-specbook.pdf

- Nebraska Department of Roadway Design Manual
  https://dot.nebraska.gov/business-center/design-consultant/rd-manuals/

- Nebraska Department of Transportation – Transportation Asset Management Plan (2022)
  https://dot.nebraska.gov/media/iqeffkxw/2022-tamp-report.pdf

Pavement ME Design Resources

- AASHTO Pavement ME Design Manual

  - Nebraska DOT User Guide for AASHTOWare Pavement ME Design
    - Created by ARA (2021)

  - Nebraska DOT User Guide for AASHTOWare Pavement ME Design Version 3.0
    - Condensed Version (Updated 2025)

  Note: AASHTOWare Pavement ME Design software along with local calibrations are used extensively. These manuals are a compilation of NDOT design practices, procedures, materials data, and related guidance used daily in conjunction with the AASHTO manual.

- Backcalculation Tool (BcT)

  - User Manual for Pavement M-E Deflection Data Analysis and Backcalculation Tool
    - AASHTOWare Backcalculation Tool, BcT – User Manual Version 1.1.5

  - AASHTOWare Backcalculation Guide
    - Desktop BcT Manual (NDOT)

Maps and Reference Information

- State and National Classification Maps by County
  https://dot.nebraska.gov/travel/map-library/func-by-county/

- Map of the Districts with Contact Information
  dist-map-with-contacts.pdf (nebraska.gov)

- AASHTO Guide to Pavement Design, Construction, and Management (Second Edition, 2026)

1.4 Policy Overview

Concrete Pavement Policies

- Installation of Dowels in Outside Surfaced Shoulders for I-80, Lincoln to Grand Island
  https://dot.nebraska.gov/media/4uzby5hw/i80-doweled-conc-shoulders-mr-23-01.pdf

- Longitudinal Joints – Limit Concrete Panel Width
  https://dot.nebraska.gov/media/ryvlsblb/longitudinal-joint-policy-mr-23-02.pdf

- Portland Cement Concrete Thickness
  https://dot.nebraska.gov/media/viub4qjs/concrete-thickness-policy-mr-23-03.pdf

2.1 Introduction

Pavement Design is the process of developing the most economical combination of pavement layers, with respect to thickness, type of materials, and subgrade treatment over the design life of cumulative traffic loading. Pavement materials comprise a large portion of the cost of a project. Sound engineering judgment and careful consideration should be combined to produce a cost-effective and structurally sound pavement design.

2.2 Overview

The objectives of the pavement design process are to aid the Pavement Design Engineer in making decisions that will improve roadway condition or provide guidance for new pavement structures. The information collected from the NDOT Pavement Optimization Program (POP), As-Built Plans, and Mechanistic Empirical (ME) Pavement Design program outputs are used to provide a structure that is capable of carrying traffic loads while addressing current pavement distresses with minimum physical deterioration.

During the design life of a project, there are four Clarity Task activities completed by Pavement Design. Clarity is a project management platform used to facilitate project delivery.

Clarity Task 5258 (Planning Pavement Determination):

The Planning Pavement Determination task is created at the initial scoping of a project. The intention is to evaluate the existing pavement conditions and determine an appropriate pavement strategy based on the Proposed Design Standard selected in the Construction Project Initiation Request Form (DR73). The Pavement Determination is posted to OnBase. Task items include:

- Review the Project Initiation Request (DR73).
- Review POP and As-Built Plans. Develop a project Histogram.
  - An example of a Histogram can be found in **Chapter 3.4 – NDOT Evaluation Tools**.
- Create a DRAFT Planning Pavement Determination.
- Create a Pavement ME Design run if the project is scoped as a New & Reconstruction project.
- Review the Planning Pavement Determination during the Internal Review Meeting with the Pavement Design Engineer.
- Complete a Core Request Form.
- Through correspondence, send the Planning Pavement Determination to the District Engineer for approval.
- Attach the approved Planning Pavement Determination into the DR73 Workflow and upload the document into OnBase.
- Send notifications. (See Email Distribution List below.)
- Mark the task complete in Clarity.
- Make revisions to the Planning Pavement Determination as the DR73 moves through the scoping process.

Clarity Task 5303 (Pavement Coring & FWD):

The Pavement Coring & FWD task is intended to evaluate the condition of the road by collecting samples and data and evaluating the pavement structure and subgrade. Shortly after the Scoping meeting for a project, a request must be created to inform the coring crew of locations where to perform Coring/FWD/GPR work. Results from the Coring/FWD/GPR will be processed and placed in OnBase for review.

- Review the Core Request Form.
- Perform Coring/FWD/GPR in locations identified from Core Request Form.
- Process Coring/FWD/GPR data.
- Upload the Coring/FWD/GPR data into OnBase.
- Send a notification to the Pavement Designer that the data is ready for review.
- Mark the task complete in Clarity.

Clarity Task 5364 (Pavement Determination Review):

The Pavement Determination Review is intended to incorporate additions and revisions that result from Coring/FWD/GPR review and Roadway Design project development. The Pavement Determination Review is posted to OnBase and has the same notification distribution.

- Created approximately 1 year after the Planning Pavement Determination.
- Create a DRAFT Pavement Determination Review.
- Incorporate any changes due to decisions made during the PCM 30/Construction Meeting.
- Include any project changes as needed from the District and Roadway Design.
- Review the Pavement Determination Review during the Internal Review Meeting with the Pavement Design Engineer.
- Review the Core Request Form for any changes.
- Send the Pavement Determination Review to the District Engineer for approval.
- Upload the approved Pavement Determination Review into OnBase.
- Send notifications. (See Email Distribution List below.)
- Mark the task complete in Clarity.
- Make revisions to the Pavement Determination Review as needed as the project progresses.

Clarity Task 5406 (Final Pavement Determination):

The Final Pavement Determination is completed after reviewing Cores/FWD/GPR results. The Final Pavement Determination is routed to the Pavement Design Engineer, the M&R Engineer, and District Engineer through Pavement Design Workflow in OnBase.

- Created approximately 1 year after the Pavement Determination Review.
- Create a DRAFT Final Pavement Determination.
- Incorporate any changes due to decisions made during the PCM 35.
- Include any project changes as needed from the District and Roadway Design.
- Create a Pavement ME Design run using backcalculation modulus for Rehabilitation projects.
- Review the Final Pavement Determination during the Internal Review Meeting with the Pavement Design Engineer.
- Review the Coring/FWD/GPR data with the Pavement Design Engineer.
- Upload the approved Final Pavement Determination into Workflow for multistep approval.
- Send notifications. (See Email Distribution List below.)
- Mark the task complete in Clarity.
- Make revisions to the Final Pavement Determination when project updates occur.

Clarity Task 5655 (Pavement Determination Verification):

The Pavement Determination Verification is a confirmation that the Final Pavement Determination is current. This verification step takes place just prior to PS&E turn-in. There is no distribution or document posted in OnBase.

- Perform approximately 6 months before PS&E Turn-in.
- Review the Final/Pre-PS&E Plans.
- Compare the Final Pavement Determination to the Final/Pre-PS&E Plans.
- Make edits to the Final Pavement Determination as needed.
- Send corrections to the Roadway Designer for making edits to the Final/Pre-PS&E Plans.
- Mark the task complete in Clarity.

Pavement Design Distribution List for Clarity Tasks:

- Project Scheduling/Program Manager,
- Assigned Roadway Designer,
- Assigned Delivery Engineer,
- Roadway Design Manager,
- Scoping Engineer,
- Roadway Design Section Head,
- and Pavement Design Staff.

2.3 Pavement Design Standards

There are three pavement design standards:

- New and Reconstruction
- Resurfacing, Restoration, and Rehabilitation (3R)
- Preventive Maintenance

It is important that the Pavement Designer conducts an early investigation to confirm which standard applies to the project. A project's proposed design standard is selected during the project's initial scoping by the District and can be found in the Project Initiation Request Form (NDOT73) in OnBase.

New and Reconstruction Standard

New and Reconstruction is a combination of base and pavement placed on a prepared subgrade to support traffic for flexible and rigid pavements. New and Reconstruction projects have an expected service life exceeding 20 years and generally consist of:

- Construction of a new road.
- Relocating an existing route on new alignment.
- Removal of the pavement structure and construction of a new base or the modification of the existing base, which will be designed to reconstruction standards.
- Modification of the base is defined as improving or strengthening the existing base through chemical (Fly Ash, Lime, etc.) or mechanical (geofabric, geogrid, etc.) means and will require designing to reconstruction standards.
- Building a new bridge or reconstructing an existing bridge.
- Adding through lanes to the existing alignment.

New and Reconstructed projects should be considered when:

- The crash history indicates the need for improvements that can significantly reduce the crash rate.
- Meeting 3R standards will require that significant existing geometric deficiencies be corrected.
- Significant grading is to be done which requires major right-of-way to be acquired and/or major utility relocations.

The minimum design standards for New and Reconstructed projects on the Nebraska Highway System may be found in the Green Book (Ref. 1.1), the Interstate Green Book (Ref. 1.2), and in Appendix H, "AASHTO Minimum Design Guidance" of the Roadway Design Manual.

Practical Design considerations may allow the application of 3R standards to a segment (or segments) of the current New and Reconstructed project (e.g., reconstructing the pavement structure at the existing width without modification of the existing base). Please refer to Chapter 1: Roadway Design Standards of the Roadway Design Manual for more information.

https://dot.nebraska.gov/media/1vshig3g/d-chap-1-design-standards.pdf

Resurfacing, Restoration, and Rehabilitation (3R) Standard

3R projects are generally undertaken to preserve the highway assets, improve the reliability of the transportation system, maintain the mobility of the highway user, mitigate highway safety issues identified through crash history and operational issues identified through analysis. Generally, it is not the purpose of 3R projects to increase highway capacity. A 3R resurfacing strategy typically has an expected service life of up to 20 years.

Application of 3R design standards to a pavement resurfacing project is, for the most part, determined by the pavement recommendation.

- Pavement recommendations that address deficiencies in the pavement structure, increase the structural capacity and extend the life of the facility by up to 20 years will usually be designed to 3R standards.
- Pavement recommendations that require pavement replacement and restoration of the base can be designed to 3R standards.
  - Restoration of the base is defined as restoring the original condition of the base (subgrade preparation).
  - A portion of the existing base may be removed to accommodate the required pavement thickness based on the pavement recommendation.
- Pavement recommendations that require removal of the entire pavement structure and the construction of a new base or the modification of the existing subgrade will be designed to New and Reconstructed standards.
  - Modification of the base/subgrade is defined as improving the drainage layer or strengthening the existing base/subgrade through chemical (fly ash, lime, etc.) or mechanical (geofabric, geogrid, etc.) means.
  - Practical Design considerations may allow the application of 3R standards to a segment (or segments) of the current New and Reconstructed project (e.g. reconstructing the pavement structure at the existing width without modification of the existing base/subgrade).

The 3R design standard utilizes a cost/benefit paradigm, including such strategies such as:

- Practical Design,
- 2+2 Projects, and
- Super 2 Roadways.

For NDOT 3R guidance, see Chapter 17: Resurfacing, Restoration and Rehabilitation (3R) Projects of this manual and the MDS (Ref. 1.3).

https://dot.nebraska.gov/media/f5ffhjyt/t-chapter-17-3r.pdf

Preventive Maintenance Standard

Preventive Maintenance projects are programmed for the preservation of the existing roadway surfacing back to its' original condition without significantly increasing the structural capacity. Preventive Maintenance is typically applied to pavements in good condition which have significant service life remaining. A Preventive Maintenance project has an expected service life of up to 12 years.

The Board of Public Roads Classifications and Standards has issued maintenance standards applicable for each functional classification of roadway (Chapter 2, Section 003, of the MDS, Ref. 1.3). Building curb ramps and upgrading roadway appurtenances (such as guardrail) are allowed on Preventive Maintenance projects. Please refer to Chapter 1: Roadway Design Standards for more information.

For the design process, Preventive Maintenance projects are initially separated into two categories, Roadway Design Maintenance and Materials & Research Maintenance.

- Roadway Design Maintenance
  - Asphalt Overlays
  - Hot-In-Place Recycling
- Materials & Research Maintenance
  - Microsurfacing
  - Fog Sealing
  - Concrete Pavement Repairs
  - Joint and/or Crack Sealing
  - Diamond Grinding and Texturing
  - Penetrating Concrete Sealer

During the evaluation of a project, the Pavement Design Engineer may change the pavement design standard to better fulfill the needs of the roadway. In rare cases, for a variety of factors, it may be anticipated that the typical maintenance strategy may fail before a 12-year expected service life. Factors include, but are not limited to the existing pavement conditions, overall pavement thickness, heavy truck loading and environmental conditions. The Pavement Design Engineer may evaluate the expected service life of a proposed maintenance strategy. If the Pavement Design Engineer determines the strategy will not meet its anticipated service life, the surfacing thickness may be increased to reach a service life of approximately 12 years.

Summary

In comparison to New and Reconstruction projects, 3R projects generally have shorter project delivery time, have fewer impacts on the environment, fewer and less extensive right-of-way acquisitions, and are less costly. Maintenance projects generally have even fewer impacts and lower costs.

For resurfacing projects, the appropriate minimum design standards are applied to the project segment based on the expected service life. Each design standard and the associated project's expected service life are as follows:

- **New and Reconstruction:** A pavement strategy typically involves construction or reconstruction of an entire pavement, base, and subgrade system. These have an anticipated service life exceeding 20 years.

- **Resurfacing, Restoration, and Rehabilitation (3R):** A 3R resurfacing strategy typically has an anticipated service life up to 20 years.

- **Preventive Maintenance:** A highway surface Preventive Maintenance strategy has an anticipated service life of approximately 12 years.

The Pavement Design Engineer may adjust the pavement design strategy of a project based on the current pavement conditions and the traffic needs of the roadway.

2.4 Pavement ME Design

For complete instructions for running Pavement ME Design, see NEDOT PMED User Manual.

Background

The AASHTOWare Pavement ME Design software tool (referred to as Pavement ME Design® (PMED)) was developed based on the results and findings from a series of National Cooperative Highway Research Program (NCHRP) research studies, namely, NCHRP 1-37A, 1- 40B, and 1-40D, conducted from 1998 through 2008.1 The PMED software accounts for changes in traffic, material, and environmental conditions and incorporates mechanistic-based algorithms, computations, and transfer functions to simulate the impact of these changes on the long-term performance of the pavements.

The software computes pavement responses such as stresses, strains, truck axle load deflections, and accumulated pavement damage over the pavement design/analysis period. The accumulated pavement damage is then used to determine pavement distresses that are indicative of the performance of pavements over the design or analysis period.

Such a rational engineering design approach provides a reliable and cost-effective method of diagnosing pavement problems, as well as forecasting maintenance and rehabilitation needs. AASHTO adopted this procedure in 2008 and published a Manual of Practice for its use (AASHTO, 2008). Additional Revisions of the Manual of Practice was published in 2021.

Overview of NDOT User Guide

The ME User Guide provides an overview of the Pavement ME Design software and MEPDG procedure. It was prepared for Nebraska DOT's (NDOT) with inputs specific to Nebraska's site conditions, traffic, climate, and materials used to accomplish the following:

- Provide guidance on using the software for new pavement and rehabilitation design strategies.
- Determine the inputs of the PMED software.
- Provide guidance on using the PMED software to perform pavement designs for most new and rehabilitated pavement types, specifically for Nebraska conditions (traffic and climate) and materials.
- The design strategies include:
  - New or reconstructed flexible or asphalt pavement.
  - New or reconstructed rigid pavement: Jointed Plain Concrete Pavement (JPCP).
  - Flexible pavement rehabilitation:
    - Asphalt overlay of existing flexible pavement.
    - In place recycling of asphalt pavements.
    - JPCP overlay of existing flexible pavement.
  - Rigid pavement rehabilitation:
    - Asphalt overlay of existing JPCP.
    - Fractured JPCP with the use of asphalt overlays.
    - JPCP overlay of existing JPCP.
  - JPCP and Asphalt maintenance to extend the life cycle of flexible and rigid pavements.

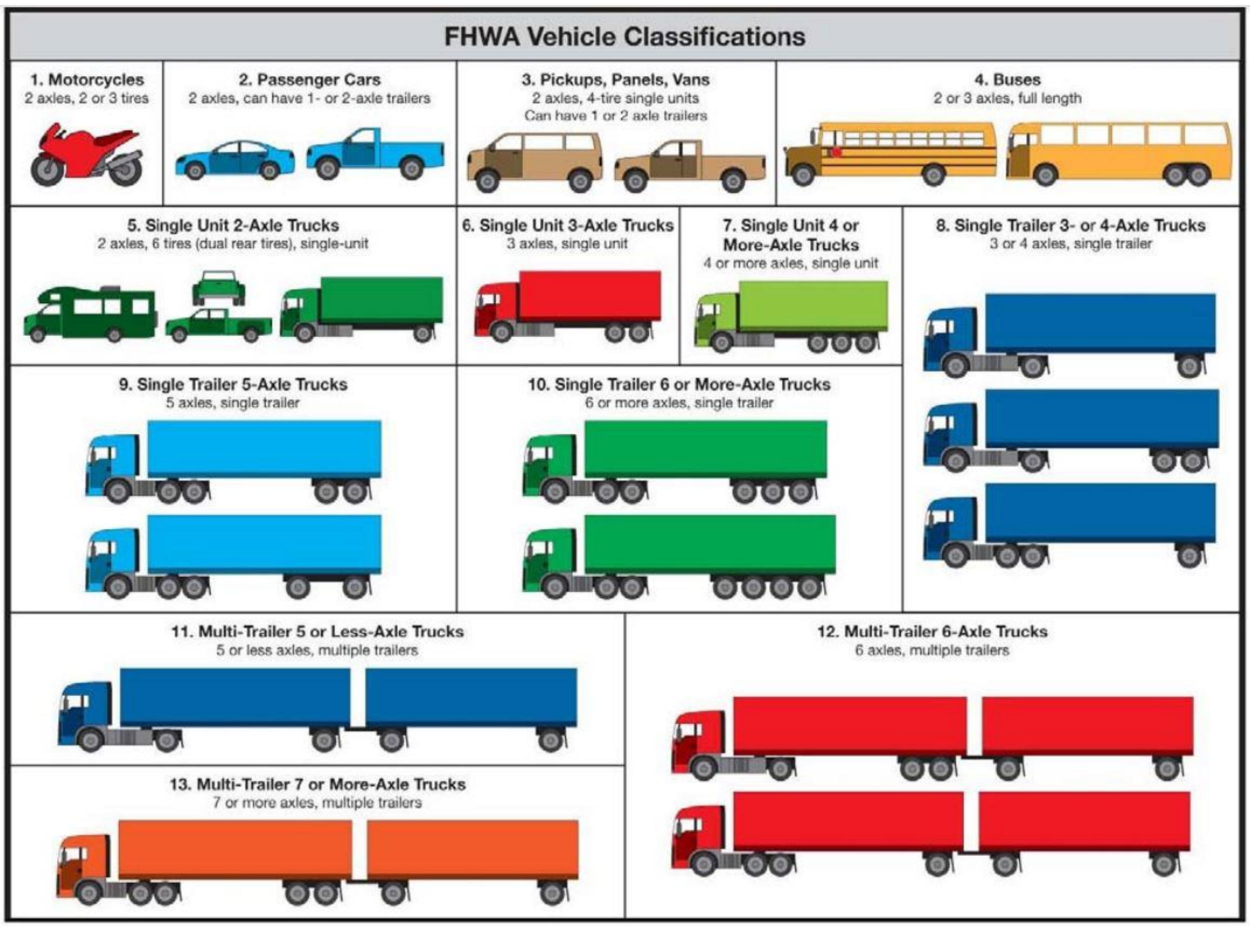
2.5 Vehicle Classifications and Pavement Loading Analysis

During the design process, the pavement designer should have an understanding of the amount of loading the pavement will be subjected to. Different vehicle classifications will develop different loading applications.

Vehicle Classifications

The Federal Highway Administration (FHWA) developed a standardized vehicle classification system in the mid-1980s. FHWA classifies vehicles in terms of their configuration rather than weight. This type of classification system is more conducive to traffic applications but can be adapted for pavement loading applications. The FHWA *Traffic Monitoring Guide* (TMG) recommends classifying vehicles into 13 different categories.

Figure 2.5-1 – FHWA Vehicle Classifications



For more information on vehicle classification rules, please refer to the FHWA's Traffic Monitoring Guide at the link below.

Traffic Monitoring Guide - Policy | Federal Highway Administration (dot.gov)

Equivalent Single Axle Load

The most common historical approach to determining the amount of damage a pavement will be subjected to is called an Equivalent Single Axle Load. An Equivalent Single Axle Load (or ESAL) is an approach that converts wheel loads of various magnitudes and repetitions of "mixed traffic" to an equivalent number or "standard" or "equivalent" loads. The normal designated ESAL in the United States is 18,000 lbs. For example, a given vehicle on a given type of pavement is 3.0 ESALs means that one pass by the vehicle has the same effect on the pavement as three passes by an 18,000 lb. single axle.

At the time of its development (early 1960s at the AASHO Road Test) it was much easier to use a single number to represent all traffic loading in the somewhat complicated empirical equations used for predicting pavement life. There are 2 standard ESAL equations (one for Flexible Pavement and one for Rigid Pavement) that are derived from the AASHO Road Test results. Both equations involve the same basic format except the exponents are slightly different.

Flexible Pavement ESAL Equation from Pavement Interactive:

Flexible Pavement ESAL Equation – Pavement Interactive

Rigid Pavement ESAL Equation from Pavement Interactive:

Rigid Pavement ESAL Equation – Pavement Interactive

Overloaded Vehicles

Overloaded, or overweight vehicles are vehicles that exceed the designed load-bearing capacity of the roadway. Pavement structures are designed to support a certain amount of weight, and when that weight limit is exceeded, the extra load inducing stress can lead to shortening the pavement life expectancy of the pavement. Consequently, the pavement structure is subjected to accelerated damage and manifests as potholes, cracking, rutting, and fatigue failures. An increase in pavement distresses leads to an increase in maintenance costs and frequency of repairs.

Damage to the pavement is not linear, but exponential. The graph below shows the damage inflicted on the pavement as the weight increases on the axles.

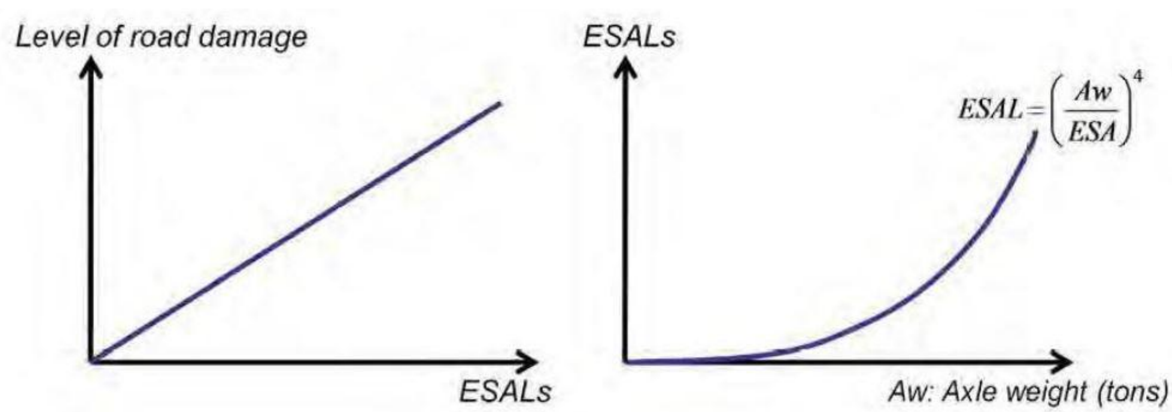


Figure 2.5-2 – Level of Road Damage due to Vehicle Loading
(Study for Harmonization of Vehicle Overloading Control in the East African Community, JICA, 2011)

For example:

One standard axle load (A_w) for a single axle is 18,000 lbs = 1.0 ESAL

$$ESAL = \left(\frac{A_W}{ESA}\right)^4 \quad = \quad ESAL = \left(\frac{18{,}000\ lbs}{18{,}000\ lbs}\right)^4 \quad = \quad ESAL = 1.0$$

If the standard axle load (A_w) is increased by 15% to 20,700 lbs, then:

$$ESAL = \left(\frac{A_W}{ESA}\right)^4 \quad = \quad ESAL = \left(\frac{20{,}700\ lbs}{18{,}000\ lbs}\right)^4 \quad = \quad ESAL = 1.75$$

If the standard axle load (A_w) is doubled to 36,000 lbs, then:

$$ESAL = \left(\frac{A_W}{ESA}\right)^4 \quad = \quad ESAL = \left(\frac{36{,}000\ lbs}{18{,}000\ lbs}\right)^4 \quad = \quad ESAL = 16.0$$

An example of an overweight vehicle calculation is shown on the next page.

FHWA Comprehensive Truck Size and Weight Study for Pavements and Truck Size Weight Regulations

https://www.fhwa.dot.gov/reports/tswstudy/tswwp3.pdf

For additional information on Equivalent Single Axle Loading, please visit Pavement Interactive at the following link:

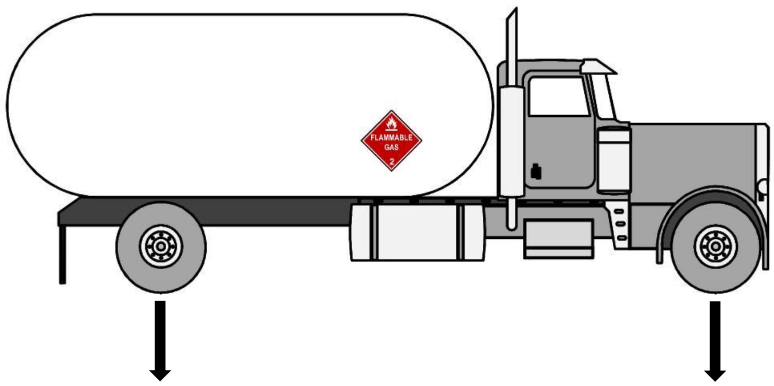
Equivalent Single Axle Load – Pavement Interactive

In conclusion, overweight vehicles pose a significant challenge to pavement structures, accelerating deterioration and reducing the overall lifespan. The excessive loads these vehicles impose can lead to common forms of pavement distress, such as rutting, cracking, and pavement fatigue, which in turn increases the maintenance costs and compromise road safety. Understanding and mitigating the impacts of overweight vehicles is crucial for preserving infrastructure integrity, ensuring long-term performance, and optimizing public investment in transportation networks.

For information on legal vehicle loads in Nebraska, please refer to the Nebraska Truck Information Guide at the Nebraska State Patrols website.

Carrier Enforcement | Nebraska State Patrol

Example: Class B Single Vehicle



20,000 lbs 20,000 lbs

Legal Load = 40,000 lbs
= (20,000 lbs front axle + 20,000 lbs rear axle)

ESAL = $(20{,}000\text{ lbs} / 18{,}000\text{ lbs})^4 + (20{,}000\text{ lbs} / 18{,}000\text{ lbs})^4$
= **3.05 ESALs per Truck**

15% over legal weight = 46,000 lbs
= (20,000 lbs front axle + 26,000 lbs rear axle)

ESAL = $(20{,}000\text{ lbs} / 18{,}000\text{ lbs})^4 + (26{,}000\text{ lbs} / 18{,}000\text{ lbs})^4$
= **5.88 ESALs per Truck**

Vehicle Damage Factor = 5.88 ESALs per Truck / 3.05 ESALs per Truck
= 1.93

A 15% load increase causes 1.93 times the damage of a legal weight Class B Single Vehicle.

Load Spectra

The most recent method for measuring vehicular traffic loading is by the load spectra method. According to Pavement Interactive, load spectra is a representation of the distribution of vehicle axle loads categorized by axle type and load range. ESALs represent all traffic as a single, equivalent axle load, while load spectra directly characterize traffic by the number of axles, their configuration, and their individual weights, without conversion to a single equivalent value. This information can be used with a series of mechanistic-empirical equations to develop a pavement structural design.

Some key advantages of the load spectra approach are:

- Load spectra is compatible with the FHWA's Traffic Monitoring Guide (TMG) and thus many agencies are already collecting the appropriate data.

- The load spectra method offers a hierarchical approach to traffic data input depending upon the users' needs and resources. There are three levels of potential input:

    - Level 1 Inputs – Use of volume/classification and axle load spectra data directly related to the project.
    - Level 2 Inputs – Use of regional axle load spectra data and project-related volume/classification data.
    - Level 3 Inputs – Use of regional or default classification and axle load spectra data.

- Load spectra already includes information on traffic distribution including directional, lane and temporal distribution (if needed) as well as traffic growth rates.

AASHTO Pavement ME Design utilizes the load spectra approach when performing pavement analysis.

To view the FHWA Traffic Monitoring Guide, please visit the following link below:

Traffic Monitoring Guide - Policy | Federal Highway Administration

3.1 Introduction

The purpose of this chapter is to help the user recognize the distresses typically found on pavements in Nebraska and understand the possible causes. Evaluating and understanding pavement conditions is vital to the success of a project. There are many tools available to a Designer to evaluate a pavement including Cores/FWD/GPR Data, Pavement Optimization Program (POP), PathWeb, IHI data, As-Built Plans, and District Maintenance feedback.

Distress is defined as a condition of pavement structure that reduces serviceability or leads to a reduction in serviceability. Serviceability is defined as the ability of a pavement to provide a safe and comfortable ride for its users.

Distresses may be treated with a range of repairs, each having a varying degree of success. Some of the treatments shown for the distresses will provide only a short-term solution, which may be all that is needed.

3.2 Flexible Pavement

Flexible pavement distresses include a wide variety of pavement surface defects that generally fall into the following categories:

- Cracking
  - Alligator
  - Reflective
  - Longitudinal
  - Random/Block
  - Transverse
  - Edge
- Raveling/Weathering
- Distortion
- Rutting
- Excess Asphalt
- Pumping

For each distress, a number of possible treatments are available. In many instances, multiple types of distress occur so the treatment selected must be appropriate for all distresses that are present.

Alligator Cracking

Alligator cracking is a series of interconnected cracks in an asphalt layer forming a pattern which resembles an alligator's hide or chicken wire. The cracks indicate fatigue failure of the asphalt layer generally caused by repeated traffic loadings. This distress allows water to penetrate the surfacing material and subgrade, which furthers the damage. Alligator cracking, also called fatigue cracking, usually begins as a single longitudinal crack in the wheel path or a transverse crack.

Possible causes include:
- Insufficient Pavement Structure
- Inadequate Base Support
- Poor Base Drainage
- Aging and Traffic Loading

Possible treatments:
- Patching
- Mill/Fill Asphalt Overlay with Patching
- Stabilized Bituminous
- Remove and Replace Asphalt and Address Subgrade
- Improve Subgrade Drainage

Alligator Cracking

Reflective Cracking

Reflective cracking is a type of pavement failure that occurs when cracks or joints in an existing, underlying pavement layer propagate into an overlay layer.

Possible causes include:
- Traffic
- Temperature Variations
- Poor Constriction Practices
- Ability of the Asphaltic Material to Absorb Stress (Flexibility)

Possible treatments:
- Crack Seal
- Chip Seal/Armor Coat
- Mill/Fill Asphalt Overlay
    - Thicker Asphalt Overlay
    - Trenching to Underlying Problem
    - Pavement Fabric
    - Pavement Fibers

Reflective Cracking

Longitudinal Cracking

Longitudinal cracking denotes cracks that run predominantly parallel to the centerline. These cracks may be in the wheel paths, between wheel paths, construction joints, and/or at lane joints such as centerline or mainline/shoulder.

Possible causes include:

- Traffic Loading (Wheel Path)
- Environmental (Frost Heave)
- Construction Related (Joint Cracks)
- Poor Drainage
- Reflection Cracks

Possible treatments:

- Crack Seal
- Chip Seal/Armor Coat
- Patching
- Mill/Fill Asphalt Overlay

Longitudinal Cracking

Random/Block Cracking

Random or Block cracks is a series of rectangular cracks that are interconnected. These cracks usually cover a big area of the road or pavement.

Possible causes include:

- Environmental (Thermal)
- Aging
- Ability of the Asphaltic Material to Absorb Stress (Flexibility)

Possible treatments:

- Crack Seal/Fog Seal
- Chip Seal/Armor Coat
- Mill/Fill Asphalt Overlay

Block Cracking

Transverse Cracking

Transverse cracks are those considered to extend three-fourths of the width of the pavement or more, generally perpendicular to centerline.

Possible causes include:

- Environmental (Thermal)
- Swelling or Shrinkage of the Subgrade
- Reflection Cracks
- Settlement (Trench, Backfill)

Possible treatments:

- Crack Seal
- Chip Seal/Armor Coat
- Patching
- Mill/Fill Asphalt Overlay

Transverse Cracking

Edge Cracking

Edge cracking is similar to alligator cracking only located within 1 to 2 feet of the edge of the pavement. Failure begins at the edge of the pavement and progresses towards the wheel path. Pavement edge distress can result in worsening of the wheel path condition and allowing moisture into the subgrade soils and base materials. Edge cracking also includes longitudinal cracking associated with a widened base.

Possible causes include:

- Traffic Loading/Oversized Vehicles
- Off-tracking
- Insufficient Pavement Structure
- Inadequate Base Support
- Poor Base Drainage
- Construction Related
- Low Shoulder
- High Shoulder Holding Water

Possible treatments:

- Patching
- Mill/Fill Asphalt Overlay
- Trenched Widening
- Remove and Amend or Replace Subgrade

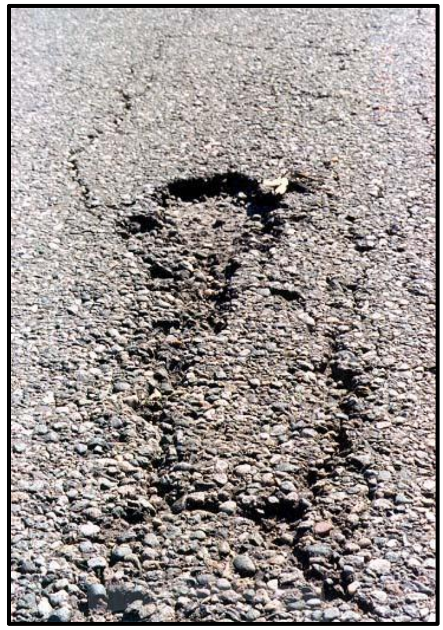
Edge Cracking

Raveling/Weathering

Raveling or Weathering is the progressive wearing away of the pavement from the surface downward caused by the loss of asphalt binder and the dislodging of aggregate particles.

Possible causes include:
- Poor Asphalt Mix Quality
- Asphalt Hardening due to Aging
- Insufficient Asphalt Content
- Improper Construction Methods

Possible treatments:
- Fog Seal
- Chip Seal/Armor Coat
- Microsurfacing
- Patching
- Mill/Fill Asphalt Overlay



Raveling/Weathering

Distortion

Distortion is defined as that distress in the pavement caused by densification, consolidation, swelling, heave, creep or slipping of the surface or foundation.

Possible causes include:
- Inadequate Support or Overloading
- Environmental (Freeze-Thaw)
- Loss of Bonding between Structural Layers
- Insufficient Tack Coat
- Material Defect in Tack Coat
- Static Load (Depressions)
- Soft Binder (Shoving)

Possible treatments:
- Patching
- Mill/Fill Asphalt Overlay

Shoving

Rutting

A rut is a surface depression in the wheel path after pavement layers or subgrade deform from traffic load applications.

Possible causes include:

- Poor Mix Quality
- Incorrect Binder Selection
- Insufficient Support
- Improper Construction Procedures

Possible treatments:

- Mill with Chip Seal/Armor Coat
- Microsurfacing
- Patching
- Mill/Fill Asphalt Overlay

Rutting

Excess Asphalt

Excess asphalt, also called bleeding or flushing, is used to describe a free film of asphalt on the surface of the pavement that creates a smooth, shiny, greasy, and reflective surface. It is usually found in the wheel paths and becomes quite sticky when hot.

Possible causes include:

- Armor Coat with Loss of Aggregate
- Poor Mix Quality
    - Poor Binder
    - Low Air Voids
    - High Binder Content
- Improper Construction Procedures
- Paving Over Excess Asphalt

Possible treatments:

- Mill, then Chip Seal/Armor Coat
- Mill, then Microsurfacing
- Mill/Fill Asphalt Overlay

Bleeding/Flushing

Pumping

Pumping is a pavement issue that occurs when water and fine subbase material are forced out of the pavement through cracks or joints. Pumping can also occur with Rigid Pavements but are more visible with Asphalt or Composite Pavements due to the contrast in color.

Possible causes include:

- Heavy Traffic Loads
- Improper Construction Procedures
- Poor Drainage of the Subgrade
- Poor Load Transfer at Joint
- Small Fractured Concrete Panels
- Poor Joint/Crack Sealing Practices

Possible treatments:

- Joint/Crack Seal
- Patching
- Installing Additional Drains
- Underlying Concrete Repairs
- Mill/Fill Asphalt Overlay
- Pavement and Subgrade Replacement

Pumping

3.3 Rigid Pavement

The most significant and severe distresses in rigid pavements generally occur along joints. Joint deterioration reduces pavement performance substantially and increases the need for maintenance. Other distresses occur within a concrete slab away from joints, leading to a loss of ride quality and structural failure of the pavement.

Types of distresses in Portland cement concrete (PCC) pavements include:

- Joint Distresses
- Faulting
- Transverse Cracking
- Longitudinal Cracking
- Durability Cracking
- Pattern Cracking
- Surface Distress
- Alkali-Silica Reaction (ASR)

For each distress, a number of possible treatments are available. In many instances, multiple types of distress occur so the treatment selected must be appropriate for all distresses that are present.

Joint Distress

Joint distress reflects the deterioration of the concrete within 2 feet on either side of a joint. Breaking or chipping of the pavement joints usually results in fragments with feathered edges.

Possible causes include:

- Expansive internal pressure due to alkali-aggregate reactivity between the cement and the aggregates.
- Expansive internal pressure due to corrosion and deterioration of dowel bars.
- Seized dowel bars combined with thermal expansion (Freeze-Thaw)
- Misaligned Dowel Bars
- Lack of support at joint due to pumping action and voids.
- Overloading

Joint Distress

Possible treatments:

- Partial Depth Repair (if Concrete is Sound)
- Full Depth Repair
- Joint Repair
- Joint/Crack Seal
- Asphalt Overlay

Faulting

Faulting is a differential vertical displacement of a slab or other member adjacent to a joint or crack. Faulting may be either longitudinal or transverse and creates a "step" deformation of the pavement surface. Faulting commonly occurs in transverse joints of Portland cement concrete pavements that do not have load transfer devices (dowels). Usually the "upstream" slab is higher than the "downstream" slab.

Possible causes include:
- Uneven Roadbed Support
- Thermal and Moisture Stresses (Freeze-Thaw)
- Pumping of Slabs due to lack of Dowels
- Insufficient Pavement Structure.

Possible treatments:
- Slab Jacking
- Slab Replacement
- Diamond Grinding
- Dowel Bar Retrofit with Diamond Grinding
- Asphalt Overlay

Faulting

Transverse Cracking

Transverse cracks are cracks that run perpendicular to centerline, resulting in a panel that is broken into two or more pieces. Panels broken into two pieces are rated as Class I and panels broken into more than two pieces are rated as Class II.

Possible causes include:
- Thermal Contractions
- Poor Construction Practices (Curing)
- Long Joint Spacing
- Overloading
- Subgrade Issues
  - Swelling
  - Shrinkage
  - Settlement

Possible treatments:
- Joint/Crack Sealing
- Partial Depth Repair
- Full Depth Repair
- Slab Replacement
- Asphalt Overlay

Transverse Cracking

Longitudinal/Slab Cracking

Longitudinal or Slab cracking is used to describe any unplanned longitudinal or diagonal structural crack that extends through the depth of the slab.

Possible causes include:

- Overloading
- Wide Joint Spacing
- Shallow or Late Joint Sawing
- Pumping of the Subgrade
- Non-uniform Base
- Curling or Warping of Slab
- Culvert or Utility Trench Subsidence

Possible treatments:

- Joint/Crack Seal
- Cross Stitching
- Partial Depth Repair
- Full Depth Repair
- Asphalt Overlay

Slab Cracking

Durability Cracking

Durability cracking, also known as D-Cracking, consists of a series of closely spaced, crescent-shaped cracks that form near joints, corners, or existing cracks.

Possible causes include:

- Thermal and Moisture Stresses
    - Freeze-Thaw cycle deterioration of the aggregates

Possible treatments:

- Partial Depth Repair
- Full Depth Repair
- Slab Replacement
- Asphalt Overlay

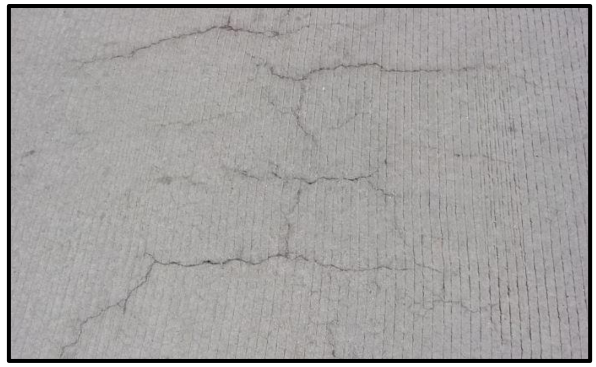
D-Cracking

Pattern Cracking

Pattern cracking refers to occasional to extensive interconnected cracks that may appear anywhere within a panel but do not extend throughout the entire depth of the slab.

Possible causes include:
- Shrinkage Cracks
- Material Related Distress
    - ASR

Possible treatments:
- Penetrating Concrete Sealer
- Asphalt Overlay
- Concrete Surface Milling



Pattern Cracking

Surface Distress

Surface distress is the scaling, spalling, chipping or disintegration of the concrete wearing surface that leads to roughness and poor durability. It is measured in square feet per panel but does not include any distresses within 2 feet of the joint.

Possible causes include:
- Poor Materials
- Poor Construction Practices
    - Too Much Water
    - Overworking
- Thermal and Moisture Stresses
- Corrosion of Reinforcing Steel
- Reinforcing Steel too Close to Surface

Possible treatments:
- Partial Depth Repair
- Full Depth Repair
- Slab Replacement
- Asphalt Overlay

Scaling

Alkali-Silica Reaction (ASR)

Alkali-silica reaction (ASR) is a harmful chemical reaction that leads to cracking, spalling and deterioration in concrete. ASR occurs when reactive silica in certain aggregates reacts with alkalis (sodium and potassium) in the cement paste, forming an alkali-silica gel around the reactive aggregate particles. When exposed to moisture, the gel expands, generating internal pressure that causes tensile cracking around the aggregates. Once cracking begins, additional moisture can enter the concrete, further accelerating the ASR process and worsening the deterioration.

Possible causes include:
- Reactive Aggregates
- No ASR Mitigation
    - No/Not Enough SCMs

Possible treatments:
- Penetrating Concrete Sealer
- Slab Replacement
- Asphalt Overlay

Alkali-Silica Gel in Cracks

See Chapter 7 of this Pavement Design Guide for additional information on alkali-silica reaction in concrete pavement.

3.4 NDOT Evaluation Tools

NDOT has many programs to evaluate pavement conditions. This section will touch on different programs/data retrieval techniques that aid the Designer and the Pavement Design Engineer in evaluating the current pavement conditions and creating a strategy for the Pavement Determination.

Cores/FWD/GPR

Pavement cores are cylindrical samples taken from the surface of a pavement to assess its condition and quality. Cores are evaluated to:

- Determine the pavement's structure and condition.
- Identify defects such as stripped material.
- Test the pavements composition, strength, and thickness.
- Assess the drainage characteristics of the pavement.
- Verify Ground Penetrating Radar (GPR) data.
- Test unbound layers.

Pavement cores are extracted using a special drilling machine with a diamond-tipped core-barrel. Once the cores have been extracted from the pavement, a soil sample is typically taken to evaluate the subgrade properties under the roadway. The holes are then patched with a cold patch material.

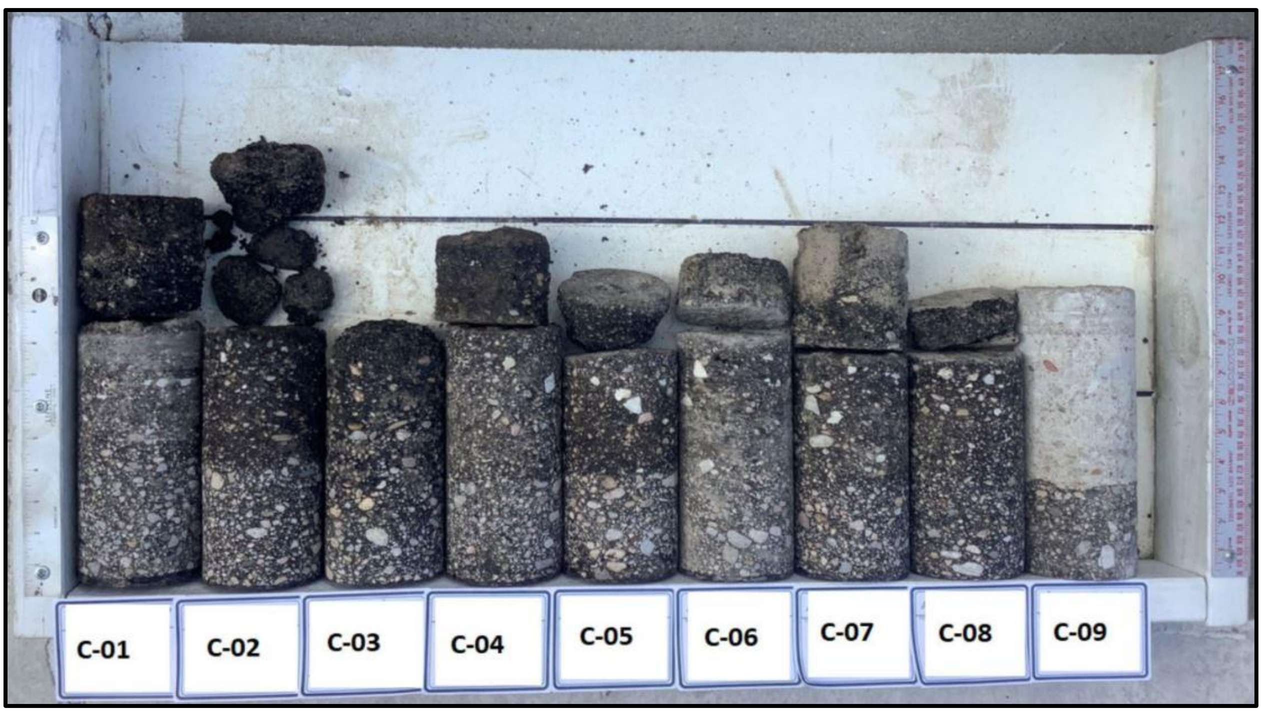


Figure 3.4-1 – Cores from CN 71236

A Falling Weight Deflectometer (FWD) is a non-destructive device that measures the stiffness and structural strength of pavement by dropping a weight onto the pavement and measuring the resulting deflection. The FWD is mounted on a trailer and towed to the pavement being tested. The operator controls the weight dropped onto the pavement and measures the resulting deflection using sensors. The deflection data is used to calculate the pavement stiffness, resilient modulus, and other factors.

Project Information	Pavement Rating Scale		Subgrade Rating Scale	
71236	<150000	Very Weak	<2000	Very Weak
Holbrook - Arapahoe East	150,000-250,000	Weak	2000-3000	Weak
NH-6-3(132)	250,000-350,000	Marginal	3000-5000	Marginal
6/28/2023	350,000-450,000	Strong	5000-7000	Strong
120.63 to 130.50	>450,000	Very Strong	>7000	Very Strong

Reference Post	Pavement Modulus	Pavement Rating	Subgrade Modulus	Subgrade Rating
AVERAGE	**334,095**	**Marginal**	**5,624**	**Strong**
120.999	368,000	Strong	6,600	Strong
120.999	368,000	Strong	6,600	Strong
120.999	354,800	Strong	6,105	Strong
120.999	354,800	Strong	6,105	Strong
121.101	454,500	VS	4,389	Marginal
121.101	454,500	VS	4,389	Marginal
121.101	432,000	Strong	3,960	Marginal
121.101	432,000	Strong	3,960	Marginal
121.204	407,100	Strong	5,610	Strong
121.204	407,100	Strong	5,610	Strong
121.204	403,000	Strong	5,049	Strong
121.204	403,000	Strong	5,049	Strong
121.304	160,400	Weak	3,894	Marginal
121.304	160,400	Weak	3,894	Marginal
121.304	166,400	Weak	3,498	Marginal
121.304	166,400	Weak	3,498	Marginal
121.402	331,400	Marginal	4,950	Marginal
121.402	331,400	Marginal	4,950	Marginal
121.402	320,700	Marginal	4,620	Marginal
121.402	320,000	Marginal	4,653	Marginal
121.499	251,200	Marginal	4,191	Marginal
121.499	251,200	Marginal	4,191	Marginal
121.499	255,600	Marginal	3,894	Marginal
121.499	253,400	Marginal	3,960	Marginal
121.605	239,300	Weak	4,158	Marginal
121.605	241,400	Weak	4,125	Marginal
121.605	240,600	Weak	4,059	Marginal
121.605	239,900	Weak	3,993	Marginal
121.705	240,000	Weak	4,257	Marginal
121.705	240,800	Weak	4,290	Marginal
121.705	232,300	Weak	4,092	Marginal
121.705	230,000	Weak	4,092	Marginal
121.8	240,500	Weak	3,894	Marginal

Figure 3.4-2 – Resilient Modulus Determined from FWD Results, CN 71236

The resilient modulus of a pavement is used as a key parameter to measure the stiffness of a pavement's material. The higher the resilient modulus indicates that the pavement material is stiffer, meaning it will resist deformation more effectively under traffic loads.

Ground Penetrating Radar (GPR) is a non-destructive technology that uses electromagnetic pulses to evaluate the condition of a pavement. GPR provides a continuous profile of the existing pavement condition that can measure:

- Thickness of the pavement structure,
- Identify construction layers and changes in materials,
- Indicate increased moisture content,
- Detect voids, delamination, and other defects,

The thickness results from the GPR data can be used in conjunction with the FWD results to further aid in creating a more accurate existing pavement structure. GPR data is plotted on a graph and compared to the physical cores taken from the same roadway. GPR is a great way to see what is happening to the existing pavement between cores.

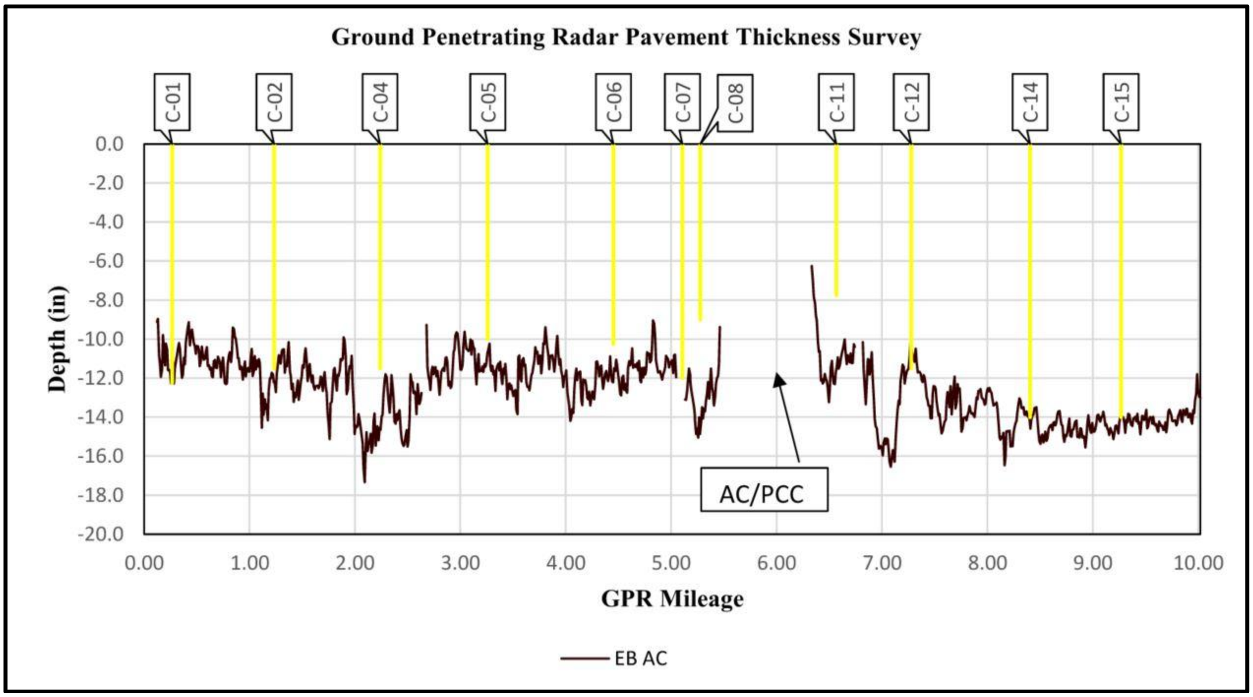


Figure 3.4-3 – Ground Penetrating Radar Results, CN 71236

NDOT Pavement Optimization Program

The Nebraska Department of Transportation utilizes a pavement management data system called the NDOT Pavement Optimization Program (POP). This data management system is controlled and updated by the NDOT Asset Management Division for the purpose of providing existing pavement conditions. The Asset Management Division uses equipment mounted on vans that drive the roads once a year to gather data about existing conditions including rut depths, faulting, International Roughness Index (IRI), etc.

For more information about POP, see the Pavement Optimization Program – User Manual on the NDOT website.

https://dot.nebraska.gov/media/fdhjq3sn/popmanual.pdf

PathWeb

PathWeb is a web-based program that stores pictures of the existing pavement surface and other pavement data collection for review. The information obtained from the Asset Management Section is uploaded into PathWeb for users to view the pavement surface and evaluate pavement conditions. PathWeb is a helpful tool in evaluating current pavement conditions and comparing the current pavement conditions to previous years quickly to track any distresses seen on the pavement surface.

To access PathWeb, please select the hyperlink below.

https://pathweb.pathwayservices.com/ne

As-Built Plans

As-Built Plans are a set of plans that show the final dimensions, materials, and details of a road after construction. They are a record of what was built in previous projects and notes any changes made during construction that differ from the proposed letting plans. As-Built Plans are useful for evaluating the pavement structure, joint layouts, and subgrade materials.

As-Built Plans can be found in OnBase.

C1 Program

The C1 Program is an internal mainframe program. Pavement Design uses the following portions of the C1 Program for evaluating a roadway segment:

- Maintenance Cost Data
- Traffic Counts
- IHI Data (Rutting, Faulting, Cracking, IRI)

The C1 Program also holds As-Built Plan information used to create a Pavement Histogram. The Pavement Histogram is a visual representation of what the pavement structure is comprised, when the material was installed, and where it was placed.

An example of a Pavement Histogram is on the next page.

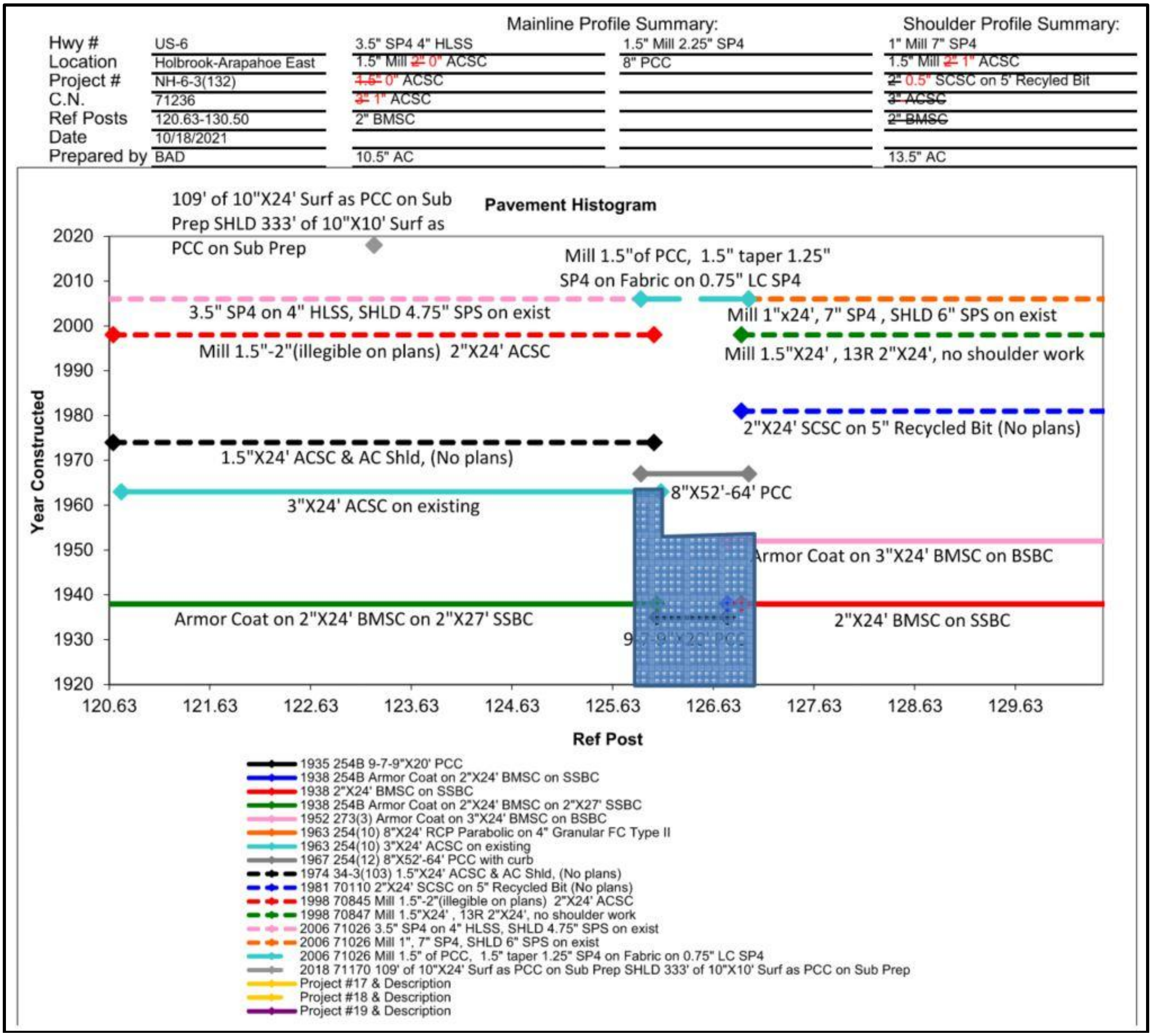


Figure 3.4-4 – Pavement Histogram, CN 71236

3.5 Pavement Serviceability Definitions

The pavement distresses described in this chapter directly correlate to the serviceability of a road as perceived by the traveling public. The NDOT collects a pavement's distress data and correlates it into an algorithmic equation that can be used to define a pavement's serviceability. This serviceability number of a pavement helps the NDOT prioritize pavements that need maintenance, repair, or replacement. Below are several indicators and indexes that the NDOT uses to evaluate a pavement's serviceability.

For additional information about NDOT's Pavement Management Systems, please refer to the State of Nebraska Pavement Management Systems Manual.

https://dot.nebraska.gov/media/gjdmgjmg/pavement-management-systems.pdf

Skid Resistance

Friction is the resistance to movement between two objects in contact. In the context of pavements, it refers to the force generated between a vehicle's tire and the road surface. Skid resistance specifically describes the friction between a tire and the pavement surface when the tire is locked and sliding. It is commonly measured using a locked-wheel tester, which records the force required to drag a non-rotating tire across the pavement. The result is expressed as a skid number (SN) – the higher the skid number, the greater the skid resistance.

Friction can decrease due to factors such as aggregate polishing, loss of surface texture, adverse weather conditions, or roadway alignment. According to *Pavement Interactive*, the typical allowable skid numbers are listed below:

Skid Resistance	
Skid Number	**Comment**
< 30	Take Measures to Correct
≥ 30	Acceptable for Low Volume Roads
31 – 34	Monitor Pavement Frequently
≥ 35	Acceptable for Heavily Traveled Roads

Rutting

Rutting is a permanent depression in the wheel path of a pavement that's caused by repeated traffic loads. Ruts can be narrow or wide, and they can affect multiple paths or just two. Rutting is only found in flexible pavements but can be found in composite pavements with the rutting occurring within the asphaltic concrete layer. NDOT has adopted the following qualitative rutting categories:

Rutting	
Description	**Average Rut Depth**
Good	< 4 mm
Fair	4 mm – 9 mm
Poor	> 9 mm

Faulting

Faulting is a displacement between two adjacent concrete slabs, measured at the common joint. Faulting can result from a combination of factors such as inefficient load transfer at joints, slab pumping, slab settlements, curling, warping, and inadequate base support conditions. NDOT has adopted the following qualitative faulting categories:

Faulting	
Description	**Inches**
Low	⅛ - ¼
Moderate	¼ - ½
High	≥ ½

International Roughness Index (IRI)

The International Roughness Index (IRI) is a standard measurement of how rough a pavement's surface is. A pavement's IRI number is used to evaluate the overall ride quality of a pavement. A lower IRI value indicates a smoother pavement surface, while a higher IRI value indicates a rougher pavement surface.

The IRI is calculated using a mathematical model of a vehicle's suspension. A continuous profile along the pavement is measured and analyzed to summarize qualities of pavement surface deviations that impact vehicle suspension movement. NDOT has adopted the following qualitative IRI categories:

International Roughness Index (IRI)		
Rating	**in/mile**	**mm/m**
Good	< 95	< 1.50
Fair	95 – 170	1.50 – 2.68
Poor	> 170	> 2.68

IRI for Asphaltic Concrete - Resurfacing Projects:

- The intent is to build or resurface the roadway with an IRI ≤ 68 in/mile. All dips and bumps greater than 0.4" shall be corrected.
- Refer to Section 502 of Standard Specifications for Highway Construction for additional information.

IRI for Portland Cement Concrete (PCC):

- The intent is to build a roadway with an IRI between 68 in/mile and 99 in/mile. Surface deviation shall not exceed 0.3" if a profiler is used or 1/8" if a 10' straight edge is used.
- Refer to Section 602 of the Standard Specifications for Highway Construction for additional information.

Nebraska Serviceability Index (NSI)

The Nebraska Serviceability Index (NSI) is a number between 0 and 100 that measures the condition of a section of Nebraska's highway. The NSI is a composite index that uses visual inspection data with automated service data by combining pavement type, pavement age, traffic usage, severity of surface distresses, and the rutting or faulting measurements.

The NSI of a pavement is calculated using a mathematical calculation. Different numerical weights are assigned to pavement distresses based on severity of the pavement distress. An algorithm then calculates all of the distress ratings and produces the NSI Rating Number. NDOT has adopted the following qualitative NSI categories:

Nebraska Serviceability Index (NSI)		
Description		NSI Rating
Excellent	Pavement Like New	90 – 100
Good	Several Years of Service Life Remaining	70 – 90
Fair	Few Years of Service Life Remaining	50 – 70
Poor	Candidate for Rehabilitation	30 – 50
Very Poor	Possible Replacement	0 – 30

The NSI represents the condition of the pavement at the time of the survey and does not necessarily reflect the rate of pavement deterioration. For evaluation of remaining service life of a pavement, the current and historically low NSI for the pavement are evaluated along with the general rate of pavement deterioration as observed overtime.

Present Serviceability Index (PSI)

The Present Serviceability Index (PSI) is a number between 0 and 5 that measures a pavement's rideability. PSI was developed at the AASHTO Road Test to assess how well a roadway is serving the traveling public.

Nebraska does not use roughness alone in deriving the PSI of a pavement. Along with using the International Roughness Index (IRI), compensation is made for cracking, rutting, faulting, joint distress, slab cracking, and repairs. Much like the Nebraska Serviceability Index (NSI), different numerical weights are assigned to pavement distresses based on the pavement distress. An algorithm then calculates all of the distress ratings and produces the PSI Rating Number. NDOT has adopted the following qualitative PSI categories:

Present Serviceability Index (PSI)		
Description		PSI Rating
Excellent	Pavement Like New	4 – 5
Good	Several Years of Service Life Remaining	3 – 4
Fair	Few Years of Service Life Remaining	2 – 3
Poor	Candidate for Rehabilitation	1 – 2
Very Poor	Possible Replacement	0 – 1

4.1 Introduction

The subgrade of a pavement is a critical component of any transportation system. The characterization and evaluation of the soils used for the subgrade are critical to the performance of pavement structures. Subgrade design for a pavement involves understanding the subgrade soil conditions and materialistic properties to ensure the pavement's long-term life.

4.2 Soil Identification and Description

Soil identification is the process of determining the type of soil by examining its physical and chemical characteristics. This can be done in a few ways:

- Visual Inspection – Examine the soil's color, texture, shape, size, and arrangement of particles. Compare the soil's physical characteristics to a standard soil chart or soil description.

- Field Test – Measure the soil's physical and mechanical characteristics such as density, moisture content, strength, compaction, and permeability.

- Laboratory Tests – Perform gradations, Atterberg Limits, and hydrometer tests.

Soil descriptions, classification, and other information obtained during the subgrade exploration and examinations are greatly relied upon throughout the remainder of the investigation program and during the design and construction phase of a project.

To provide uniformity in describing and classifying soils, the Nebraska Department of Transportation has adopted the Unified Soil Classification System (USCS) for soil sampling and classification (Soil and Foundation Workshop Manual NHI # 13212, July 1993).

The Unified Soil Classification System (USCS) classifies soils according to grain size distribution and plasticity. This classification system can be applied to most unconsolidated materials and is represented by a two-letter symbol.

In addition to the USCS soil designation, a detailed description for each material stratum encountered should be included on the log. The description should be sufficiently detailed to provide the engineer with an understanding of the material present at the site.

Two terms that are used in the site exploration process are **IDENTIFY and DESCRIBE.**

- Identification is the process of determining which components exist in a particular soil sample, i.e., gravel, sand, silt, clay, etc.

- Description is the process of estimating the relative percentage of each component and preparing a word picture of the sample.

- Identification and description are accomplished primarily with vision and touch.

During the progression of a boring, the drilling personnel should roughly identify and describe the soils encountered.

A typical soil description procedure is shown on the following pages. This procedure involves visually and manually examining soil samples with respect to texture, plasticity and color. This method presented for preparing a word picture of a sample for entering on a subsurface exploration log applies to soil descriptions made in the field and laboratory.

Definition of Terms for General Soils

• **Boulder**	A rock fragment, usually rounded by weathering or abrasion, with average dimension of 12 inches or more.
• **Cobble**	A rock fragment is usually rounded, or sub rounded, with an average dimension between 3 to 12 inches.
• **Gravel**	Rounded, sub rounded, or angular particles of rock that will pass a 3-inch square opening sieve and be retained on a No. 4 sieve.
• **Sand**	Particles that will pass the No. 4 sieve and be retained on the No. 200 sieve.
• **Silt**	Material passing the No. 200 sieve that is non-plastic and exhibits little or no strength when dried.
• **Clay**	Material passing the No. 200 sieve that can be made to exhibit plasticity (putty like property) within a wide range of water contents and exhibits considerable dry strength.
• **Fines**	The portion of a soil passing a No. 200 sieve.
• **Muck**	Finely divided organic material containing various amounts of mineral soil.
• **Peat**	Organic material in various stages of decomposition.
• **Organic Clay**	Clay containing microscopic size organic matter. May contain shells and/or fibers.
• **Organic Silt**	Silt containing microscopic size organic matter. May contain shells and/or fibers.
• **Coarse-Grained Soil**	Soil having a predominance of gravel and/or sand.
• **Fine-Grained Soil**	Soil having a predominance of silt and/or clay.
• **Mixed-Grained Soil**	Soil having significant proportions of both fine-grained and coarse-grained sizes.

Visual Identification

- **Gravel** — Identify by particle size. The particles may have an angular, rounded, or sub-rounded shape.
- **Sand** — Identified by particle size. Gritty grains that can easily be seen and felt. No plasticity or cohesion. Size ranges between gravel and silt.
- **Silt** — Identified by behavior. Fines that have no plasticity. It is difficult to roll into a thread and will easily crumble. Has no cohesion. When dry, it can be easily broken by hand into powdery form.
- **Clay** — Identified by behavior. Fines that are plastic and cohesive when in a moist or wet state. It can be rolled into a thin thread that will not crumble. When dry, it forms hard lumps that cannot be readily broken by hand.
- **Muck** — Black or dark brown finely divided organic material mixed with various portions of sand, silt, and clay. May contain minor amounts of fibrous material such as roots, leaves, and sedges.
- **Peat** — Black or dark brown plant remains. The visible plant remains range from coarse fibers to finely divided organic material.
- **Organic Clay** — Dark gray clay with microscopic size organic material dispersed throughout. May contain shells and/or fibers. It has weak structure, which exhibits little resistance to kneading.
- **Organic Silt** — Dark gray silt with microscopic size organic material dispersed throughout. May contain shells and/or fibers. It has weak structure, which exhibits little resistance to kneading.
- **Fill** — Man-made deposits of natural soils and/or waste materials.

Soil Sample Identification Procedure

1) Is sample coarse-grained, fine-grained, mixed-grained or organic?
   If mixed-grained, decide whether coarse-grained or fine-grained predominates.

2) What is the principal component?
   Use a noun in the soil description. i.e., Sand, Silt, Clay.

3) What is the secondary component?
   Use as the adjective in the soil description. i.e., Silty Sand, Silty Clay, Clayey Silt.

4) Are there additional components?
   Use as additional adjectives. i.e., Silty Sand Gravelly, Clayey Silt Sandy.

Examples of Descriptions of the Soil Components

- **Sand** — Describes a sample that consists of both fine sand and coarse sand particles.
- **Gravel** — Describes a sample that consists of both fine and coarse gravel particles.
- **Silty Fine Sand** — Major component fine sand, with non-plastic fines.
- **Sandy Gravel** — Major component gravel size, with fine and coarse sand. May contain small amounts of fines.
- **Gravelly Sand** — Major component sand, with gravel. May contain small amounts of fines.
- **Gravelly Sand, Silty** — Major component sand, with gravel and non-plastic fines.
- **Gravelly Sand, Clayey** — Major component sand, with gravel and plastic fines.
- **Sandy Gravel, Silty** — Major component gravel size, with sand and non-plastic fines.
- **Sandy Gravel, Clayey** — Major component gravel size, with sand and plastic fines.
- **Silty Gravel** — Major component gravel size, with non-plastic fines. May contain sand.
- **Clayey Gravel** — Major component gravel size, with plastic fines. May contain sand and silt.
- **Clayey Silt** — Major component silt size, with sufficient clay to impart plasticity and considerable strength when dry.
- **Silty Clay** — Major component clay, with silt size. Higher degree of plasticity and higher dry strength than clayey silt.
- **Fat Clay** — Major compound clay with high degree of plasticity. Absorbs large amounts of water and can cause pavement distress due to shrink/swell characteristics.

Other Information for Describing Soils

1)	Color of the Sample	Brown, Gray, Red, Black, Yellow, Blue, Green, etc.
2)	Moisture Condition	Dry, Moist, Wet (Saturated).
3)	Examples of Materials	Sand, Silt, Clay, Gravel, Sandstone, Ironstone, Topsoil, Organic, Ogallala, Shale, Limestone, etc.
4)	Examples of Descriptions	Slightly, Contains, Considerable, Decayed, Grains, Clean, Clayey, Silty, Fairly, Numerous, Fractured, Weathered, Trace, Eroded, Mottled, Cemented, Extremely, Intermittent, Compact, etc.

Examples of Complete Soil Descriptions

- Light Gray Silty Clay, moist, plastic, with ½-inch layers of wet gray silt
- Red Brown Clayey Silt, moist, plastic
- Brown Silty fine Sand, wet, non-plastic
- Gray Sandy Gravel, Clayey, moist, low plastic
- Fill – Brown Sandy Gravel, with pieces of brick and cinders, wet, non-plastic
- Dark Gray Organic Clay, with shells and roots, moist, plastic

Definition of Terms for Nebraska Soils

- **Topsoil**
  Surface soil that supports vegetation. Typically, loamy and dark colored, and generally described as brown silty clay.

- **Buried Topsoil**
  Former surface soil buried beneath later deposits. It may contain organic material and exhibit variable strength and compressibility.

- **Redeposited Topsoil**
  Topsoil transported downslope and deposited on terraces or bottomlands by sheet erosion from adjacent uplands. Material properties may vary depending on the source material and degree of mixing.

- **Subsoil**
  A compact subsurface zone formed by infiltration and accumulation of fines leached from overlying topsoil. It is generally described as silty clay.

- **Claypan**
  A dense, impervious clay layer formed in areas of poor drainage or slow runoff. Claypans restrict water movement and may create drainage and stability problems.

- **Buried Subsoil**
  Subsoil formed during a previous geologic period and subsequently buried by later deposition.

- **Redeposited Subsoil**
  Subsoil eroded from its original location and redeposited at a lower elevation. It is typically disturbed and less uniform than undisturbed subsoil.

- **Peorian Loess (Silty Clay to Clayey Silt)**
  A common wind-deposited parent material found throughout eastern, central, and southwestern Nebraska. Peorian loess exposed slopes commonly stand in near-vertical positions. Settlement may occur under both dry and wet conditions. Embankment stability is generally good when dry but may require staged construction when wet. Typically light brown, tan, or buff in color.

- **Redeposited Peorian**
  Loess that has eroded and accumulated out of its original position, commonly as talus at the base of exposed loess slopes. The near-vertical slope stability characteristic of intact loess is generally lost.

- **Sandy Peorian**
  Loess intermixed with sand in transitional areas between the Sandhills and the typical Peorian mantle.

- **Loveland Loess (Silty Clay)**
  An older loess deposit characterized by a reddish tint and generally heavier texture than Peorian loess. It may contain varying amounts of sand. A buried solum may occur at the contact between Loveland and Peorian deposits and is often visible in fresh roadway cuts.

- **Redeposited Loveland**
  Loveland loess displaced from its original position by slumping or erosion.

- **Sandy Loveland**
  A sandy textural phase of Loveland loess.

- **Glacial Till (Silty Clay)**
  Dense glacial deposits consisting primarily of clay intermixed with sand, silt, gravel, and rock fragments. Material properties and composition vary widely depending on the glacial source. Kansan Till is typically tan to orange, while Nebraskan Till is generally gray.

- **Glacial Gravel**
  Mixed deposits of sand, gravel, cobbles, and boulders transported by glaciers.

- **Glacial Sand**
  Localized sand deposits associated with glacial till.

- **Fine Sand and Natural Sand**
  Wind-blown dune sands common in the Nebraska Sandhills and water-deposited fine sands occurring throughout the state. Natural sand typically contains more fines than fine sand. Settlement is generally minimal and occurs rapidly, and embankment stability is typically not a problem. Beware of areas where sand is on top of shale if the shale is not flat. Water may be trapped above the shale and cause instability if the shale surface is sloped.

- **Brule Clay (Silty Clay to Clayey Silt)**
  A massive, compact, pinkish silty clay formation occurring primarily west of North Platte. Thin volcanic ash layers may occasionally be present. Settlement is generally minimal and embankment stability is good when dry. Wet conditions may lead to erosion, traffic sensitivity, and instability.

- **Redeposited Brule**
  Slumped and weathered Brule formation material. It is typically loose and mellow, with engineering behavior similar to loess in appearance and characteristics.

- **Ogallala Formation**
  A predominantly sand and gravel formation occurring primarily west of North Platte. Interbedded layers of sand, gravel, stone, and lime may be encountered. The formation is often cemented and may contain varying percentages of clay, silt, gravel, lime and sand. Settlement is minimal and embankment stability is good as long as it is dry, although erosion may be a concern.

- **Pierre Shale (Silty Clay to Fat Clay)**
  A dark gray massive clay containing some chalk, bentonite, thin sandstones, and occasionally concretions. It is highly plastic, a very poor subgrade material, and prone to slope instability. Increasing moisture greatly reduces shear strength and contributes to landslides. Most major slides in Nebraska involve shale formations. Benching is especially important where fills are placed on shale slopes. Settlement is typically minimal, but embankment stability is poor.

- **Carlile Shale**
  A gray shale containing layers of fine-grained sandstone. It is not commonly encountered at depths associated with Nebraska highway construction.

- **Graneros Shale**
  A dark gray plastic shale containing thin calcareous layers, sand, sandy shale, and coal-like material.

- **Dakota Sandstone and Dakota Shales**
  A formation containing fine sand ranging from loose, clean deposits to highly cemented sandstone and "ironstone". Cemented zones may require blasting or ripping for excavation. Interbedded shale layers are common and typically consist of fine-grained silty clay shales with high swell potential and poor subgrade characteristics. Dakota shales often have a glossy or soapy appearance and may be multicolored.

- **Alluvial Silts, Sands and Clays**
  Water-deposited materials occupying stream floodplains, including variable deposits of silt, sand clay, muck and peat. These soils are typically saturated, have high settlement potential, and exhibit poor embankment stability. Pore pressure may create stability concerns. Two-stage grading and/or wick drains work well in these soils. Surcharging may create additional stability problems. Excavation should be considered where weak layers are less than 10 feet thick.

4.3 Nebraska Group Index

The Nebraska Group Index (NGI) is a soil classification system used to assess the stability of soils for use of subgrades material in the construction of roadways and other paved surfaces. The NGI helps engineers determine the quality of the soil for supporting the load-bearing capacity of the pavement structure.

The Nebraska Group Index is calculated based on factors such as:

- Soil Plasticity – The ability of the soil to deform without cracking or breaking, which can indicate its potential for compaction and stability under load.

- Soil Gradation – The distribution of particle sizes in the soil, which affects the drainage and stability.

The Nebraska Group Index uses a range of numbers to represent a different soil type found in Nebraska. These soil types can be an indication of how structurally sound the subgrade material can be. The higher the NGI value, the more potentially problematic the soil is for supporting the pavement structure.

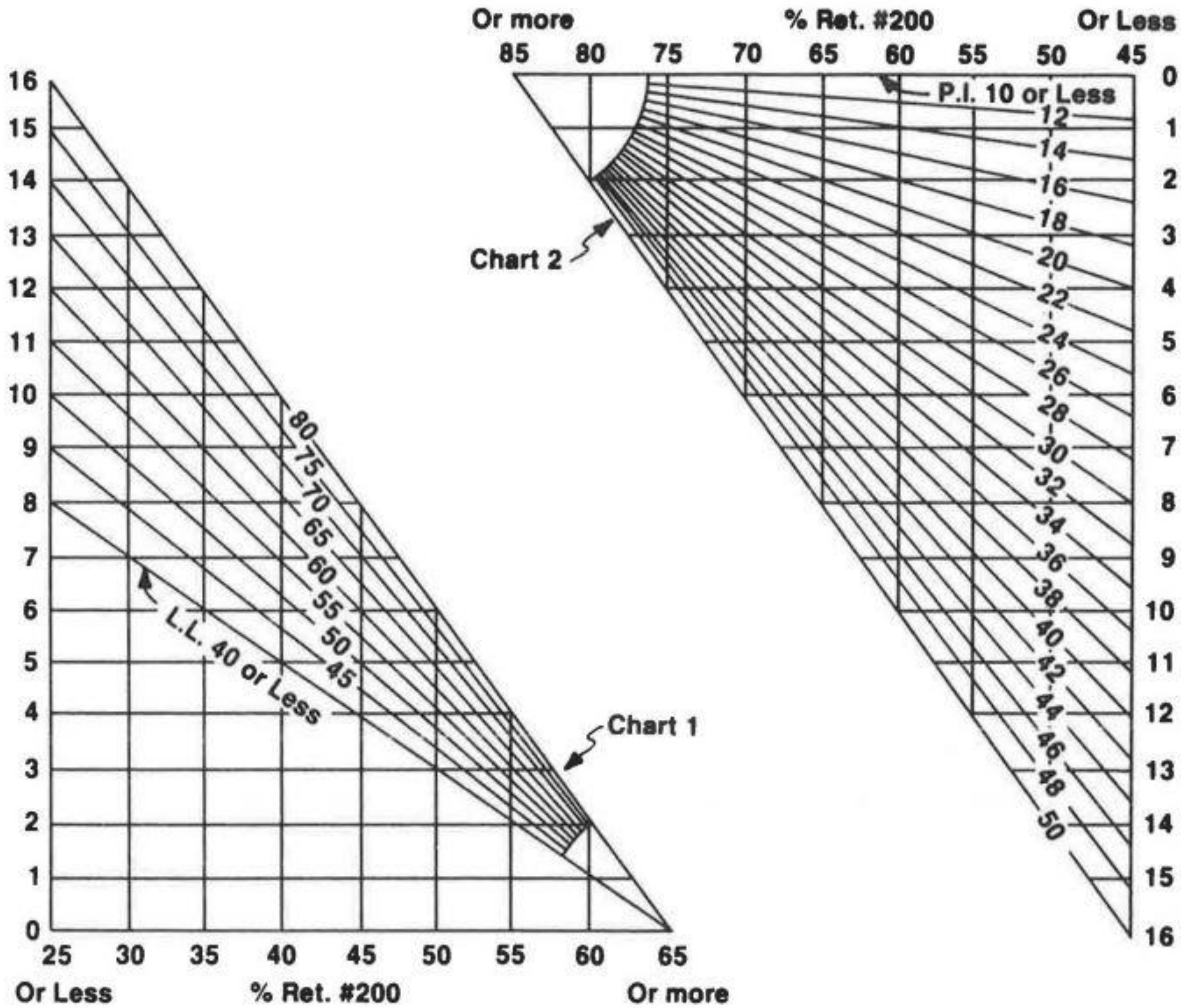
Soil Type	NGI
Gravel	-4 to -2
Fine Sand	-1 to 1
Sandy Silt	2 to 7
Loess	8 to 12
Loess/Till	13 to 14
Till	15 to 21
Shale	22 to 24

- NGI between -4 – 4 – Soils with a low NGI value are considered good for pavement construction. They are generally stable, well-drained, and have a low plasticity. These soils require minimal treatment or reinforcement for pavement support.

- NGI between 5 – 7 – Soils in this range have moderate stability but may need some improvements to support the pavement adequately. The subgrade may need additional stabilization or better drainage to prevent future distress.

- NGI between 8 – 21 – Soils with NGI values in this range are generally unsuitable for supporting heavy truck traffic without significant treatment. These soils can lead to instability in the presence of water. Soil improvement or a stronger pavement design should be considered.

- NGI > 21 – Soils with a high NGI value are poor for supporting pavements. They tend to be highly plastic or contain excessive fines, leading to high potential for failure under load. These soils may require stabilization, removal, or the use of high-quality pavement materials to ensure durability.



Group Index {
Positive Values - Sum of Readings on Vertical Scale of Charts 1 & 2.
Negative Values - Use Chart 3.

	Well Graded Gravel Base	Clean Coarse Sand	Clean Fine Sand	Loamy Coarse Sand	Loamy Fine Sand	Loamy Very Fine Sand
Group Index	- 4	- 3	- 2		- 1	0
% Ret. #10	40 Min.					
% Ret. #40	60 Min.	35 Min.	34 Max.	35 Min.	34 Max. 10 Min.	9 Max.
% Ret. #200	85 Min.	85 Min.	85 Min.	84 Max. 65 Min.	84 Max. 65 Min.	84 Max. 65 Min.
P.I.	4 Max.	4 Max.	4 Max.	10 Max.	10 Max.	10 Max.

The first group from the left into which the test data will fit is the correct classification.

Figure 4.3-1 – Nebraska Group Index Chart

Using the Nebraska Group Index Charts (Figure 4.3-1), an NGI value can be determined knowing the soil's Gradation and Atterberg Limits.

Example:

A preliminary soil sample has been taken from a future project. The soil is to be analyzed to determine what type of soil is located on the project and whether that soil type needs additional structural support.

The soil sample has a gradation consisting of:

2% retained on the No. 10 Sieve
14% retained on the No. 40 Sieve
20% retained on the No. 200 Sieve

The soil sample has an Atterberg Limit consisting of:

Liquid Limit of 8
Plastic Index of 12

Using Figure 4.3-1 on the previous page,

Chart 1 – 20% retained on the No. 200 Sieve and a Liquid Limit of 8
Chart 1 Value = 8

Chart 2 – 20% retained on the No. 200 Sieve and Plastic Index of 12
Chart 2 Value = 0.8

The Nebraska Group Index is the summation of Chart 1 and Chart 2

NGI = Chart 1 + Chart 2
= 8.0 + 0.8
= 8.8
= Loess Type Soil

Since the NGI is between 8 and 21, the existing soil for the future project should receive a stabilization strategy to structurally support the pavement structure and the traffic loadings.

For more information about the Nebraska Group Index, please visit the following links.

- Proctor Compaction Testing, NDOR Research Project SG-10, 2008
  https://dot.nebraska.gov/media/bsaca43t/final-report-p313.pdf

- Dynamic Testing of Nebraska Soils and Aggregates, G. Woolstrum, 1989
  http://onlinepubs.trb.org/Onlinepubs/trr/1990/1278/1278-004.pdf

4.4 Subgrade Stabilization

Subgrade stabilization is the modification of an existing soil or subgrade to provide structural stability for improved long-term performance of the pavement structure. Some of the reasons for needing a stabilized subgrade are as follows:

- To create a uniform base for the pavement structure.
- Increase the shear strength of the soil.
- Reduce the swelling caused by wetting and shrinking.
- Increase the soil's load-bearing capacity.
- Prevent soil cracking caused by a decrease in moisture content.

Subgrade stabilization can be achieved through several methods, including:

- Mechanical Combination: Combining natural soil and stabilizing material to create a homogeneous mix. This can be achieved by compacting the existing soil or mixing different soils/aggregates together to create a dense, well-graded blend of subgrade material that improved the subgrade's performance. An example of this includes a Soil Aggregate Base Course or a Foundation Course (Regular).

- Chemical Additives: Adding chemical compounds to the soil to modify a subgrade's properties. Typically, Cement, Fly Ash, or Lime are mixed into the soil to improve the performance of the subgrade.

- Geosynthetic Reinforcement: Using geosynthetic elements to create a well-compacted platform that can support uniform loads. Geosynthetic reinforcement provides constructability and access over very soft soils.

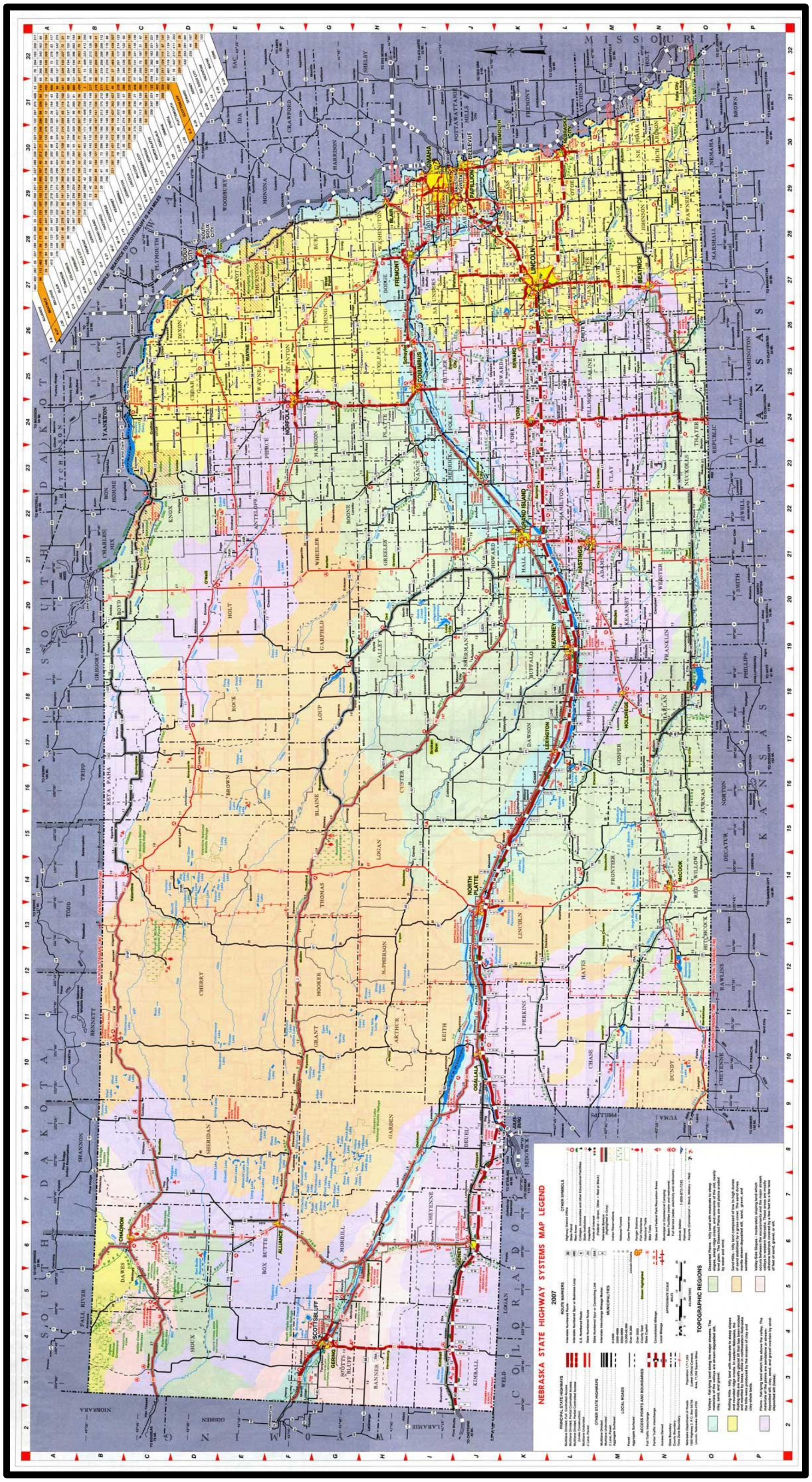


Figure 4.4-1 – Nebraska Soil Map

The Nebraska Department of Transportation uses several different stabilizing techniques depending on the characteristics of the existing soils and the needs of the pavement structure.

Subgrade Preparation

Subgrade preparation is the process of preparing the natural soil beneath the pavement to ensure it can adequately support the construction equipment, pavement structure, and traffic loads. This process involves scarifying and shaping the area to the desired grade and slope, and compacting the soil to achieve strength and stability. A properly prepared subgrade provides a uniform foundation, minimizes future settlement, and contributes to the long-term performance and durability of the pavement.

General guidance for Subgrade Preparation includes:

- The upper 6” of the subgrade shall be prepared for paving operations.
- The upper 12” of the subgrade shall be prepared for paving operations when the roadway grade is reconditioned.
- Subgrade Preparation should extend 3’ beyond the edge of pavement to support the paving equipment during construction.
- Subgrades are compacted to the requirements determined from the Proctor Test results.

Subgrade Stabilization

Subgrade Stabilization is the process of preparing and improving the natural ground (granular material) to provide a stable and supportive working platform for construction equipment. This typically involves mechanical methods such as adding and scarifying cohesive soils into granular soil to improve the load-bearing strength of a subgrade or mixing commercial binders into granular soils to reduce the risk of deformation or failure under construction equipment.

General guidance for Subgrade Stabilization includes:

- Subgrade Stabilization is used as a working platform for construction equipment.
- Construction equipment with tracks do not require subgrade stabilization.
- Construction equipment with tires may require subgrade stabilization where it is anticipated to have areas of repeated construction traffic loadings.
    - Haul Roads
    - Project Entrance/Exits
    - Placing material in front of a paver
- The upper 6” of subgrade shall be stabilized to support construction equipment.
- Clay Binder may be used for granular materials.
- A mix design using Clay Binder will be created by Materials & Research based on Soil Lab testing.
    - Typical values for estimation: 15% passing #200 (subgrade + binder) AC Laydown.
18% passing #200 (subgrade + binder) PCC Slip forming.
- Foundation Course that is pushed out ahead of the paver is often used as an alternative to subgrade stabilization.

Geosynthetics

Geosynthetics are synthetic materials used to enhance the performance and longevity of subgrades in pavement systems. They serve multiple functions, including reinforcement, separation, filtration, and drainage, which collectively improves load distribution and reduce deformation under traffic loads. Geotextiles and geogrids are two of the most used geosynthetics, each fulfilling distinct roles. Geotextiles are permeable fabrics, typically made from polypropylene or polyester, and are primarily used for separation, filtration, and drainage applications. In contrast, geogrids are grid-like structures – often made from polymers such as polyethylene or polypropylene – designed to provide tensile reinforcement by interlocking with soil or aggregate.

Geotextiles

- Woven Geotextiles – Woven Geotextiles are manufactured by interlacing two sets of fibers, similar to traditional textile weaving. This process produces a strong, durable fabric characterized by high tensile strength and low elongation. Woven geotextiles are especially effective in applications where reinforcement and separation are essential, such as roadway construction and embankment stabilization. However, due to their tightly woven structure, they typically have a lower permeability, which makes them less suitable for filtration and drainage functions compared to nonwoven alternatives.

- Nonwoven Geotextiles – Nonwoven Geotextiles are manufactured by bonding fibers together through mechanical entanglement, chemical bonding, or thermal processes. These fabrics have a random, felt-like structure that provides higher permeability and enhanced filtration capabilities. Nonwoven geotextiles are widely used in applications such as drainage, filtration, and erosion control. Although they typically have lower tensile strength than woven geotextiles, their hydraulic properties make them ideal for situations where water flow and soil retention are critical.

The Apparent Opening Size (AOS) of a geotextile refers to the approximate diameter of the largest soil particle that can pass through the fabric. It serves as an indicator of the geotextile's pore size and its effectiveness in filtering soil particles. AOS is typically determined through a dry sieve test, in which glass beads are used to assess the fabric's permeability. The result is usually reported at the sieve opening size – or the diameter of the glass beads – at which 90% of the geotextile's openings are equal to or smaller than that size.

Geotextile Apparent Opening Size (AOS):

- Coarse Granular Material
  Suitable for soils with 0% to 15% passing the No. 200 sieve. Ideal for use with coarse-grained materials with minimal soil retention is needed. The maximum AOS is 0.43 mm.

- Fine Granular Material
  Designed for soils with 15% to 50% passing the No. 200 sieve. Provides balanced filtration and soil retention in typical soil conditions. The maximum AOS is 0.25mm.

- Fine-Grained Soil
  Best suited for soils with more than 50% passing the No. 200 sieve. These geotextiles have smaller pore sizes to effectively retain fine-graded materials. The maximum AOS is 0.22mm.

Geotextiles are categorized into strength classes to ensure their appropriate use across various construction conditions and applications. The selection of a suitable strength class depends on several factors, including project specific requirements, expected loads and stresses, construction techniques, and environmental conditions. In general, lower strength classes are intended for lighter loads and less demanding situations, whereas higher classes are designed to withstand more rigorous conditions, such as those found in steep slopes or in heavy load-bearing applications.

Geotextile Strength Classes:

- Class I – Engineered for the most demanding conditions, including areas with heavy rock fill or where structural reinforcement is essential. These geotextiles are typically strong woven fabrics, offering maximum durability and performance.

- Class II – Designed to provide a balance between strength and flexibility. Commonly used for functions like separation, filtration, and moderate reinforcement. These are versatile and suited to a wide range of constriction needs.

- Class III – Suitable for light-duty uses such as weed barriers or basic drainage systems. These geotextiles are often made from nonwoven or less robust woven fabrics, offering minimal support.

Geogrids

- Uniaxial Geogrids – A uniaxial geogrid is a type of geogrid that provides tensile stiffness and strength primarily in one direction. Uniaxial geogrids are commonly used in retaining walls, steel slopes, and embankments where loads are predominantly unidirectional. Their high tensile strength in one direction makes them ideal for applications requiring controlled, directional reinforcement.

- Biaxial Geogrids – A biaxial geogrid is a type of geogrid that provides tensile stiffness and strength in two orthogonal directions. Biaxial geogrids are commonly used in subgrade stabilizations and working platforms, where loads are applied in multiple directions. Their balanced tensile strength in both directions provides effective confinement and load distribution for planar applications.

- Triaxial Geogrids – A triaxial geogrid is a type of geogrid that provides tensile stiffness and strength in multiple directions. Its unique triangular structure enhances aggregate interlock and confinement – leading to improved structural performance of the stabilized layer. Triaxial geogrids are commonly used in subgrade stabilizations and embankment applications where multidirectional loads are present. Their high radial stiffness allows for efficient stress transfer from the aggregate to the geogrid.

- There are other unique geometries developed by manufacturers that go beyond these three types of geogrids.

For more information on geotextile and geogrid selection, please review the current version of AASHTO M288 – Standard Specification for Geosynthetic Specification for Highway Applications.

Stabilized Subgrade Type Fly Ash

Stabilized Subgrade Type Fly Ash involves incorporating Fly Ash, a byproduct of coal combustion, into the soil to improve the subgrade's uniformity, durability, and load-bearing capacity. When mixed with soil, Fly Ash undergoes a pozzolanic reaction with the water within the soil that gradually binds soil particles together, enhancing the soil's load-bearing capacity. Fly Ash may also be used as a drying agent for saturated soils while causing a slight reduction of the soil's Plasticity Index. This method is particularly effective in lean clay and silty soils and can help create a more uniform and durable foundation for pavements.

General guidance for Stabilized Subgrade Type Fly Ash includes:

- The upper 8" of subgrade shall be stabilized for paving.
- Fly Ash is used to create uniformity of the subgrade with a marginal increase in strength.
- Stabilized Subgrade Type Fly Ash is typically used for soils with a Plasticity Index between 10 and 20.
- It is not recommended that Fly Ash is used for granular material as the Fly Ash will leach out overtime when water is present.
- Class "C" Ash is typically used for Stabilized Subgrade Type Fly Ash.
- Mix Designs for Stabilized Subgrade Type Fly Ash are created by Materials & Research based on Chemistry and Soil Lab Testing.
- Lab testing requires a 7-day moist curing and 24 hours room temperature drying period prior to performing the compressive tests.
- Compressive test results typically target between 100 psi and 250 psi depending on the different soil and Fly Ash sources.
- Fly Ash is typically applied at a rate between 10% and 12% by volume.
- The required moisture content for the Stabilized Subgrade Type Fly Ash is determined by the mix design and is often 1% below the Optimum Moisture Content of the virgin soil.

Stabilized Subgrade Type Cement

Stabilized Subgrade Type Cement involves mixing Portland Cement with the existing soil to improve the subgrade's uniformity, durability, and strength. When mixed with soil and water, Cement begins its' hydration process and binds soil particles together to form a strong and stable subgrade that distributes the traffic loads more effectively. However, using too much Cement in a stabilized subgrade can make the subgrade layer overly rigid, leading to shrinkage and reflective cracking in the pavement above. The Stabilized Subgrade Type Cement method is effective in both silty and granular soils as Cement is more resilient to leaching over time than Fly Ash when water is present.

General guidance for Stabilized Subgrade Type Cement includes:

- The upper 8" of subgrade shall be stabilized for paving.
- Cement is used to create uniformity of the subgrade with a major increase in strength.
- Stabilized Subgrade Type Cement is typically used for soils with a Plasticity Index under 20.
- Cement can be used for both silty and granular materials.
- Type IL Cement is typically used for Stabilized Subgrade Type Cement.
- Mix Designs for Stabilized Subgrade Type Cement are created by Materials & Research based on Chemistry and Soil Lab Testing.
- Lab testing requires a 7-day moist curing and 24 hours room temperature drying period prior to performing the compressive tests.
- Compressive test results should have a Maximum Target Value of 350 psi to avoid shrinkage and reflective cracking of the pavement above.
- Cement is typically applied at a rate between 3% and 5% by volume.
- The required moisture content for the Stabilized Subgrade Type Cement is determined by the mix design and is often 1% below the Optimum Moisture Content of the virgin soil.

Stabilized Subgrade Type Lime

Stabilized Subgrade Type Lime involves the application of lime to natural soils to improve their engineering properties for construction purposes. Lime stabilizations are particularly effective in clay-rich soils and are commonly used to create durable, stable foundation for roads. The Calcium from the lime reacts chemically with the Silica and Alumina from the clay particles in the soil to reduce the plasticity index. This process not only enhances the soil's uniformity but also improves its workability and resistance to water-related damage.

- The Liquid Limit of the soil generally decreases meaning that the soil requires less water to reach a liquid state, making it less plastic and more stable.

- The Plastic Limit generally increases meaning that the soil can retain more water at a plastic state before cracking due to the cementing effect of the lime particles.

- This results in the Plasticity Index decreasing due to the lime's ability to bind clay particles together, thus reducing the range of moisture content where the soil behaves plastically.

$$\boldsymbol{\mathit{Liquid\ Limit}} - \boldsymbol{\mathit{Plastic\ Limit}} = \boldsymbol{\mathit{Plasticity\ Index}}$$

- NDOT performs an Atterberg Test to the modified soil to confirm if the lime has changed the soil's properties to Non-Plastic.

To determine how much lime is needed to stabilize the soil, an Eades and Grim Test is performed. An Eades and Grim Test is a laboratory test used to determine the optimum amount of lime needed to stabilize a soil by measuring the pH level of the soil mixed at different percentages by volume of lime. A pH balance of at least 12.40 means that the soil will react with the lime to produce a pozzolanic reaction strong enough to modify the clay particle structure.

General guidance for Stabilized Subgrade Type Lime includes:

- The upper 8" of subgrade shall be stabilized for paving.
- Stabilized Subgrade Type Lime is typically used for soils with a Plasticity Index ≥ 20. These soils can typically be found on the eastern side of Nebraska. See Figure 4.4-1.
- Lime is used to significantly reduce a soil's frost heave potential and shrinkage and swelling ability with a slight increase in strength.
- Hydrated Lime (Slaked Lime) or Pebble Quick Lime may be used.
- Hydrated Lime Slurry is not allowed due to the slurry ponding in the wheel ruts during application.
- Mix designs for Stabilized Subgrade Type Lime are created by Materials & Research based on Chemistry and Soil Lab testing.
- Lime is typically applied at a rate between 4% and 6% as determined by an Eades and Grim test.
- The required moisture content for the Stabilized Subgrade Type Lime is determined by the mix design and is often between 2% and 4% over the Optimum Moisture Content of the virgin soil.

4.5 Drainage Layer Design

In pavement construction, an effective drainage layer improves the performance and longevity of the roadway. These layers are designed to remove water that infiltrates the pavement structure and prevent moisture accumulation, which can weaken the underlying subgrade materials. Drainage layers are typically made of permeable materials to facilitate the efficient movement of water away from the pavement. A well-designed drainage system not only enhances the structural integrity of the pavement structure, but also reduces maintenance costs and improves ride quality by minimizing surface distresses caused by moisture-related issues.

Stability and Permeability

Stability and Permeability are two fundamental properties that must be balanced in the drainage layers within pavement systems.

- Stability – Stability refers to the ability of drainage materials to maintain their structural integrity under the loads imposed by traffic and the overlying pavement layers. A stable drainage layer resists deformation and displacement, ensuring long-term performance and support of the pavement structure. Materials selected for improving the stability of a drainage layer include particles with higher angularity and soundness to create good particle interlock structure.

- Permeability – Permeability measures a material's ability to allow water to pass through it efficiently. The higher the permeability of a drainage layer, the faster water can travel through the material. Permeability is essential for removing infiltrated water and preventing moisture-related damage, such as freeze-thaw cycles or loss of structural support due to subgrade saturation.

Achieving an optimal balance between stability and permeability is important. Materials used in a drainage layer must be permeable enough to drain water effectively while being stable enough to support the pavement structure without rutting or shifting.

- Good Stability – Poor Permeability
  - Densely packed layer of material with very little to no void space between particles.
  - Particle interlock is achieved.
  - This layer will be supportive of construction equipment and the pavement structure.
  - Water will not flow through the aggregate layer easily.
  - Examples of this base material include Foundation Course (Regular) and Soil Aggregate Base Course (SABC).

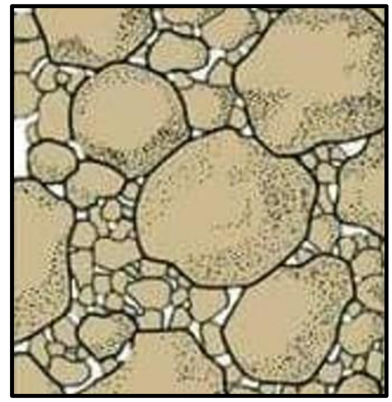


Figure 4.5-1 – Stable Base Layer Gradation

- Poor Stability – Good Permeability
  - Large void spaces between material particles.
  - Particle interlock is not achieved.
  - This layer will not be supportive of construction equipment or the pavement structure.
  - Water can flow easily through the material layer.
  - Examples of this base include Windblown and Alluvial Sand materials. These materials are found in the Sandhills of Nebraska.

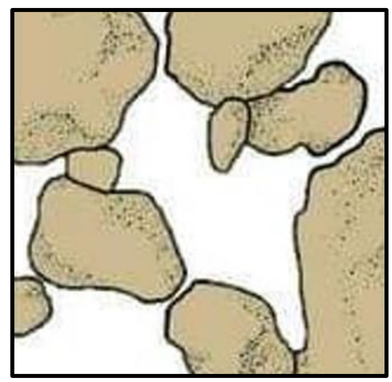


Figure 4.5-2 – Permeable Base Layer Gradation

- Good Stability – Good Permeability
  - A well-graded material layer with void spaces between particles.
  - Particle interlock is achieved.
  - This layer will be supportive of construction equipment and the pavement structure.
  - Water will flow through the material layer.
  - See Foundation Courses below for examples of good stability/good permeability base materials.

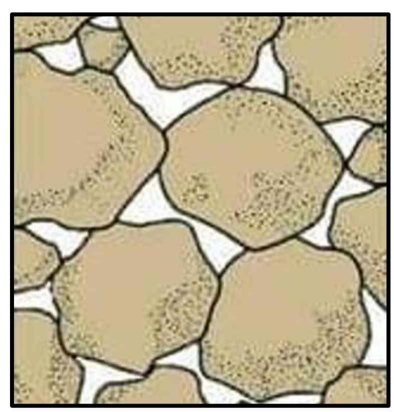


Figure 4.5-3 – Stable and Permeable Base Layer Gradation

4.6 Foundation Course

In Nebraska, the drainage layer within the pavement structure is referred to as the Foundation Course. This layer is typically located below the pavement and above the subgrade. While the Foundation Course is most commonly used under concrete pavements, it can also be used under asphaltic concrete pavements in certain situations.

Nebraska DOT's Foundation Course serves multiple purposes:

- Serve as a working platform for construction equipment.
- Distributes traffic loads to the underlying subgrade.
- Prevents subgrade distresses from propagating into the pavement above.
- Promotes the efficient removal of water from the pavement structure.

Historical Nebraska Foundation Courses

Over the years, NDOT has refined its foundation course gradation to improve both stability and permeability beneath pavements. However, this was not always the case. Early foundation courses were primarily used as construction platforms and were not designed to be drainable. Aggregates were often mixed with soil and soil binders to create a stable base, but these mixtures lacked permeability. Below is a list of foundation course types historically used in Nebraska.

- **Soil Aggregate Base Course:**
  The Soil Aggregate Base Course (SABC) was a foundation course composed of a single aggregate or a combination of gravel and sand aggregates, a soil binder, and water. It had a maximum aggregate size of 1 inch, with 90% to 100% passing the ¾-inch sieve.

- **Foundation Course (Regular):**
  The Foundation Course (Regular) consisted of a single aggregate (or a combination of gravel and sand aggregates), a soil binder, and water. It had a maximum aggregate size of ¾ inch, with 50% to 74% passing the No. 10 sieve.

- **Foundation Course (Portland):**
  The Foundation Course (Portland) consisted of a single aggregate (or a combination of gravel and sand aggregates), a soil binder, 3% Portland cement, and water. Alternatively, the contractor had the option to use 4.5% Portland cement without a soil binder. The maximum aggregate size was ¾ inch, with 50% to 74% passing the No. 10 sieve.

- **Foundation Course (Granular):**
  The Foundation Course (Granular) was a coarse aggregate mix, often referred to as "dirty," with 80% to 90% retained on the No. 200 sieve.

- Most historic foundation courses were also labeled with an additional designator: Type I indicated a uniform depth, while Type II indicated a variable depth.

As noted above, these foundation course layers were not intended to function as drainage layers, but rather as stable foundations for supporting construction equipment. Pavement designers should treat these layers as non-drainable bases when evaluating and reviewing As-Built plans.

Current Nebraska Foundation Courses

Nebraska DOT utilizes four different types of Foundation Course:

- Crushed Concrete
- Aggregate D
- Bituminous Millings
- Crushed Rock with a Geosynthetic Separation Layer

Each type of Foundation Course has a specific gradation designed to provide a balance between stability and permeability. This ensures support for construction equipment, the pavement, and traffic loads while also allowing water to drain efficiently from the pavement structure. Depending on material availability, a Contractor may select different types of Foundation Courses and use them interchangeably throughout the project, provided the cross-slope material is not switched.

In the context of permeability, a k-value is a measure of a material's ability to allow water to flow through its pore spaces and voids. A higher k-value indicates greater permeability, meaning water can flow through the material more easily. Below is a table showing the k-values of the different Foundation Courses used in Nebraska.

Foundation Course Type	k-value	
Crushed Concrete	2.5×10^{-4} cm/sec	0.4 in/hr
Aggregate D	8.4×10^{-4} cm/sec	1.2 in/hr
Bituminous Millings	1.9×10^{-3} cm/sec	2.7 in/hr
Crushed Rock	9.9×10^{0} cm/sec	14,031.5 in/hr

Table 4.5-1 – Average k-values for Different Foundation Courses

The table above comparing the k-values of Nebraska's four Foundation Course types highlights significant differences in permeability. While all options provide adequate drainage, the Crushed Rock Foundation Course stands out with a significantly higher k-value, indicating a much faster rate of water flow. Due to this high permeability, a geosynthetic separation layer is required beneath the crushed rock to prevent the migration of fine particles from the subgrade. Without this layer, there is a risk of material loss or contamination between the subgrade and the Foundation Course, which can compromise the stability and long-term performance of the pavement structure.

When comparing the k-values of the Foundation Courses to those of different types of soil, it becomes evident that various Foundation Course types allow water to flow more easily than the soil in the subgrade.

Soil Type	**k-value**	
Clay	< 1.0 x 10^{-7} cm/sec	< 0.00014 in/hr
Silt	1.0 x 10^{-7} cm/sec to 1.0 x 10^{-5} cm/sec	0.00014 in/hr to 0.014 in/hr
Dirty Sand	1.0 x 10^{-5} cm/sec to 1.0 x 10^{-3} cm/sec	0.014 in/hr to 1.4 in/hr
Clean Sand	1.0 x 10^{-3} cm/sec to 1.0 x 10^{-1} cm/sec	1.4 in/hr to 141.7 in/hr
Course Gravel	> 1.0 x 10^{-1} cm/sec	> 141.7 in/hr

Table 4.5-2 – Average k-values for Different Soil Types

Understanding the role of the Foundation Course is essential for pavement design and construction. This layer not only supports equipment and traffic loads but also plays a key role in protecting the pavement by improving drainage and preventing moisture-related subgrade issues from reaching the surface. With multiple material options available – each offering different levels of permeability – engineers can select the most suitable Foundation Courses based on site conditions and project needs.

4.7 Drainage Layer Outlets

Once water infiltrates the drainage layer, it must be directed away from the pavement structure to reduce excess moisture and minimize pore pressure. If water becomes trapped within a poorly drained layer, it can create a “bathtub” effect, leading to premature pavement failures caused by frost heave and saturated subgrades. To prevent this, drainage systems are installed to provide an outlet for water to escape. The type of drain used often depends on whether the pavement is located in an urban or rural setting, as site conditions and space constraints can influence drainage design.

NDOT utilizes five common drainage layer drains:

- Granular Subdrains – Granular subdrains are drainage outlets constructed from granular materials that allow water to exit the drainage layer and flow into the foreslope or the bottom of a ditch. They are the most commonly used method for draining pavement structures in rural areas due to their simplicity and effectiveness. The spacing of granular subdrains depends on the slope of the pavement’s centerline profile: they are typically installed every 100 feet when the slope is less than 1%, and every 200 feet when the slope exceeds 1%. For active construction projects, these subdrains are typically installed after the earth shoulder construction and ditch embankment have been completed. On existing roadways showing signs of pavement distress due to saturated subgrade conditions, additional subdrains can be installed later – provided there is an existing foundation course beneath the pavement.

- Longitudinal Subdrains – Longitudinal subdrains are drainage outlet systems designed to remove water from beneath pavements where water accumulation can cause damage or instability. These subdrains are typically installed along the edge of the pavement and serve to connect multiple granular subdrains together, allowing for continuous and efficient water removal throughout the drainage network. Longitudinal subdrains have the same gradation as the granular subdrains.

- Pipe Underdrains (Urban) – Pipe underdrains are drainage outlets that combine granular material with a perforated pipe to direct water from the drainage layer and into a storm sewer system. They are the preferred method for discharging drainage layers in urban areas, where a direct outlet to a ditch or foreslope is typically not feasible. A pipe underdrain consists of a perforated pipe surrounded by granular material, which allows water to filter through and enter the pipe. To prevent clogging, the granular material is wrapped in filter fabric that blocks fine particles and debris. Pipe underdrains are usually installed along the outside edge of the pavement and connect the drainage layer to a city’s storm sewer system.

- Pipe Underdrains (Rural) – Pipe underdrains used in a rural setting are a drainage outlet system that combines features of both granular subdrains and pipe underdrains. This system consists of a perforated pipe surrounded by granular material and wrapped in filter fabric, similar to a typical urban pipe underdrain. However, instead of discharging into a storm sewer system, the water exits into the foreslope or bottom of a ditch – like a granular subdrain. Outlet spacing and placement follow the same guidelines used for granular subdrains. Pipe underdrains (rural) are typically used in rural areas where faster water removal from the drainage layer is needed to protect the integrity of the pavement structure and prevent premature asset deterioration.

- Cut-off Drains – Cut-off drains, also known as transverse drains, are drainage outlet systems that use granular material similar to that found in granular or longitudinal subdrains. These drains could also include a pipe similar to a pipe underdrain with an adjusted stiffness requirement. Cut-off drains are installed transversely beneath the pavement structure to intercept and redirect water that would otherwise flow along the centerline profile slope. These drains can be installed in sag areas – where water tends to accumulate – or in advance of structures to prevent water from collecting against them. Cut-off drains typically span the full width of the roadway and discharge into the foreslope or the bottom of a ditch.

Chapter 5: Gravel Road Design

5.1 Introduction

Gravel roads form a critical part of Nebraska's transportation infrastructure, providing essential connectivity for rural communities, agricultural operations, and local industries. Unlike paved roads, gravel roads consist of unbound layers of aggregate material and are typically used in areas with lower traffic volumes. Despite their seemingly simple structure, gravel roads require thoughtful design and regular maintenance to remain functional.

In Nebraska, where approximately 75% of roads are unpaved, maintaining the gravel road network is a persistent challenge due to variability in locally sourced materials and a continued reliance on empirical design practices. The extensive road network in the state spans approximately 194,938 miles, including 72,134 miles of gravel roads – 39 miles of which are classified as state highways. This significant mileage underscores the importance of establishing consistent design standards, improving maintenance strategies, and understanding the unique behaviors of unpaved surfaces under local climate and loading conditions.

Gravel roads not only serve as critical farm-to-market routes but also support emergency access, school transportation, and general mobility in rural areas. A well-designed gravel road system contributes to economic vitality and public safety, particularly in regions where paving is not economically feasible. As such, this chapter aims to provide a practical framework for the design and evaluation of gravel roads in Nebraska, aligning with NDOT's broader goal of supporting a resilient and efficient transportation system.

5.2 Common Distresses on Gravel Roads

The performance of unpaved roads is influenced by multiple factors, including material properties, drainage, climate conditions, traffic loads, and maintenance practices.

Gravel roads are susceptible to a range of surface and structural distresses that can compromise safety, ride quality, and service life. These distresses often result from the interaction of traffic loads, environmental factors, material properties, and maintenance practices. Understanding the common types of gravel road distresses are critical for pavement designers and maintenance professionals. This knowledge not only aids in accurate assessment and timely intervention but also informs design improvements that enhance durability and performance.

Below is a list of typical distresses commonly observed on gravel roads, followed by a detailed overview of each distress, including its causes, characteristics, and effects on road performance.

Washboarding

Washboarding, also known as corrugation, is the formation of a series of regular, transverse ripples or ridges – resembling a washboard – across the surface of a gravel road. These ridges are typically spaced a few inches apart and create a bumpy, uncomfortable ride. Common causes include vehicle acceleration or braking, high speeds, improper grading, dry or loose surface material, and insufficient compaction.

Erosion

Erosion on gravel roads is the process by which the road surface and its underlying materials are worn away and displaced, primarily due to water or wind. It may occur in both the transverse and longitudinal directions of the roadway. Common contributing factors include rainfall, heavy traffic, profile and/or cross slope, and inadequate drainage. Erosion can lead to significant surface damage, such as rutting, potholes, and the loss of aggregate material.

Rutting

Rutting on gravel roads refers to the formation of shallow, longitudinal depressions or grooves in the wheel paths caused by repeated traffic and environmental conditions. These ruts typically result from the compression and lateral displacement of gravel particles under vehicle loads, erosion from surface water flow, and moisture infiltration that weakens the road structure.

Potholes

Potholes on gravel roads are small depressions or holes that develop in the road surface. They are commonly caused by water infiltration beneath the surface, which can lead to material displacement during freeze-thaw cycles or under the repeated stress of vehicle traffic. Over time, the continued movement of gravel and lack of support beneath the surface contribute to the expansion of these holes.

Raveling

Raveling on gravel roads refers to the gradual disintegration of the road surface through the loss of individual gravel particles. This occurs when the binding material weakens or fails, causing particles to become dislodged. The result is a loose, uneven surface that can reduce ride quality and increase maintenance needs.

Dusting

Dusting on gravel roads refers to the process where fine particles of soil and gravel are kicked up into the air, creating a cloud of dust, especially during dry weather or with vehicle traffic. This phenomenon can be a nuisance, impacting visibility and contributing to the degradation of the road surface.

Loss of Crown

Loss of crown on gravel roads refers to the gradual flattening of the road surface, where the center becomes lower than the edges, impairing proper drainage. This condition typically results from the displacement of gravel and fine materials due to traffic, maintenance practices, and environmental factors. As a result, water tends to pond on the surface, softening the roadbed and potentially leading to structural issues. NDOT Minimum Design Standards for unpaved roadways include a cross slope between 2% and 4%.

Frost Damage

Frost damage on gravel roads typically occurs during the winter and spring months as a result of freeze-thaw cycles. When water becomes trapped in the gravel and underlying soil, it expands upon freezing, causing the road surface to heave and crack. As the ice melts in the spring, the surface becomes soft and unstable, which can lead to common forms of deterioration such as potholes, washboarding, and rutting.

5.3 Material Selection and Gradation Requirements

Material selection and particle-size distribution are critical factors in determining the strength and performance of gravel road materials. Strength is primarily achieved through particle interlocking and stiffness, both of which depend on well-graded materials. Gradation also affects permeability, particularly the proportion of particles smaller than 0.5 mm, which can influence drainage and stability.

Beyond gradation, the plasticity of fine materials plays an important role. When wet, high-plasticity fines can reduce interlocking and weaken the structure. However, in dry conditions, these fines can provide cohesive strength, helping to bind the aggregate together. Maintaining a cross slope of 3% to 4% is ideal for reducing the effects of stagnant water saturating the gravel road.

Hardness and durability are also essential. Aggregates must be hard enough to resist deformation under both compaction and repeated traffic loads. Durable materials ensure long-term performance by resisting breakdown under environmental exposure.

Recommended Properties for Base Aggregate and Surfacing Aggregate

For use as a base and surfacing material, crushed aggregate must offer structural support while also resisting surface issues like raveling and washboarding. To achieve this, the aggregate should be well graded and contain an appropriate amount of plastic fines acting as a binder.

The Federal Highway Administration has a well-informed gravel road construction and maintenance guide. Please see the link below.

https://www.fhwa.dot.gov/construction/pubs/ots15002.pdf

Granular Stabilization

Granular stabilization is a technique that involves blending additional granular materials – such as natural gravel, crusher run or selected fine-grained soils – with native (virgin) aggregate to improve its properties. Materials sourced from gravel pits are often open-graded or poorly graded, which limits their performance under traffic loads. Granular stabilization addresses these challenges by correcting deficiencies in the base material through the incorporation of suitable granular blends.

To minimize hauling and transportation costs associated with road construction, materials sourced locally within the same county are blended for a granular stabilization. The blending process involves systematically combining two to three materials in varying proportions to produce a wide range of potential gradation curves. Each blended gradation starts with a baseline mix ratio of coarse aggregate, fine aggregate, and binder material.

Addressing gravel road distresses begins with evaluating the existing material gradation in comparison to the optimized gradation band – a specific particle size distribution of aggregates that yields optimal performance in terms of stability, durability, and drainage. This band is typically represented by an area between two curves: an upper limit and a lower limit. Improvement potential is assessed by analyzing current material properties and adjusting the mix using different aggregate sources, specifically by determining the proportions of coarse aggregate, fine aggregate, and binder materials. These materials are then incrementally adjusted to explore a range of gradation curves. In some trials, the percentage of coarse aggregate is reduced to assess its influence on the finer portion of the curve; in others, the fines content is varied to evaluate its effect on overall gradation. Blends that show potential are further refined through targeted adjustments to component ratios, allowing the gradation curve to approach the desired range without exceeding specified limits. Promising blends are replicated to confirm consistency in achieving the target gradation.

Based on an extensive literature review, it is recommended that a blended material should meet the following gradation criteria:

- Gravel Content – Minimum of 25%
- Sand Content – Not to exceed 60%
- Fines Content – Maintained between 5% and 20%

On the next page, Figure 5.3-1 shows an ideal gradation band for surface course and base course aggregates used for gravel roads.

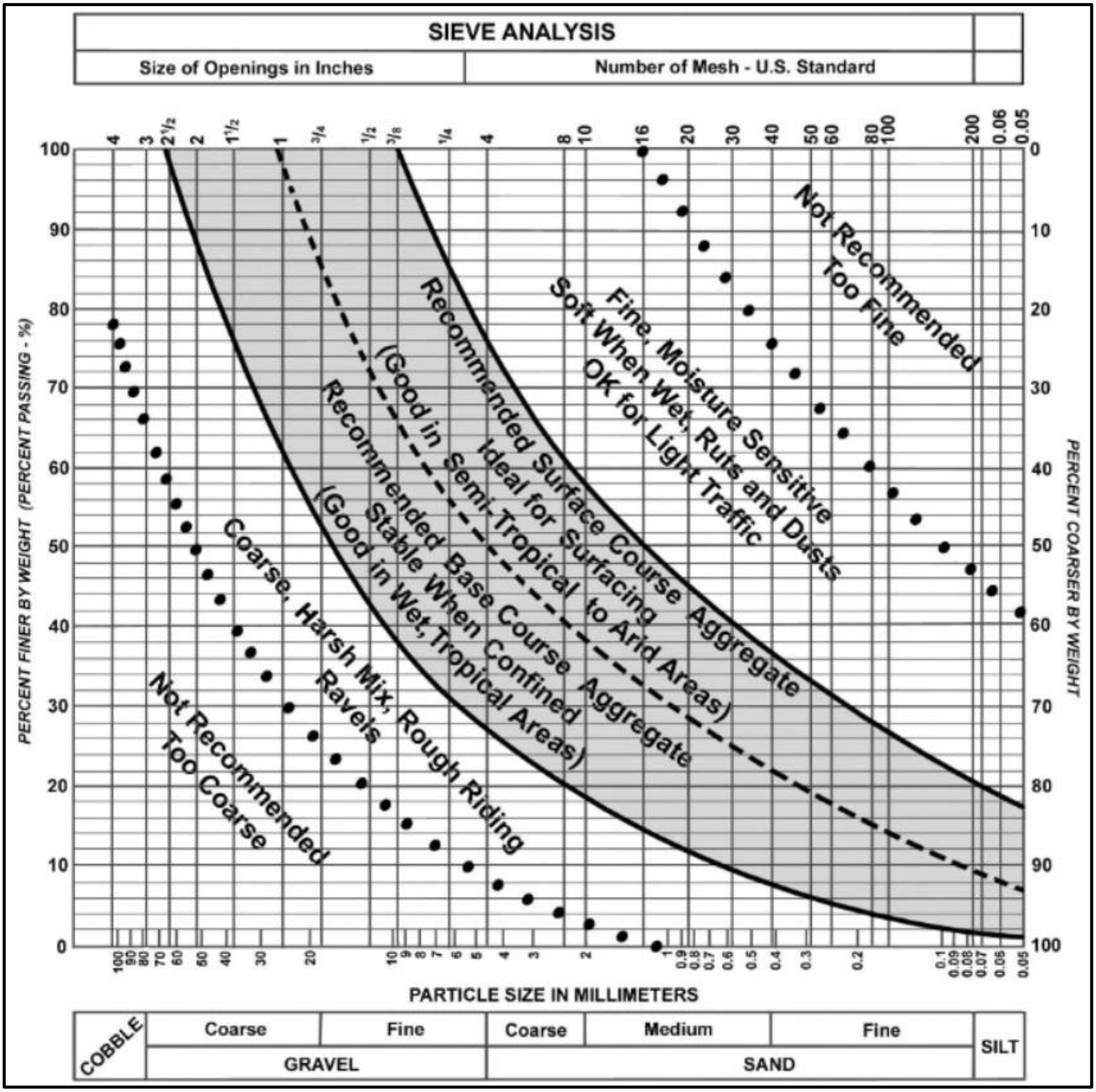


Figure 5.3-1 – Ideal Grain Size Distribution for Surface-Course and Base-Course Aggregate

However, the optimized gradation band used in Nebraska differs from the ideal grain size distribution described above. Nebraska’s gradation band is derived from a statistical analysis of materials that demonstrated the highest Maximum Resilient Moduli and the least Permanent Deformation under testing. For additional details, refer to the research study *Gravel Road Performance Enhancements (SPR FY23(014))*.

5.4 Mechanical Properties of Gravel Road Materials

Resilient Modulus

The resilient modulus (M_R) is defined as the ratio of applied deviatoric stress to the recoverable (elastic) strain experienced by a material under repeated loading conditions, such as traffic. It represents the elastic response of unbound pavement materials and serves as a fundamental measure of stiffness for composite gravel roadways, subgrade, and base layers. Because M_R reflects material behavior under realistic loading conditions, it is a key input parameter in mechanistic pavement analysis used to predict pavement distresses such as rutting and surface roughness.

Laboratory M_R testing is typically performed using repeated cyclic loading triaxial equipment to simulate traffic wheel loads acting on compacted soil specimens. The applied stress levels depend on the specimen's position within the pavement structure. Confining pressure represents the overburden stress corresponding to the specimen's depth, while the axial deviatoric stress consists of a cyclic component simulating traffic loads and a static seating load to maintain specimen contact during testing. M_R is determined as the ratio of cyclic stress ($\Delta\sigma$) to the corresponding recoverable elastic strain and is stress-dependent, varying with both confining pressure and applied deviatoric stress.

The resilient modulus characterizes the elastic response of soils subjected to cyclic loading and indicates a material's ability to resist deformation. M_R is calculated as the ratio of deviatoric stress to recoverable strain and is stress dependent. A given material's resilient modulus may vary with changes in confining pressure and applied deviatoric (vertical) stress.

$$\theta = \sigma_1 + \sigma_2 + \sigma_3$$
or
$$\theta = \sigma_d + 3\sigma_3$$

- Bulk Stress (θ) – A uniform pressure exerted on a material when forces act inward from all directions, perpendicular to its surface. Bulk stress causes a change in volume without a change in shape.

- Deviatoric Stress (σ_d) – A portion of a material's total stress responsible for changes in shape (distortion or shearing) rather than volume. Deviatoric stress represents unequal stresses (variable loadings) that lead to plastic deformation and yielding.

  $$\sigma_d = \sigma_1 - \sigma_3$$

  Where: σ_1 = Major Principal Stress
  σ_3 = Confining Pressure

- Confining Stress ($\sigma_{2\&3}$) – The confining pressure surrounding a material sample in a triaxial testing apparatus.

The figure below (Figure 5.4-1) illustrates the variation of M_R values for a sample under different stress levels conditions.

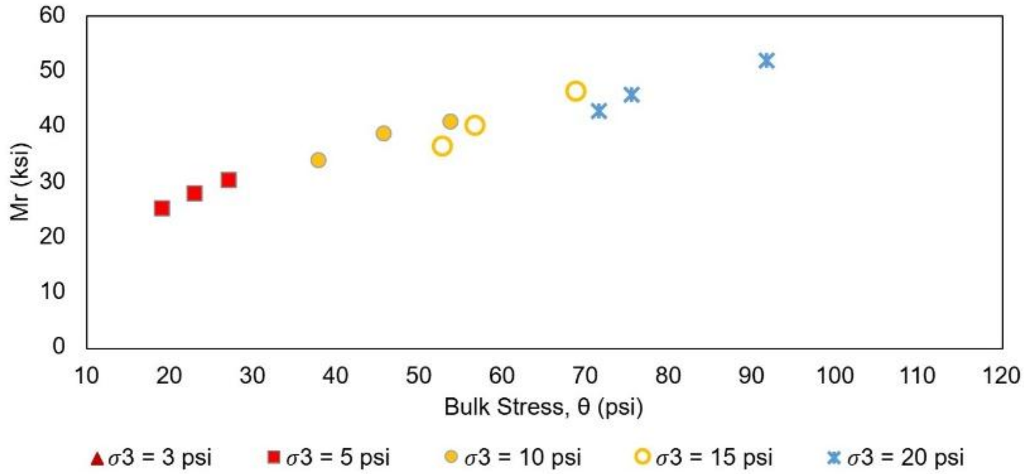


Figure 5.4-1 – Resilient Modulus Plot of a Nebraska Material

In addition to laboratory testing, M_R characterization may be obtained using Falling Weight Deflectometer (FWD) measurements, which evaluate in-situ pavement response under impulse loading that closely approximates traffic conditions. Both laboratory and FWD-based approaches provide reliable indicators of unbound material stiffness within pavement systems.

Traditional strength parameters, such as cohesion and friction angle, alone do not adequately represent the structural capacity of unbound pavement layers under repeated traffic loading. Pavement systems must resist cumulative deformation and fatigue under repeated traffic loadings rather than single-load failure. As a direct measure of stiffness under cyclic loading, M_R plays a critical role in modern pavement design and is incorporated into structural response models to predict long-term pavement performance.

Understanding the stress–strain behavior of pavement foundation layers is essential for optimizing pavement performance, as these layers strongly influence gravel road distresses. Research has shown that both stiffness and strength of subgrade and base materials significantly affect pavement durability, particularly under varying moisture conditions, inadequate drainage, and freeze–thaw cycles.

Summary Resilient Modulus

The summary resilient modulus (SM_R) represents the overall stiffness of a gravel road by combining the resilient modulus values of both the surface and subgrade layers. SM_R is expressed as a single representative resilient modulus (M_R) value for a given material or soil at a specified stress condition. For example, using the resilient modulus data from Figure 5.4-1, a fitted curve can be developed using a universal model (NCHRP 1-37A). The M_R value corresponding to a selected bulk stress from this curve is defined as the SM_R for that material.

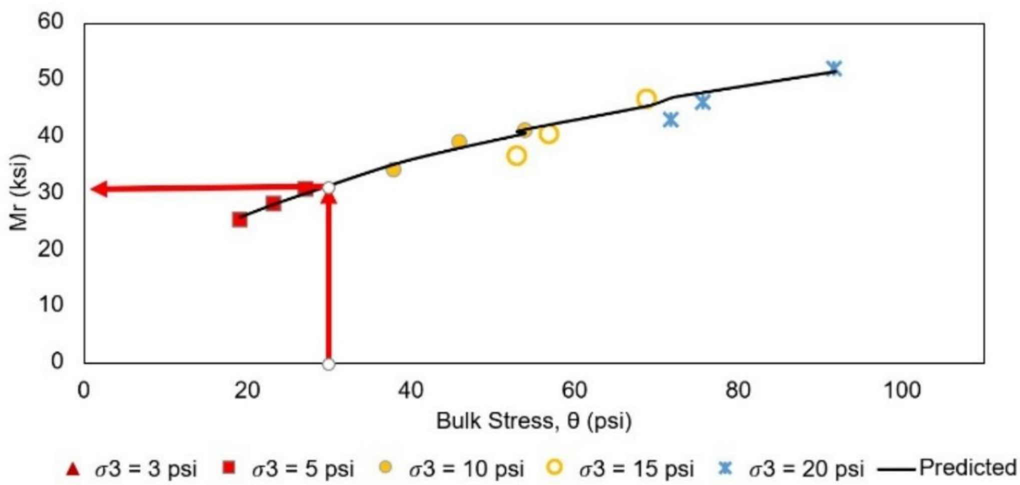


Figure 5.4-2 – Summary Resilient Modulus Plot of a Nebraska Material

For the example shown in Figure 5.4-2, selecting a bulk stress of 30-psi to the material results in a SM_R of 31-ksi. The selection of a 30-psi bulk stress is based on guidance from NCHRP 1-28A, one of the most used references for M_R and SM_R analysis. According to NCHRP 1-28A, a bulk stress of 30-psi is representative of stress conditions typically experienced by base layers in the field.

To calculate the SM_R, standardized stress conditions are used based on recommendations from the National Cooperative Highway Research Program (NCHRP) 1-28A report.

National Cooperative Highway Research Program (NCHRP) 1-28A Report

NCHRP01-28A_FR-Vol1.pdf

Figure 5.4-3 – Resilient Modulus System (VJ Tech Limited)

The SM_R results for the surface and subgrade materials across various counties in Nebraska are presented in Table 5-4.1.

Table 5.4-1 – SM_R and Model Parameters for Virgin Materials

County	Materials Name (Virgin)	SM_R (ksi)	k_1	k_2	k_3	R^2
Douglas	Clean (1.5”)	19.5	1016.5	0.39	-0.05	0.75
	Clean (1”)	29.2	1620.0	0.31	-0.05	0.74
	Road Gravel (0.75”)	12.6	610.5	0.48	-0.02	0.92
	Crusher Run (1.5”)	15.0	740.7	0.46	-0.02	0.88
	Crusher Run (3/4”)	17.3	843.6	0.47	-0.01	0.87
	Subgrade	5.0	486.3	-0.08	-2.02	0.85
Harlan	Crusher Run (1” Minus)	17.5	763.6	0.63	-0.01	0.95
	Green Surface Sand	10.6	483.4	0.58	-0.03	0.95
	Surface Sand	NA	NA	NA	NA	NA
	Gravel	16.0	824.4	0.41	-0.05	0.77
	Subgrade (Silty Loam)	5.8	937.8	0.28	-4.62	0.44
	Subgrade (Clay)	12.5	852.2	0	0	0.55
Scottsbluff	Rock (3/8”-1”)	13.4	612.3	0.56	-0.01	0.89
	Road Gravel (3/4”-1”)	16.3	844.9	0.41	-0.06	0.91
	Subgrade	10.9	834	-0.26	-0.93	0.29
Cherry	Section 27	18.2	926.3	0.41	-0.01	0.91
	Section 7	14.0	672.7	0.49	-0.02	0.81

Where: k_1, k_2, k_3 = Regression Coefficients
SM_R = Summary Resilient Modulus
R^2 = Coefficient of Determination

- Regression Coefficients (k_1, k_2, k_3) – A material coefficient that characterize the non-linear behavior of geomaterials under cyclic loading.

- Coefficient of Determination (R^2) – Represents the standard error of the regression model. The higher the number, the lower the variance.

Breakage

Breakage refers to the process by which aggregates fracture into smaller pieces, typically due to external forces such as loading, impact, or weathering. The mechanical degradation or abrasion of granular materials used in unpaved road surfaces and pavement base layers can significantly affect their mechanical properties, drainage characteristics, and long-term durability.

To assess the abrasion resistance of a granular material during compaction, a gyratory compactor may be used to perform a gyratory abrasion test. This test simulates the wear and degradation aggregates undergo under dynamic traffic loads. It provides a reliable measure of the aggregate's ability to withstand mechanical breakdown.

Using Hardin's concept to evaluate aggregate degradation provides a representation of a granular material's ability to break down under applied stresses.

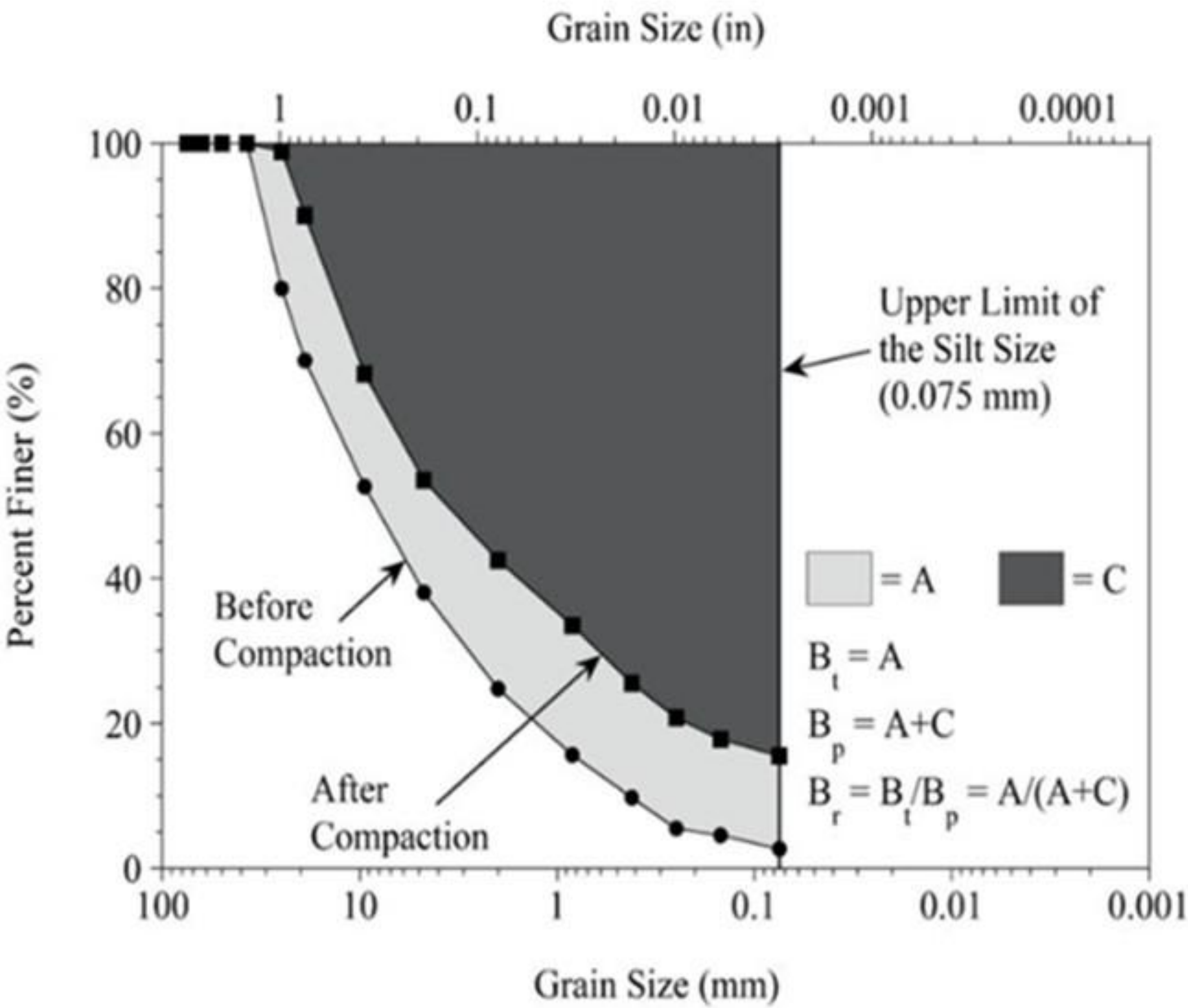


Figure 5.4-4 – Hardin's Concept to Evaluate the Degradation of Aggregates (Hardin, 1985)

- Breaking Potential (B_p) – Breaking Potential (B_p) refers to the tendency of particles within a material to fracture or crush under applied stress. It is quantified as the area between the initial gradation curve and a vertical line representing the upper limit of silt-sized particles (0.075 mm) on a particle size distribution graph.

- Total Breakage (B_t) – Total Breakage (B_t) represents the overall amount of particle fragmentation within a granular sample. It is calculated as the area between the initial and final gradation curves on a particle size distribution graph.

- Relative Breaking (B_r) – Relative Breakage (B_r) quantifies the extent of particle crushing or fragmentation within a granular sample relative to its maximum potential breakage. It is defined as the ratio of Total Breakage (B_t) to Breaking Potential (B_p).

Figure 5.4-5(a) represents a granular sample before testing, while **Figure 5.4-5(b)** illustrates the condition of the sample following the gyratory abrasion test.

Figure 5.4-5(a)

Figure 5.4-5(b)

Gyratory compaction tests were performed on a range of granular materials collected from various locations across Nebraska. The results revealed significant variation in particle breakage depending on the location and the type of material tested. The gyratory abrasion results for these granular materials are summarized in Table 5-4.2.

Table 5.4-2 – Gyratory Abrasion Results

County	Material Name	Total Breakage (%)	Relative Breakage (%)
Douglas	Clean (1.5”)	17.3	6.7
	Clean (1”)	14.0	6.0
	Road Gravel (0.75”)	3.9	2.3
	Crusher Run (1.5”)	4.3	2.3
	Crusher Run (3/4”)	6.7	3.7
	Subgrade	NA	NA
Harlan	Crusher Run (1” Minus)	11.3	5.7
	Green Surface Sand	2.9	2.2
	Surface Sand	2.8	2.0
	Gravel	3.0	1.7
	Subgrade (Silty Loam)	NA	NA
	Subgrade (Clay)	NA	NA
Scottsbluff	Rock (3/8”-1”)	7.9	4.0
	Road Gravel (3/4”-1”)	3.3	1.9
	Subgrade	NA	NA
Cherry	Section 27	16.3	7.2
	Section 7	4.2	2.5

Where: NA = Not Applicable

Overall, materials exhibiting high breakage values – such as clean materials and Section 27 – may require stabilization to improve durability. In contrast, well-graded materials like Crusher Run, Surface Sand, and Road Gravel demonstrate greater resistance to breakage, making them more suitable for applications where durability and structural integrity are critical.

5.5 Shakedown Behavior and Performance Thresholds

In gravel road design, shakedown behavior refers to the response of granular materials to repeated cyclic loading, such as traffic loads. It describes how these materials deform over time when subjected to recurring stresses. The shakedown concept classifies material behavior into three main categories, depending on the stress levels and the material's capacity to resist the accumulation of permanent deformation (plastic strain):

- **Range A – Plastic Shakedown**
  At low stress levels, the material initially undergoes limited plastic deformation. However, with continued loading, the rate of permanent deformation stabilizes, and the material responds elastically in subsequent cycles. This is considered stable and desirable behavior for gravel road design.

- **Range B – Plastic Creep**
  Under moderate stress levels, the material continues to accumulate permanent deformation, but at a decreasing rate. Eventually, the strain rate approaches a constant value. This response is generally acceptable, but it is recommended that the virgin material be blended.

- **Range C – Incremental Collapse**
  At high stress levels, the material exhibits ongoing and significant accumulation of permanent deformation with each loading cycle. This progressive failure leads to severe rutting or structural collapse and is deemed unacceptable in gravel road performance.

The shakedown limit represents the critical stress level that distinguishes stable material behavior (plastic shakedown and plastic creep) from unstable behavior (incremental collapse). Figure 5.5-1 illustrates the three behavioral ranges defined by shakedown theory.

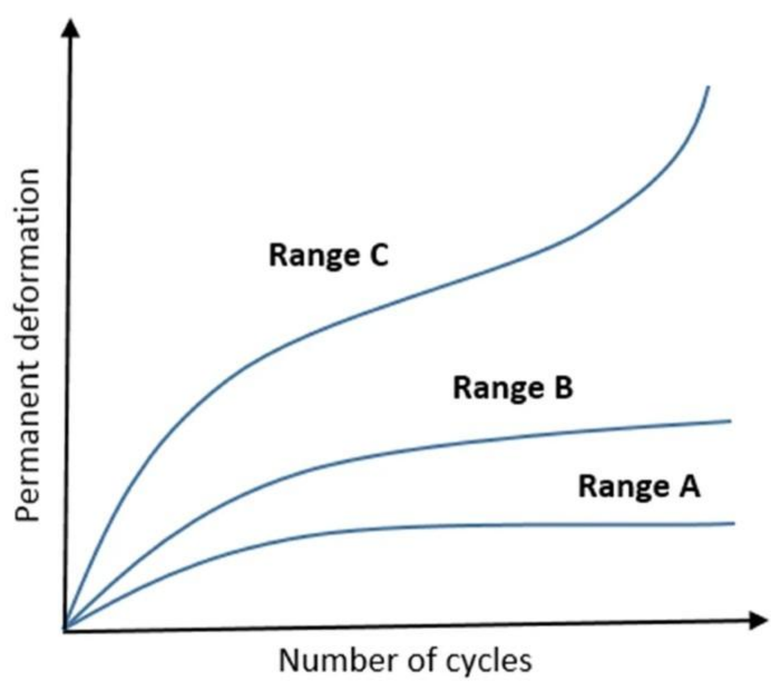


Figure 5.5-1 – Limits of Shakedown Behavior Theory

When repeated traffic loads are relatively low, the accumulation of permanent deformation stabilizes after a certain number of load cycles. Beyond this point, the material responds primarily elastically – this is referred to as Range A behavior.

Under moderate repeated traffic loads that exceed the elastic limit, permanent deformation continues to accumulate but at a decreasing rate. Eventually, the strain rate levels off, and the material reaches a stable condition – characteristic of Range B. Virgin materials that fall into Range B should be blended to improve resistance to permanent deformation.

However, when traffic loads are high, the accumulation of permanent deformation increases rapidly with each loading cycle. This can lead to failure through shear deformation or overstressing of the unbound granular materials – representing Range C, or incremental collapse. Range C materials will need to be blended to become structurally stable.

Using the Werkmeister criterion, numerical thresholds can be assigned to define the three ranges of shakedown behavior. These values help assess the performance of unbound granular materials under repeated loading. Table 5.5-1 presents the corresponding Werkmeister criterion values associated with each behavioral range.

Table 5.5-1 – Shakedown Theory Ranges using Werkmeister Criterion

Shakedown Ranges	Werkmeister Criterion
Range A	$\varepsilon_{P,5000} - \varepsilon_{P,3000} < 4.5 \times 10^{-5}$
Range B	$4.5 \times 10^{-5} < \varepsilon_{P,5000} - \varepsilon_{P,3000} < 4.5 \times 10^{-4}$
Range C	$\varepsilon_{P,5000} - \varepsilon_{P,3000} > 4.5 \times 10^{-4}$
Where: $\varepsilon_{P,3000}$ = Permanent Axial Strain at the 3,000th load cycle $\varepsilon_{P,5000}$ = Permanent Axial Strain at the 5,000th load cycle	

A general understanding of shakedown behavior helps design durable gravel roads, as it enables engineers to predict the long-term performance of unbound granular materials under repeated traffic loading. By accounting for the shakedown limits of these materials, road structures can be designed to minimize excessive permanent deformation, thereby enhancing the road's longevity and serviceability. Achieving this often requires careful selection of materials, appropriate compaction, and optimized layer thicknesses.

5.6 Material Performance Prediction

Angularity

The angularity of the aggregate should also be visually assessed during the sieve analysis. Cubic or angular materials (Figure 5.6-1(a)) provide better interlock compared to rounded materials, such as uncrushed alluvial aggregates (Figure 5.6-1(b)). To enhance interlock and reduce the risk of raveling, rounded aggregates should be crushed to ensure they have at least two fractured faces.

Figure 5.6-1(a)

Figure 5.6-1(b)

Maximum Aggregate Size

Although the grading coefficient is calculated using material passing the 1 in. sieve – and many specifications list this as the maximum aggregate size – larger particles ranging from 1½ in. to 1¾ in. are generally acceptable for ensuring adequate all-weather passability. However, using aggregates larger than this can negatively impact ride quality, increase road noise, and create difficulties for the maintenance grader operator.

As a general guideline, the maximum aggregate size should not exceed one-third of the thickness of the compacted layer.

Grading Analysis

Another useful performance prediction tool is understanding the relationship between grading analysis and shrinkage.

This approach utilizes five key sieve sizes from a standard laboratory grading analysis that are used to evaluate material performance and guide the selection of appropriate chemical treatments. These key sieve sizes are 1.0 in., #4, #8, #40, and #200. The first three sieve sizes assess the proportion of coarse particles. The other two sieves assess the intermediate and fine particle potential for shrinkage. The coarse fraction can be quantified using the following simple formula known as the grading coefficient (Gc):

$$Gc = ((P1.0\text{ in.} - P\#8) \times P\#4) / 100$$

Where P is percent passing

The percentage of material passing the #200 sieve is an important indicator of unpaved road performance and influences the choice of chemical treatment.

- High fines content (i.e., more than 20%) typically results in dusty roads when dry and slippery conditions when wet.
- Conversely, low fines content (i.e., less than 10%) often leads to washboarding, requiring frequent grader maintenance.

Many unpaved road wearing course specifications, which are often based on paved road base course standards, limit fines content to about 5% to 8%, mistakenly assuming this reduces dust. However, determining the percentage passing the #200 sieve – usually by a wet process as part of a standard grading analysis – is more complex than measuring material passing the #8 sieve (2.36 mm), which can be quickly assessed using a dry sieve analysis as a field indicator.

To provide a practical understanding of material performance, this approach incorporates the #200 sieve material into the grading coefficient equation as part of the material passing the #8 sieve. Nevertheless, the actual percent passing the #200 sieve remains essential for selecting appropriate chemical treatments.

The percentage of material passing the #40 sieve, when combined with a plasticity test, helps assess the influence of clay content in the material. This relationship is discussed in the following section.

Shrinkage

The plasticity index, determined from Atterberg limit tests, is used in conjunction with the percentage of material passing the #40 sieve. This combination helps evaluate the influence of clay content on material performance. The relationship is expressed using the following simple formula, known as the shrinkage product (S_p):

$$S_p = (PI \times 0.5) \times P\#40$$

When plasticity index values are available, the following recommendations should be applied to improve performance assessments:

- If PI $\geq$ 1: Use the actual PI value without modification.

- If the material is Non-Plastic (PI = 0):
  - If % passing the #200 sieve < 20%, set PI = 0 in the equation.
  - If % passing the #200 sieve $\geq$ 20%, set PI = 1 in the equation.

- If the material is classified as "slightly plastic":
  - If % passing the #200 sieve < 20%, set PI = 1 in the equation.
  - If % passing the #200 sieve $\geq$ 20%, set PI = 2 in the equation.

Material Performance Predictor Chart

A simple chart plotting grading coefficient on the x-axis and shrinkage product on the y-axis – along with the optimal performance limits described previously – can be used to evaluate the expected in-service performance of a material. This tool allows for quick, visual identification of materials likely to perform well or poorly in specific road conditions.

Local calibration of grading coefficient and shrinkage product ranges may be necessary to reflect regional conditions. For example:

- The upper limit of the shrinkage product range (e.g., reduced to 250) may need to be lowered for:
  - Roads with high truck traffic volumes
  - Roads that are shaded most of the day
  - Roads in regions with high annual rainfall or frequent high-intensity storms

- The lower limit (e.g., reduced to 50 or 75) may be appropriate for:
  - Roads with very low traffic volumes
  - Roads used primarily by slow-moving vehicles
  - Roads that are shaded or located in wet climates

To perform local calibration, practitioners should sample materials from both well-performing and poorly performing roads within their jurisdiction. These samples should be tested and the results plotted on the grading coefficient vs. shrinkage product chart **(Figure 5.6-2)**. Based on this analysis, the acceptable range thresholds can be adjusted to reflect observed local performance.

These calibrated ranges can then guide future material selection and acquisition, ensuring more reliable road performance under local conditions.

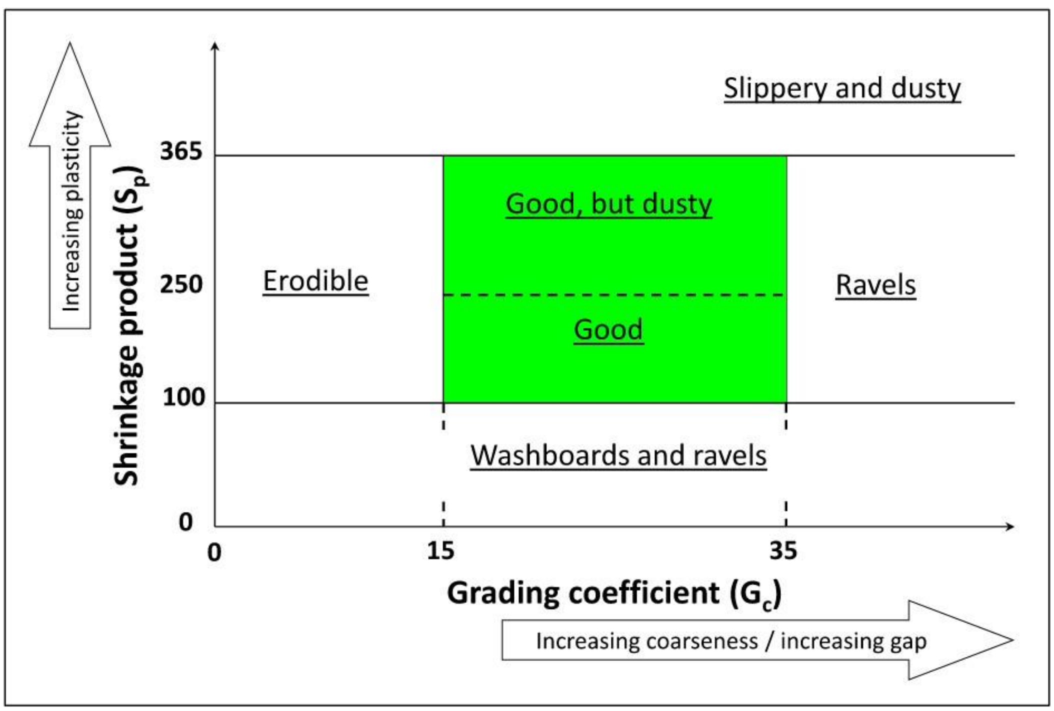
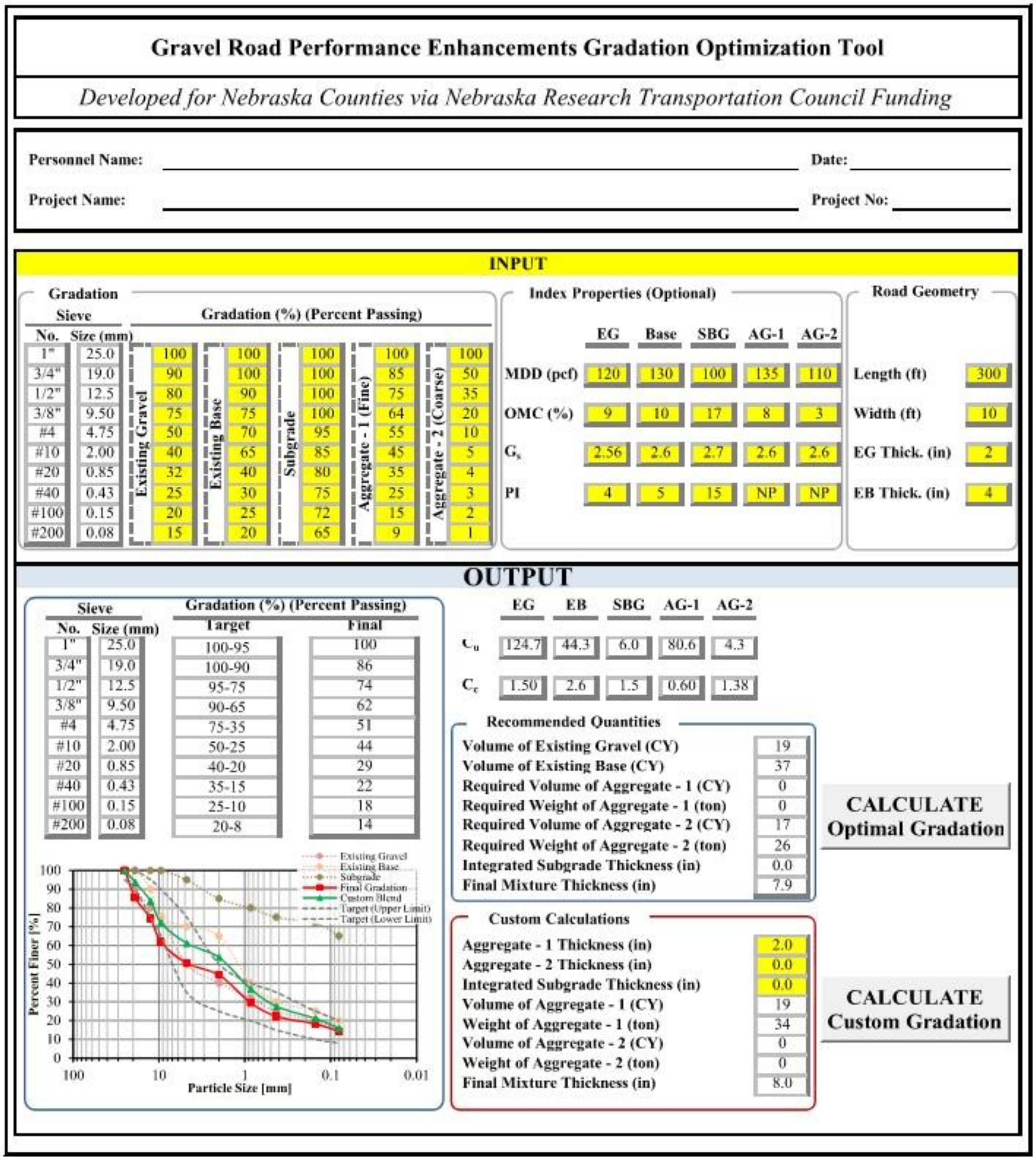


Figure 5.6-2 – Material Performance Predictor Chart (adapted from Paige-Green)

NDOT Gravel Gradation Optimization Tool

In 2025, the Nebraska DOT sponsored a research project through Michigan State University called *Gravel Road Performance Enhancements (SPR FY23(014))*. The goal of this research project was to improve the gravel road performance by systematically evaluating Nebraska's local materials and develop an optimization tool for better gradation control.



Gravel Road Performance Enhancements Gradation Optimization Tool

Developed for Nebraska Counties via Nebraska Research Transportation Council Funding

Personnel Name: ____________ Date: ________

Project Name: ____________ Project No: ________

INPUT

Gradation

Sieve No.	Size (mm)	Existing Gravel	Existing Base	Subgrade	Aggregate - 1 (Fine)	Aggregate - 2 (Coarse)
1"	25.0	100	100	100	100	100
3/4"	19.0	90	100	100	85	50
1/2"	12.5	80	90	100	75	35
3/8"	9.50	75	75	100	64	20
#4	4.75	50	70	95	55	10
#10	2.00	40	65	85	45	5
#20	0.85	32	40	80	35	4
#40	0.43	25	30	75	25	3
#100	0.15	20	25	72	15	2
#200	0.08	15	20	65	9	1

Index Properties (Optional)

	EG	Base	SBG	AG-1	AG-2
MDD (pcf)	120	130	100	135	110
OMC (%)	9	10	17	8	3
G_s	2.56	2.6	2.7	2.6	2.6
PI	4	5	15	NP	NP

Road Geometry

Length (ft)	300
Width (ft)	10
EG Thick. (in)	2
EB Thick. (in)	4

OUTPUT

Sieve No.	Size (mm)	Gradation (%) (Percent Passing) Target	Final
1"	25.0	100-95	100
3/4"	19.0	100-90	86
1/2"	12.5	95-75	74
3/8"	9.50	90-65	62
#4	4.75	75-35	51
#10	2.00	50-25	44
#20	0.85	40-20	29
#40	0.43	35-15	22
#100	0.15	25-10	18
#200	0.08	20-8	14

	EG	EB	SBG	AG-1	AG-2
C_u	124.7	44.3	6.0	80.6	4.3
C_c	1.50	2.6	1.5	0.60	1.38

Recommended Quantities

Volume of Existing Gravel (CY)	19
Volume of Existing Base (CY)	37
Required Volume of Aggregate - 1 (CY)	0
Required Weight of Aggregate - 1 (ton)	0
Required Volume of Aggregate - 2 (CY)	17
Required Weight of Aggregate - 2 (ton)	26
Integrated Subgrade Thickness (in)	0.0
Final Mixture Thickness (in)	7.9

CALCULATE Optimal Gradation

Custom Calculations

Aggregate - 1 Thickness (in)	2.0
Aggregate - 2 Thickness (in)	0.0
Integrated Subgrade Thickness (in)	0.0
Volume of Aggregate - 1 (CY)	19
Weight of Aggregate - 1 (ton)	34
Volume of Aggregate - 2 (CY)	0
Weight of Aggregate - 2 (ton)	0
Final Mixture Thickness (in)	8.0

CALCULATE Custom Gradation

Figure 5.6-3 – Gravel Road Enhancement Gradation Optimization Tool (Nebraska DOT)

Example of using the Gravel Road Enhancement Gradation Optimization Tool

A county in central Nebraska is planning improvements to a gravel road by blending in stockpiled aggregate over a section approximately 1,000 feet in length. The road has an average width of 20 feet and an average depth of 6 inches – consisting of a 2-inch wearing surface course over a 4-inch base course layer placed on top of the subgrade soil.

There are two available stockpile sources to choose from for this maintenance project.

Below are the material properties for each source:

Material Properties		Existing Gravel Road Materials			Available Materials	
Sieve Size		Wearing Course	Base Course	Subgrade	Stockpile 1	Stockpile 2
No.	Size (mm)					
1”	25.0	99	100	100	99	100
¾”	19.0	90	89	100	87	96
½”	12.5	67	68	100	71	80
3/8”	9.50	57	62	100	68	65
#4	4.75	38	48	97	53	37
#10	2.00	31	22	90	21	19
#20	0.85	25	15	82	14	11
#40	0.43	19	14	75	10	8
#100	0.15	14	13	66	8	5
#200	0.08	11	13	61	7	4
MDD		121	116	112	134	120
OMC		11.3	6.4	15.9	7.3	7.3
Plasticity Index		4	5	20	NP	NP

Where: MDD = Maximum Dry Unit (lb/ft^3)
OMC = Optimum Moisture Content (%)
Gradation Percentages are expressed as Percent Passing (%)

The figure below presents the input values entered into the Gravel Road Enhancement Gradation Optimization Tool, based on the material properties obtained from the gravel road and each stockpile source.

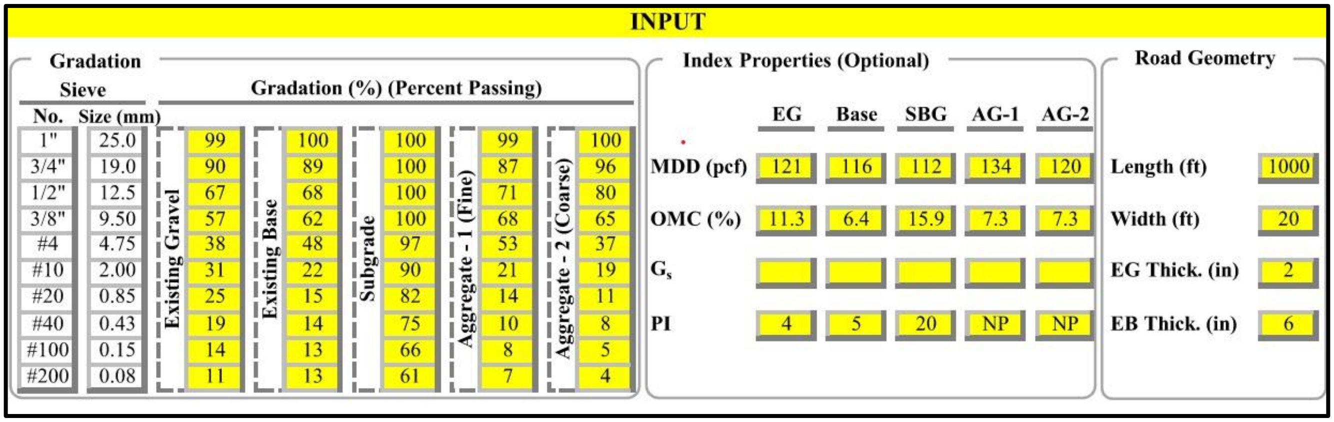


Figure 5.6-4 – Input Values for Gravel Road Enhancement Gradation Optimization Tool

By inputting the data into the Gravel Road Enhancement Gradation Optimization Tool, it was determined that achieving an optimal gradation will require approximately 600 tons of Stockpile 2. Stockpile 1 is not needed to meet the target gradation specifications.

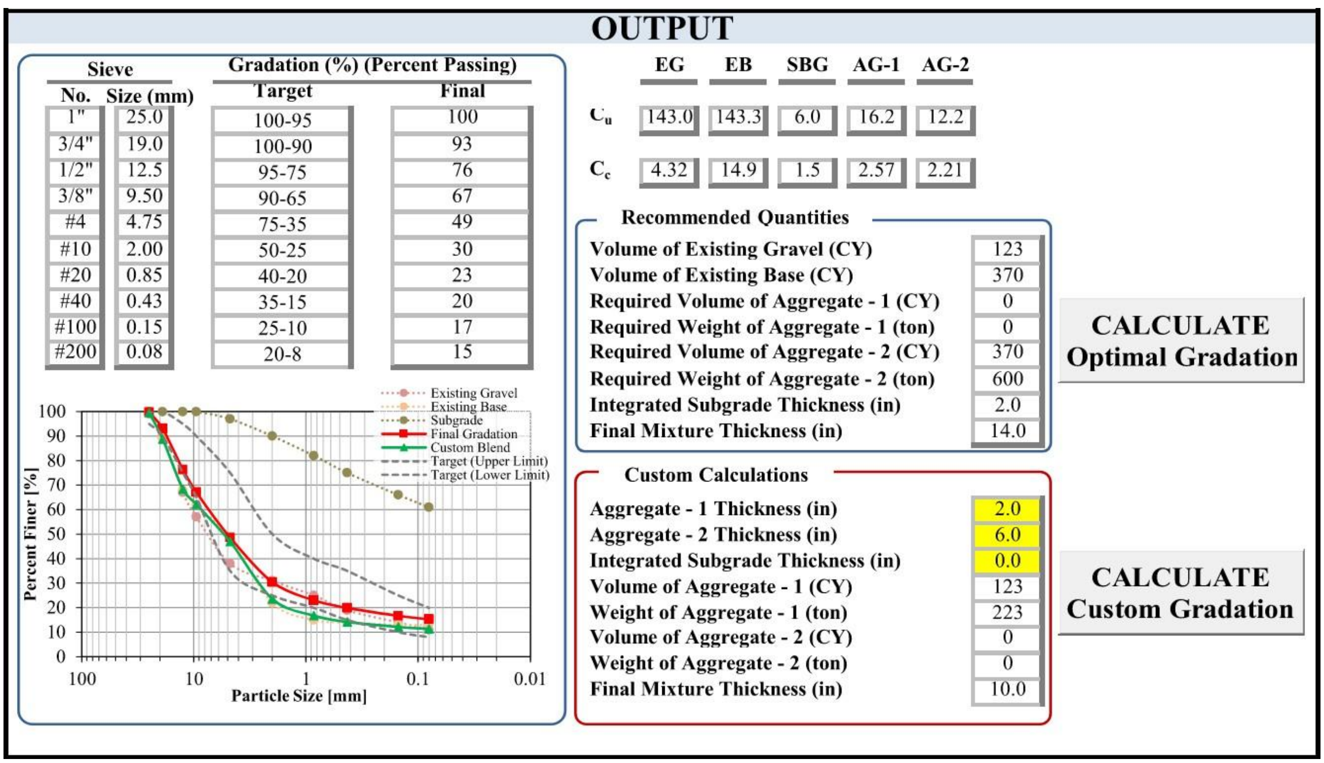


Figure 5.6-5 – Output Values for Gravel Road Enhancement Gradation Optimization Tool

6.1 Introduction

Flexible pavement is a type of road surface constructed of asphalt or bitumen that is designed to resist cracking and rutting under traffic loads. It typically consists of multiple layers of bituminous materials placed over a prepared subgrade. The structural capacity of flexible pavement depends on the properties of these materials, including their strength, stiffness, and ability to distribute loads. In Nebraska, asphalt is the primary material used in flexible pavement systems. Other bituminous treatments – such as Armor Coats, Chips Seals, and Microsurfacings – are also used, but mainly for preventive maintenance rather than as structural layers.

The flexible nature of bituminous materials makes asphalt pavements easier to resurface compared to rigid pavements. While flexible pavements are generally less expensive to construct, but have a shorter service life and may require more frequent maintenance over time.

6.2 Historical Nebraska Asphalt Mix Designs

Nebraska has been using flexible pavements in its road system for many years. Over time, NDOT has developed, evaluated, modified, and, in some cases, phased out various asphalt mix designs based on their field performance. Despite these changes, many older asphalt mixes remain in place throughout the state's pavement network. Some of these past asphalt layers continue to perform well and can serve as a strong foundation for new asphalt layers. Others, however, may contribute to pavement distress and may require additional structural thickness or full removal and replacement. It is important for designers to be familiar with past asphalt mix designs when evaluating pavement conditions, as understanding these materials is key to diagnosing issues and selecting appropriate rehabilitation strategies. The following list outlines historical NDOT asphalt mix designs and their typical applications.

- **Type 1** – This mix consisted of a combined mineral aggregate containing at least 50% crushed rock. Crushed rock mineral aggregates included no more than 15% naturally occurring fines retained on the No. 10 sieve. A maximum of 60% limestone was permitted. Type 1 was used for the same types of projects as Type 11.

- **Type 1R** – This mix was identical to Type 1, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. It was used for the same types of projects as Type 11.

- **Type II** – This mix was composed of Mineral Aggregate No. 2-A, Mineral Aggregate No. 5 (fine sand), and mineral filler.

- **Type IIR** – This mix was identical to Type II, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate.

- **Type 3** – This mix was composed of crushed quartzite or granite, with mineral filler added if required. It was used for the same types of projects as Type 13.

- **Type 3R** – This mix was identical to Type 3, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. It was used in the same types of projects as Type 13.

- **Type 4** – This mix consisted of at least 30% crushed rock. Crushed mineral aggregates may include no more than 20% naturally occurring fine aggregates retained on the No. 10 sieve. Mineral filler may be added if required, and a maximum of 60% limestone was permitted. It was used for the same types of projects as Type 14.

- **Type 4R** – This mix was identical to Type 4, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. It was used in the same types of projects as Type 14.

- **Type 7** – This mix consisted of a combined mineral aggregate, with a maximum of 60% limestone permitted. It was used for the same types of projects as Type 17.

- **Type 7R** – This mix was identical to Type 7, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. It was used in the same types of projects as Type 17.

- **Type 11** – This mix was designed to achieve an 80% crushed value for the combined mineral aggregate, with a maximum of 60% limestone to enhance skid resistance. It followed a 75-blow Marshall design and targeted a field air void of 4.0%. Type 11 was intended for use on high-volume roads with a truck count of 350 or more.

- **Type 11R** – This mix was identical to Type 11, except that Reclaimed Asphalt Pavement (RAP) was used to supplement the virgin aggregate. All other properties remain the same as Type 11.

- **Type 13** – This mix was designed to achieve an 80% crushed value and was composed of at least 50% quartzite or granite. It followed a 75-blow Marshall design and targeted a field air void of 4.0%. Type 13 was used on high-volume roads, often as a cap over Type 11, and in urban projects where the lift thickness ranged from 2 to 2 ½ inches.

- **Type 13R** – This mix was identical to Type 13, except that Reclaimed Asphalt Pavement (RAP) was used to supplement the virgin aggregate. All other properties remain the same as Type 13.

- **Type 14** – This mix was designed to achieve a 60% crushed value for the combined mineral aggregate, with a maximum of 60% limestone to enhance skid resistance. It follows a 50-blow Marshall design and targets a field air void of 4.0%. It was used on medium-volume roads with truck traffic ranging from 125 and 350 vehicles.

- **Type 14R** – This mix was identical to Type 14, except that Reclaimed Asphalt Pavement (RAP) was used to supplement the virgin aggregate. All other properties remain the same as Type 14.

- **Type 17** – This mix was designed with no crushed value for the combined mineral aggregate, a maximum of 60% limestone for skid resistance, a 50-blow Marshall design, and a targeted field air void of 3.5%. It was used primarily for shoulders on the Interstate and Expressway systems. However, it was found to be susceptible to rutting.

- **Type 17C** – This mix was designed to have a crushed value of either 20% or 40% for the combined mineral aggregate, with a maximum of 60% limestone for skid resistance. It followed a 50-blow Marshall design and targeted a field air void of 3.5%. The 20% crushed value was used for shoulders on Interstate and Expressways, while the 40% crushed value was used for mainline pavement under traffic volumes of 125 trucks or fewer. However, it was found to be susceptible to rutting.

- **Type 17R** – This mix was identical to Type 17, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. All other properties remained the same as Type 17. However, it was found to be susceptible to rutting.

- **Type 17RC** – This mix was identical to the Type 17C – whether with 20% or 40% crushed value – except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. All other properties remained the same as Type 17C.

- **Type A** – This mix was composed of crushed rock, mineral filler, and 3-A crushed sand gravel. It was used historically as both a base and surface course.

- **Type A Special** – This mix was composed of crushed rock, mineral filler, and 3-A crushed sand gravel. It was used as a base course. Compared to the standard Type A mix, it featured a slightly coarser crushed rock gradation and a higher percentage of crushed rock content.

- **Type AX** – This mix was composed of crushed rock, fly ash, and mineral aggregate. It was used as both a base and surface course on Interstate projects. However, it was found to be susceptible to rutting.

- **Type AX Special** – This mix was composed of the same materials as Type AX but contained a higher percentage of crushed rock. It was used as a base course on Interstate projects.

- **Type RAX** – This mix was identical to Type AX, except that Reclaimed Asphalt Pavement (RAP) material was added to supplement the virgin aggregate. It was used in the same application as Type AX. However, it was found to be susceptible to rutting.

- **Type RAX Special** – This mix was identical to Type AX Special, except that Reclaimed Asphalt Pavement (RAP) material was added to supplement the virgin aggregate. It was used in the same applications as Type AX Special.

- **Type B** – This mix was composed of 70% to 86% 2A Gravel, 10% to 20% fine sand, and 4% to 10% mineral filler, with asphalt cement content ranging from 3.5% to 5.5%. It was used for mainline paving on rural roadways throughout the state. However, it was found to be susceptible to rutting.

- **Type Q** – Quartzite
  This mix was composed of crushed quartzite or crushed granite. It was used as a surface layer on the Interstate.

- **Type RQ** – RAP with Quartzite
  This mix was identical to Type Q, except that Reclaimed Asphalt Pavement (RAP) material was used to supplement the virgin aggregate. It was used for the same types of projects as Type Q.

- **Type MQ** – Mixed Quartzite
  This was an open-graded mix composed of quartzite or granite, gravel, sand, and mineral filler. It was used as a surface layer on the Interstate.

- **Types CC, CC1 & CC2** – Crushed Concrete
  These mixes were composed of crushed concrete, 3-A sand, and mineral filler.

- **Type RCC** – Recycled Crushed Concrete
  This mix was composed of Reclaimed Asphalt Pavement (RAP), approximately 82% crushed concrete, and 18% 3-A sand gravel. It was used as a base course on Interstate projects.

- **Type SMA** – Stone Mastic Asphalt
  This European mixture was composed of crushed rock, 3-A crushed gravel, and mineral filler. It was used on high-traffic volume roads and is a gap-graded mix.

- **Type GGCRM** – Gap Graded Crumb Rubber Modified
  This mix was placed as a surface course, typically 1.5 to 2.5 inches thick. It resembled a Stone Mastic Asphalt (SMA) mix and featured a high binder content, making it resistant to rutting and cracking. It was used on high-volume roadways.

- **Type GGCRMLV** – Gap Graded Crumb Rubber Modified Low Volume
  This mix was placed as a surface course, typically 1.5 to 2.5 inches thick. It resembled a Stone Mastic Asphalt (SMA) mix and featured a high binder content, providing resistance to rutting and cracking. It was used on low to medium volume roadways.

- **Type RLC** – Regular Leveling Course
  This mix was used as a leveling course and included fractionated Reclaimed Asphalt Pavement (RAP). It had the same gradation as a standard Leveling Course (LC) and utilized standard PG binder types and contents, targeting typical mainline volumetrics. The current Type LC mix is the same as the Type RLC mix.

- **Type OGFC-CRM** – Open Graded Friction Course – Crumb Rubber Modified
  This mix was placed as a surface course, typically 1 to 1.5 inches thick. It was coarser than a standard Open Graded Friction Course (OGFC) and contained a higher binder content. The mix used a 58-28 binder modified with crumb rubber, providing a high-friction, well-drained, and quiet pavement surface. It was used on mainline roadways and ramps.

- **Type HRB** – High Rap Base
  This was a very fine-graded, single-aggregate mix used only in lower lifts. It contained a minimum of 25% or 35% RAP (as specified) and a maximum of 50% RAP. The mix did not include lime and used a minimum of 5.5% PG 64-22 binder (PG 64-34 starting in 2010). It was a very stiff mix designed for lower lifts on low to medium volume roadways. Type HRB was constructed for approximately two seasons.

- **Type SPL** – Static Pressure Loading
  This was a well-graded Marshall mix available in both fine and coarse gradations. It was primarily used for camper pads, parking lots, lower lifts, and temporary pavement. While RAP was not required, it was often added to achieve the required 230 psi bearing capacity. The mix contained no lime and used a minimum of 5.2% PG 64-22 binder (PG 64-34 starting in 2010). Type SPL has been replaced by Type SPR.

- **Types SP1 – SP6** – SuperPave 1-6
  These mixes were designed for a range of traffic loads, with Type SP1 intended for the lowest traffic and Type SP5 for the highest. All mixes were developed using Superpave criteria, including criteria such as Coarse Aggregate Angularity (CAA), Fine Aggregate Angularity (FAA), number of gyrations to achieve 4% air voids in the lab, elongated pieces, clay content, Voids in Mineral Aggregate (VMA), Voids Filled with Aggregate (VFA), binder content, and dust-to-asphalt ratio. Types SP1 through Type SP3 sometimes exhibited rutting issues. As a result, only Types SP4, SP4 Special, and SP5 were eventually used, with Type SP5 designated for roadways carrying 750 or more trucks per day. Type SPH mix is the same as the Type SP5 mix.

- **Type SPR Fine** – This mix meets the requirements of Type SPR except that the gradation allowed greater variance making it easier to include additional RAP. The variance in gradation gave the contractor more control based on how the asphalt was placed. Often this mix was placed in lift thicknesses less than 1.5 inches as a leveling course and was still capable of being used in intermediate or top lifts on the same project. The current Type SPR asphalt mix has the same gradation as the Type SPR Fine.

- **Type SP4 Special** – SuperPave 4 Special
  This mix meets the requirements of Type SP4 except the gyratory effort to meet target air voids was reduced. This mix was intended for use on Roadways with lower truck volumes than Type SP4.

- **Type SPH** – Superpave Heavy-load (> 750 Trucks/Day)

  Type SPH was designed for heavy truck applications, such as Interstates, expressways, and high-volume urban corridors. It incorporated high-angularity aggregates and typically included 15% to 25% RAP. Gyratory compaction levels were adjusted to meet performance standards, improving binder content and dust-to-asphalt ratios. These modifications aimed to enhance long-term durability, reduce permeability, and improve in-place density.

  However, Type SPH was more commonly used as an asphalt base course, as its high-fine aggregate angularity led to longitudinal joint density challenges. This mix used high polymer-modified PG 58H-34 binder to deliver enhanced performance under heavy loading. Type SLX Super-Mod replaced the Type SPH.

6.3 Current Nebraska Asphalt Mix Designs

Nebraska uses several different asphalt mix designs, selected based on factors such as pavement distresses, truck traffic volumes, the existing pavement structure, and the intended use of the proposed asphalt. Currently, the Nebraska DOT utilizes a modified version of the Superpave Method to develop these mix designs. This modified Superpave system specifies asphalt binders and mineral aggregates and provides a framework for developing and analyzing asphalt mixtures.

NDOT continually develops, modifies, and refines its asphalt mix types based on actual field performance to better address key performance parameters. The list below outlines and describes the most recent updates and applications of NDOT's flexible pavement mix designs.

- **Type LC** – Leveling Course
  Type LC is primarily used as a scratch course or leveling course, typically placed over bare concrete or heavily patched, rough roadway segments. Its main purpose is to improve surface smoothness before placing subsequent asphalt lifts. This mix is extremely fine-graded and requires the use of fractionated RAP. The result is a dense mix that incorporates polymer-modified binders to enhance performance.

- **Type LC-S** – Leveling Course SAMI Layer
  Type LC-S is a specialized leveling or scratch course designed for use over bare concrete or heavily patched, rough pavement. Like Type LC, it aims to enhance surface smoothness before placing additional asphalt layers. However, Type LC-S incorporates features of a Stress Absorbing Membrane Interlayer (SAMI), making it especially effective at mitigating reflective cracking. This extremely fine-graded mix has low air voids and a high binder content, resulting in a dense mixture that utilizes significant amounts of polymer-modified binder for improved flexibility and durability.

- **Type SPR** – Standard Paving Recycle (≤ 750 Trucks/Day)
  Type SPR is a Superpave mix designed for roadways with daily truck traffic of 750 or fewer. It combines high-quality angular aggregates with approximately 45% RAP. The mix uses highly polymerized PG 58H-34 binder and optimized dust-to-asphalt ratios to produce a rich mastic with increased film thickness. These characteristics contribute to a high strength modulus, providing superior structural capacity, rut resistance, and improved in-place density. Type SPR also offers enhanced laydown and placement characteristics, resulting in lower permeability and greater resistance to aging and longitudinal joint deterioration.

- **Type SPR35 –** Standard Paving Recycle (35% Maximum RAP)
  Type SPR35 is the same as the Type SPR mix but limits the maximum RAP content to 35%. This asphalt mix was developed to prevent the recycling of existing asphalt layers with higher RAP percentages and to reduce the stiffness of the new asphalt layer.

- **Type SLX** – Surface Laminate X-treme
  Originally developed for thin-lift maintenance projects, Type SLX is now used in Expressway and Interstate overlay applications, as well as on high truck volume corridors. It offers improved joint density compared to Type SPH, making it a preferred choice for urban, high-traffic environments. The mix design includes a minimum of 20% ¼-inch limestone chips, which enhance aggregate angularity. Type SLX uses highly polymer-modified binders, typically PG 58V-34 binder, with PG 58E-34 for Interstates and select Expressway projects to meet higher performance demands.

- **Type SLX Super-Mod** – Surface Laminate X-treme (Modified)
  Type SLX Super-Mod is similar to the Type SLX mix, with the primary difference being a higher required aggregate angularity. It is intended for use as a surface wearing course on heavily loaded corridors and arterial roadways in Omaha and Lincoln.

- **Type HPS4** – High Performance Surfacing Mix (4 mm)
  Type HPS4 is a high-performance, gap-graded surface course mix. The mix requires 25% crushed ledge rock with a mineralogy of quartzite, granite, or dolomite to produce a durable, friction-enhanced wearing surface. The maximum aggregate gradation size is a No. 4 sieve, which corresponds to the "4" in HPS4. It is typically placed in a 1-inch to 1½-inch thick layer. This mix does not allow RAP and requires a PG 58E-34 binder.

- **Type SRM** – Special Reclamation Mix
  Type SRM is a mix developed as an alternative recycling strategy, allowing for up to 50% RAP. It includes a minimum of 10% crushed rock, resulting in a stiff base mix suitable for bridging areas affected by asphalt stripping. Due to its stiffness, Type SRM requires a surface course overlay using either Type SLX or Type SPR. The high stiffness of the RAP binder is balanced with the addition of a PG 58H-34 binder to improve overall performance.

- **Type SPS** – Surfaced Paved Shoulders
  Type SPS is a mix specifically designed for surfaced shoulders. It uses a PG 58S-34 binder and is formulated to achieve a target air void content of 1.5%. The mix encourages the use of RAP at rates between 35% and 50%, significantly reducing the need for virgin binder and aggregates – potentially by up to 50%. Notably, Type SPS contains no lime. As a general guideline, if the total quantity of Type SPS on a project is less than 5,000 tons, the mainline mix should be used for surfaced shoulder instead, based on cost-effectiveness and constructability.

The Nebraska Department of Transportation has begun exploring the use of Balanced Mix Design (BMD) approaches and incorporating performance testing methods using the IDEAL-CT (Cracking Tolerance) and IDEAL-RT (Rutting Tolerance).

6.4 Asphalt Binders and Emulsions

Binders

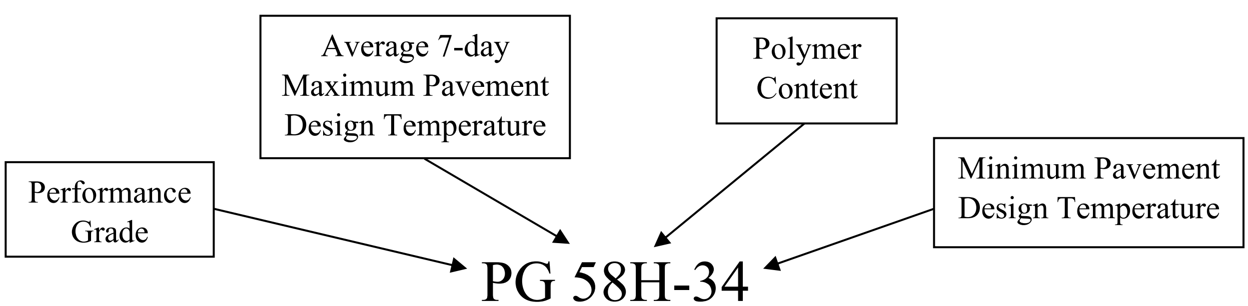
A binder is a substance that acts as a glue to hold the asphalt together. It is a byproduct of petroleum refining, derived from the thick residue left after distilling petroleum to produce fuels and lubricants. To meet the specific requirements of asphalt pavement, binders are produced from carefully selected crude oil blends and processed to achieve the appropriate grade.

The Performance Graded (PG) system is a method used to evaluate asphalt binder performance through a series of tests that measure its physical properties. Based on these test results, each binder is assigned a PG designation reflecting its suitability for specific traffic and pavement temperature conditions. The PG grade consists of two numbers representing pavement temperatures in degree Celsius: the first number (e.g., PG 58-XX) indicates the high pavement temperature, while the second number (e.g., PG XX-34) indicates the low pavement temperature. It is important to note that these temperatures refer to pavement – not air – temperatures. The high temperature rating addresses the binder's resistance to rutting, whereas the low temperature rating relates to its ability to resist fatigue cracking in cold conditions.

The Multiple Stress Creep Recovery (MSCR) grading system eliminates the need to "bump" the high-temperature grade for roadways with high truck volumes to resisting rutting. Previously, a binder graded for 58°C might have been increased to 64°C to improve rut resistance. Under the MSCR system, the actual 7-day average maximum pavement temperature (in °C) is used directly, and traffic loading is addressed through a letter designator that reflects the expected truck volume and corresponding polymer modification level. The letter designators are as follows:

- S – Standard traffic
- H – Heavy traffic
- V – Very heavy traffic
- E – Extremely heavy traffic

For example, a binder graded under the PG system indicates both the minimum and maximum design temperatures and includes a letter designator to specify its suitability for various truck traffic volumes.



Emulsions

An emulsion is a liquid mixture consisting of asphalt, water, and an emulsifying agent.

An emulsifying agent is a chemical additive that enables the asphalt and water to form a stable emulsion. Acting similarly to soap, it allows tiny droplets of asphalt to remain suspended in water, creating a workable liquid form of asphalt that can be applied during road construction and maintenance. Emulsifying agents are typically surfactants, meaning they contain both hydrophilic (water-attracting) and hydrophobic (oil-attracting) components. This dual nature allows them to bond with both the water and asphalt phases, maintaining stability in the mixture.

Depending on the specific performance requirements of the emulsion and its interaction with aggregates, different types of emulsifiers can be used, including:

- Cationic Emulsion – A positively charged emulsion. Cationic emulsions are designed to interact with negatively charged mineral aggregates, improving adhesion and bond strength. They are typically emulsified using fatty acids and are widely used due to their rapid breaking behavior and strong compatibility with many aggregate types.

- Anionic Emulsion – A negatively charged emulsion. Anionic emulsions bond well with positively charged mineral aggregates and are often preferred in cooler climates due to their slower breaking rates. They are usually emulsified using “tall oils”, such as wood oil.

Another important characteristic of an emulsion is its setting time – the time it takes for the emulsion to bond to the surface after application. Setting time is influenced by several factors, including the type of emulsion, application rate, surface temperature, and environmental conditions. Once the emulsion has set, is said to have “broken”, meaning the water within in the mixture has evaporated. Allowing the suspended asphalt droplets to adhere to the pavement surface

Bitumen emulsions are typically classified into four categories based on their setting time:

- Slow Setting (SS) – In slow setting emulsions, the bitumen and the water remain mixed for an extended period and do not separate easily. These emulsions begin to adhere to the surface only after the water has fully evaporated. Their longer curing time allows for greater workability and flexibility during application, making them ideal for jobs requiring extended handling or mixing. However, the slower break time can result in longer project durations and potentially higher costs.

- Medium Setting (MS) – Medium setting emulsions separate (or “break) more quickly than slow setting emulsions but still allow for short handling or mixing time. They are formulated to maintain workability for a few minutes after application, ensuring proper distribution before breaking begins.

- Quick Setting (QS) – Quick setting emulsions break faster than medium setting emulsions, with the bitumen and water separating almost immediately after application. They are designed for use in situations with strict time constraints, such as surface treatments where minimal traffic disruption and rapid curing are essential.

- Rapid Setting (RS) – Rapid setting emulsions break almost immediately upon contact with the pavement surface, typically within 20 minutes of application. They are ideal for high-demand conditions, such as urban or nighttime work zones, where quick curing is essential due to traffic constraints or temperature sensitivity. Their fast break time helps minimize delays and ensures performance even in challenging environments.

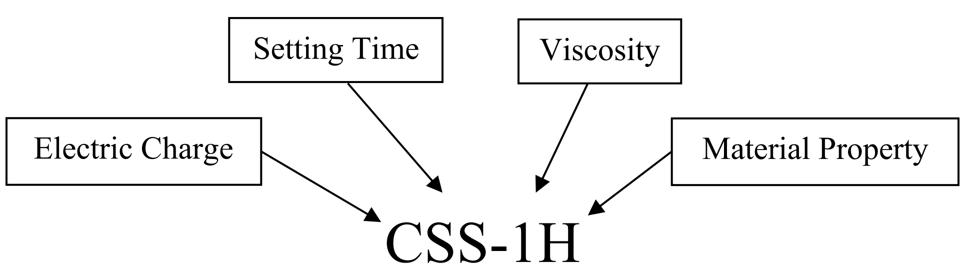
Emulsions, like binders, use a combination of abbreviations and numbers to identify their type and grade. The emulsion designation typically consists of a leading abbreviation and a trailing number or letter. The leading abbreviation indicates the emulsion's electric charge (cationic or anionic) and setting time (rapid, medium, slow, or quick setting). The trailing number and/or letter describes the emulsion's material and viscosity properties, such as polymer modification or high float characteristics.

1. Electrical Charge – The first part of an emulsion's name indicates its electric charge. If the emulsion contains positively charged bitumen particles (cationic), the name will begin with a "C". If the emulsion contains negatively charged bitumen particles (anionic), there is no letter at the beginning.

2. Setting Time – The second part of an emulsion's name indicates its setting time. The abbreviations corresponding to the setting categories described earlier (e.g., Slow Setting = SS, Medium Setting = MS, etc.).

3. Viscosity – The third part of an emulsion's name indicates its viscosity. Emulsion with low viscosity end with a "1", while emulsions with a higher viscosity end with a "2".

4. Material Property – The fourth part of an emulsion's name identifies specific material properties based on the additives used in its formulation. For instance, a "P" indicated that the emulsion is polymer-modified, an "L" indicates the use of latex, or "H" indicates the hardness of the material. These modifiers enhance performance characteristics such as elasticity, durability, and resistance to cracking.



Below are several types of emulsions commonly used by NDOT for various project applications, each selected based on specific performance requirements and construction conditions.

Table 6.4-1 – Emulsions Acronym List

Electric Charge		Setting Time		Viscosity		Material Property	
Acronym	Description	Acronym	Description	Acronym	Description	Acronym	Description
“_”	Anionic	SS	Slow Setting	1	Low Viscosity	H	Hard
C	Cationic	MS	Medium Setting	2	High Viscosity	P	Polymer
		FS	Fast Setting			L	Latex
		QS	Quick Setting				
		RS	Rapid Setting				

Other Acronyms:

- ARA – Asphalt Rejuvenating Agent
- HFE – High Float Emulsion
- VH – Very Hard

Below is a list of current emulsions used by NDOT for various pavement strategies.

Table 6.4-2 – Current NDOT Emulsions List

Work Description	Grade
Armor Coat or Chip Seal	CRS-2P, HFE-150, CRS-2VHL
Bituminous Sand Base Course	HFE-300
Cold In-Place Recycling	HFE-300
Fog Seal	CFS-1, FS-1, CSS-1H, SS-1H
Hot In-Place Recycling	ARA
Hydrated Lime Slurry Stabilization	SS-1, SS-1H, CFS-1, FS-1, CSS-1, CSS-1H
Microsurfacing	CQS-1H
Scrub Seal	CRS-2P
Tack Coat	SS-1, SS-1H, CSS-1, CSS-1H

Emulsion Calendar Work Schedule

Temperature requirements are critical for emulsion applications, as they significantly influence the stability and performance of the emulsion. Temperature affects the physical properties of both phases – such as viscosity, interfacial tension, and surfactant solubility – which in turn impact the emulsion's overall behavior and durability. Maintaining proper temperatures during emulsification, storage, and application is essential to ensure product quality and to prevent issues like phase separation or coalescence. Below is a list of pavement strategies that utilize emulsions, along with the temperature requirements for their proper application.

Table 6.4-3 – Temperature Requirements for Emulsion Applications

Type of Work	Ambient Air Temperature
Armor Coat or Chip Seal	60˚F Minimum
*Bituminous Sand Base Course	60˚F Minimum
** Fog Seal	60˚F Minimum
*** Microsurfacing	50˚F Minimum

* The application of asphaltic materials will not be allowed after September 15th.

** On certain projects, both fog sealing and crack sealing of bituminous surfaces are specified. For these cases, a late spring or early summer letting is recommended to allow the fog seal to be completed between June 1st and September 1st, followed by crack sealing during the winter months, between November 1st and March 31st. See Chapter 8 of this guide for Crack Sealing Bituminous Surfaces.

*** Microsurfacings can be applied when the ambient temperature is 50˚F or higher, but it must remain above 32˚F for at least 24 hours after placement.

For the most current temperature requirements, refer to the latest version of the Standard Specifications for Highway Construction and/or the project-specific Special Provisions.

6.5 Pavement Geosynthetics

Pavement geosynthetics are synthetic materials designed to enhance the performance of asphalt pavement systems. Installed between asphalt lifts, they act as a tensile reinforcement layer, helping reduce cracking caused by traffic loading. These materials are commonly used in both full-depth asphalt pavements and composite pavement structures, where they are embedded within the asphalt layers.

The two most widely used types of pavement geosynthetics are textiles and grids. Textiles are nonwoven, petroleum-based sheets that function as waterproofing membranes or reinforcement layers. In contrast, grids are open, grid-like structures – often made from fiberglass – designed primarily to provide tensile reinforcement and control reflective cracking.

Nonwoven Pavement Textile

A nonwoven pavement textile is a nonwoven, needle-punched polypropylene sheet used primarily as a waterproofing membrane within asphalt pavement structures.

When properly installed, pavement textiles help extend pavement life in three key ways:

- Preventing water intrusion into the pavement structure.
- Slowing reflective cracking from underlying asphalt or concrete layers.
- Increasing the fatigue life of the asphalt by providing stress absorption.

Figure 6.5-1 – Nonwoven Pavement Textile

Proper binder application is critical to ensure proper bonding between the pavement textile and the surrounding asphalt layers. Without adequate binder, the textile cannot adhere effectively, reducing its overall performance.

Pavement Grid

A pavement grid is a geosynthetic reinforcement system used in asphalt road construction to prevent or reduce cracking and extend pavement life. It consists of a grid-like mesh made from fiberglass strands coated with an elastomeric polymer. The material is formed from weaving individual fibers into a textile-like structure, which is embedded between asphalt layers. Pavement grids provide tensile strength to the asphalt, helping to resist cracking caused by underlying pavement movement and repeated traffic loads.

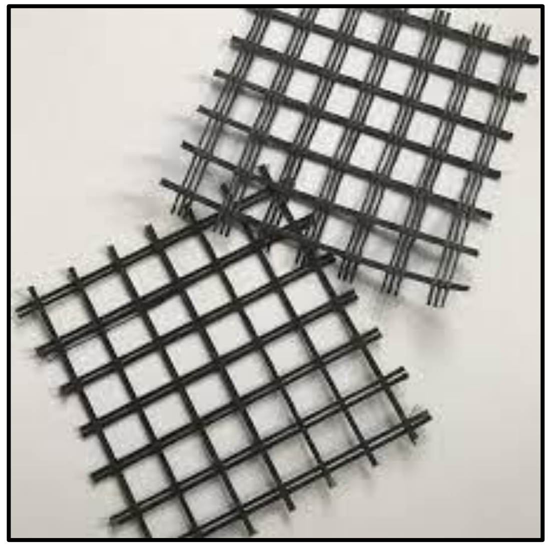


Figure 6.5-2 – Pavement Grid

Composite Pavement Reinforcement System

A composite pavement reinforcement system integrates a high-strength pavement grid with a synthetic backing or membrane to create a multifunctional geosynthetic layer designed to facilitate asphalt laydown operations. The backing acts as a bond breaker during construction so that the grid is not tracked or wrinkled by construction equipment during the placement of asphalt.

The composite may feature a pressure-sensitive or adhesive backing to eliminate or reduce the need for liquid asphalt binder and improving installation efficiency. Once installed between asphalt layers, the system resists cracking, and contributes to longer pavement service life under repeated traffic loads and thermal cycling.

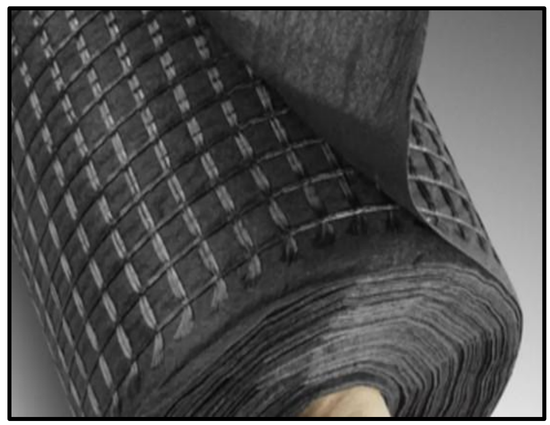


Figure 6.5-3 – Composite Pavement Grid with Nonwoven Pavement Textile Backing

6.6 Asphalt Mix Design Guidance

The following is a series of general guidelines that the NDOT uses when selecting asphalt mix designs.

Use	Mix Type
Mainline: 0 – 750 Heavy Trucks per day	SPR SPR35
Expressway and High Traffic Urban	SLX SLX Super-Mod HPS4
Interstate	SLX SLX Super-Mod HPS4
Shoulder	SPS (Surfaced Shoulder ≥ 5,000 tons) Mainline Mix (Surfaced Shoulder < 5,000 tons)

Mix Type	Performance Graded Binders
Leveling Course	
Type LC	PG 58E-34 (with Fractionated RAP)
Type LC-S	PG 58E-34 (virgin mix/no RAP)
Surfaced Shoulder	
Type SPS	PG 58S-34
Mainline Mix Type	Mainline Mix Type Binder
Mainline	
Type SPR	PG 58H-34 PG 58V-34
Type SPR35	PG 58V-34
Type SLX	PG 58V-34 (Lower Volume) PG 58E-34 (Interstate)
Type SLX Super-Mod	PG 58E-34
Type SRM	PG 58H-34
Type HSP4	PG 58E-34

General Asphalt Thickness Guidance

Heavy Trucks per Day	Thickness**
0 – 200	8”
200 – 1600	10” (8” in pure sand regions)
1600+	12” (10” in pure sand regions)

**Pavement ME Design currently used to determine structural thickness.

6.7 Flexible Pavement Strategies

The Nebraska Department of Transportation employs a variety of strategies to preserve, rehabilitate, and restore the performance of flexible pavements. These treatments are selected based on pavement condition, traffic levels, and desired service life. Below are several key strategies NDOT uses to maintain and improve flexible pavements.

New & Reconstruction Strategies

Full-Depth Asphaltic Concrete Pavement

As the name implies, Full-Depth Asphaltic Concrete Pavement involves a complete reconstruction of the roadway by removing existing pavement layers down to the subgrade and rebuilding from the ground up. Milling machines or excavation methods are used to remove the existing pavement. The exposed subgrade must be properly prepared and stabilized, treated with Fly Ash, Cement or Lime – depending on the soil type – to provide a strong, uniform foundation.

Multiple layers of new asphalt are then placed over the prepared subgrade. The final pavement thickness is determined using Pavement ME Design, which considers traffic loading, subgrade support, and environmental factors to ensure long-term performance.

Resurfacing, Restoration, and Rehabilitation (3R) Strategies

Hot Mix/Warm Mix Asphalt Overlay (without Milling)

A hot mix asphalt (HMA) overlay, or a warm mix asphalt (WMA) overlay is a paving technique where a new layer of hot mix asphalt is laid directly over an existing asphalt surface. It is commonly used to restore the smoothness, appearance, and functionality of roads that are still structurally sound but have surface-level wear and tear. Before applying the overlay, the existing asphalt is prepared and may be patched to ensure a proper bond and stability of the now base course material. The new asphalt layer, usually around 1 to 4 inches thick, then provides a refreshed, durable surface.

Resurfacing is ideal for pavements with minor to moderate damage, such as cracking, rutting, or surface deterioration. It also adds structural capacity by increasing pavement thickness. Asphalt overlays offer a quicker alternative to repaving, extending the life of the pavement while improving its performance.

Mill and Placement of Asphalt Overlay

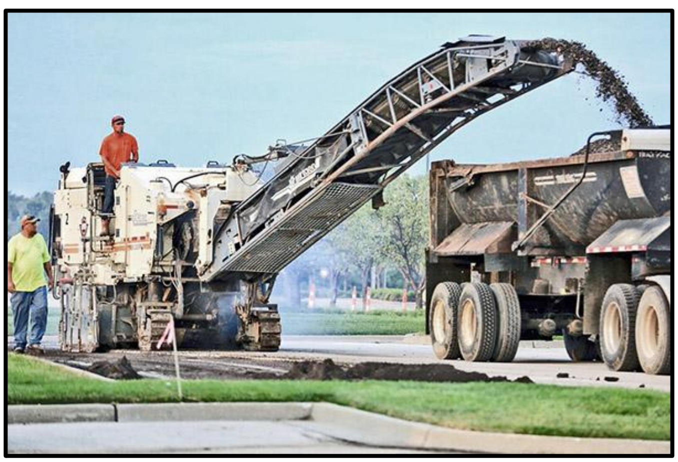
A Mill and Placement of Asphalt Overlay is a pavement maintenance or rehabilitation strategy that involves removing the top layer of worn or damaged asphalt (milling) and replacing it with a new layer of fresh asphalt. This method is commonly used to restore roadways that have surface deterioration such as ruts, cracks, or unevenness but still have a solid underlying base.

Depth and overlay thickness are determined based on project-specific conditions, including:

- The type and thickness of existing asphalt lifts, along with the total depth of the pavement structure.
- Pavement core conditions, such as stripping, cracking, and bonding to underlying pavements.
- Falling Weight Deflectometer (FWD) data, including back-calculated pavement and subgrade modulus values.
- Applicable design standards (e.g., Preventative Maintenance vs. 3R) and the intended design life.
- Available budget and funding constraints.

Milling machines grind off the top few inches of the existing surface, typically between 2 to 4 inches, without disturbing the foundation. After the surface is cleaned and prepped, a new layer of asphalt is applied and compacted to create a smooth, durable finish.

This process offers several advantages over a simple asphalt overlay. Because the damaged top layer is removed first, the new surface bonds better and tends to last longer. It may also maintain the original road elevation, which is important for drainage and reduced grading impacts. Milling and overlay is a way to significantly improve pavement conditions without the expense of full reconstruction.



Asphalt Pavement Milling

Asphalt Overlay Paving

Hot In-Place Recycling

Hot In-Place Recycling (HIR) is an asphalt pavement rehabilitation technique that involves heating the existing pavement surface, remixing it with rejuvenating agents or new binders, and then laying it back down – all on site. The process typically treats the top 1 to 2 inches of asphalt and is performed using a specialized train of equipment that completes heating, scarifying, mixing, and repaving in a continuous operation. HIR restores the pavement's flexibility and smoothness without the need to remove or haul away old material.

This method is particularly effective for roads with surface-level issues like cracking, raveling, or oxidation, provided the underlying pavement structure is still sound. HIR offers two benefits, including faster construction times and utilizing 100% RAP on site. It's an efficient option for extending the life of asphalt pavements with minimal disruption to traffic.

General guidance for Hot In-Place Recycling includes:

- The HIR process involves a long train of specialized equipment, including alternating propane burners and milling heads:
    - The propane burners heat the existing asphalt surface.
    - Milling heads remove a shallow layer of softened bituminous material.
    - The reclaimed material is then reheated, mixed with additional binder, and re-laid using a laydown machine before final compaction.
    - Because of the long train operation, roadways with sharp or multiple curves are not ideal.
- HIR can be used on its own to rejuvenate the pavement surface or in combination with a subsequent mill and fill asphalt overlay.
- HIR is comparative in cost and performance to a Mill 3”/Fill 3” asphalt overlay.
- A wearing surface is typically applied over the HIR layer:
    - Most commonly, a Hot Mix Asphalt overlay is used as the wearing course.
    - On low-volume roads, an Armor Coat may be used instead.
- A mix design is developed by a private laboratory and must be approved by the Materials & Research Division.
- HIR is not suitable where concrete repairs are needed or where asphalt patching is anticipated.
- HIR should not be used on roadways with extensive thermal cracking or significant amounts of crack sealant.
- Do not use on roadways that have received more than two applications of Armor Coat or Chip Seal.
- The process is best suited for recycling Superpave asphalt mix types.

Hydrated Lime Slurry Stabilization

Hydrated Lime Slurry Stabilization (HLSS) is an asphalt pavement rehabilitation method used to address surface distresses such as cracking, stripping, and depressed thermal cracks. It may also be applied in cases of moderate rutting as lime effectively stiffness of the asphalt mix, improving its structural integrity. HLSS is most effective when applied to pavements with at least 6 inches of hot mix asphalt (HMA) over a soil aggregate base course (SABC) or a bituminous sand base. The process involves milling the pavement down to just above the base layer and treating the reclaimed material with a mix of emulsion and hydrated lime slurry. After a curing period, a new asphalt overlay is applied. Before treatment, Falling Weight Deflectometer (FWD) testing and core sampling are necessary to verify subgrade strength and pavement thickness, ensuring the site can support the heavy stabilization equipment.

General guidance for Hydrated Lime Slurry Stabilization includes:

- Pavements with 6 inches or more of Hot Mix Asphalt (HMA) over a Soil Aggregate Base Course (SABC) or a bituminous sand base are best suited for HLSS treatment.
- Falling Weight Deflectometer (FWD) testing and pavement coring are required to assess the subgrades and pavement’s ability to support the heavy HLSS equipment.
- Historical project data should be reviewed to verify pavement thickness.
- Machinery must be capable of processing 3 to 5 inches of bituminous material; 4 inches deep is typical for Nebraska.
- The typical treatment includes 1.5% CSS-1 emulsion and 1.5% hydrated lime slurry by weight.
- A fog seal must be applied to prevent moisture infiltration, especially ahead of storms or to reduce surface raveling.
- A 3-inch asphalt overlay is typically placed on top of the stabilized layer.
- The overlay should be installed after a 7-day curing period but no later than 28 days after HLSS application.
- Mix design is prepared by a private lab and must be approved by the M&R Division.

Fly Ash Slurry Injection

Fly Ash Slurry Injection is a specialized pavement treatment method of utilizing a mixture of Fly Ash and water (slurry) to repair thermal cracks. This treatment is not used frequently in Nebraska primarily due to challenges associated with accessing and treating thermal cracks. While thermal cracking is a common pavement distress, many of these cracks lack a continuous underlying void, limiting the effectiveness of slurry injection as a solution. When viable, this method is employed alongside standard rehabilitation treatments such as milling and asphalt overlays to restore structural integrity and minimize surface deformation.

The process involves drilling near targeted thermal cracks and injecting a Fly Ash-based slurry into subsurface voids. This injection must not exceed a ½-inch lift of the pavement. For the material to be considered structurally sound, the slurry must meet a minimum unconfined compressive strength of 400 psi after 7-days. Any proposed Fly Ash mix design must be submitted to the M&R Division for approval prior to field use.

General guidance for Fly Ash Slurry Injection includes:

- Rarely used in Nebraska due to inconsistent subsurface voids beneath thermal cracks.
- Used in conjunction with milling and asphalt overlay.
- Involves drilling and injecting Fly Ash slurry to fill voids beneath thermal cracks.
- Injection is limited to a ½-inch pavement lift.
- Slurry must reach a minimum 7-day unconfined compressive strength of 400 psi.
- Mix designs require M&R Division approval before application.

Fly Ash or Cement Stabilized Bituminous

Fly Ash or Cement Stabilized Bituminous treatment is a full-depth pavement rehabilitation strategy used when surface distresses are severe, such as extreme cracking, stripping, depressed thermal cracks, or rutting. It is primarily selected when the existing pavement has deteriorated beyond the point of conventional rehabilitation and when Hydrated Lime Slurry Stabilization is not viable – often due to the pavement's inability to support the heavy stabilization equipment. However, this method is a short-term solution in areas with poor subgrade conditions. Long-term strategies are needed to address subgrade and drainage concerns.

The process typically involves stabilizing the entire pavement structure along with 1 to 2 inches of the underlying subgrade. Specialized reclaiming equipment can reach depths of up to 16 inches. A Cement or Fly Ash additive is used to strengthen the reclaimed material, which is then covered with a new asphalt overlay. The stabilized layer must cure for at least 2 days and should be overlaid within 28 days. A fog seal is applied to protect the surface during curing.

General guidance for Fly Ash or Cement Stabilized Bituminous includes:

- Used when severe cracking, stripping, depressed thermal cracks, or rutting are present.
- Not effective in poor subgrade conditions. Poor drainage needs to be addressed for long-term preservation.
- Stabilizes full-depth pavement plus 1 to 2 inches of subgrade; reclaiming equipment can process up to 16 inches deep.
- Asphalt overlay requirements:
  - Typical 4 inches of asphalt overlay thickness
  - Curves with > 2° Degree of Curvature (< 2865ft Radius): 4-inch asphalt overlay minimum thickness to prevent the asphalt overlay from slipping or tearing.
  - Asphalt overlay must be placed after 2 days and within 28 days of stabilization.
- Stabilizing agents:
  - Fly Ash – 8% to 12%
  - Cement – 3% to 7%
  - Water – Approximately 4%
- Strength target: Minimum 90 psi after 7-day moist cure and 24-hour dry period. Desirable to have 120 psi to 300 psi.
- Fog seal is applied within 2 hours after the stabilization has begun for surface protection and curing prior to overlay.
- Prior to placing the asphalt overlay, milling is required to establish the proper slope and improve initial surface smoothness. A second fog seal should be applied immediately after milling to protect the exposed surface.
- Milling shall be performed a minimum of 2-days prior to the asphalt overlay to allow traffic to knead the fog seal into the roadway surface.
- Mix designs provided by the M&R Division; process governed by method specifications.

Cold In-Place Recycling w/ High Float Emulsion

Cold In-Place Recycling with High Float Emulsion (CIR w/HFE) is a specialized pavement rehabilitation technique that is typically applied on existing asphaltic or bituminous sand pavements to restore old, dry, and cracked surfaces. This process uses High Float Emulsion, generally applied at rates between 2.5% and 3%, and is designed based on Marshall Stability and Retained Stability criteria. Due to its limitations and evolving practices, CIR w/HFE has largely been replaced by Cold In-Place Recycling using Foamed Asphalt.

General guidance for Cold In-Place Recycling with High Float Emulsion includes:

- Rarely used in Nebraska due to its sensitivity to environmental conditions.
- Detours are required.
- Applied sparingly to existing asphaltic or bituminous sand pavements.
- Utilizes High Float Emulsion (HFE-300).
- HFE is typically applied at a rate of 2.5% to 3%.
- Mixes are designed using Marshall Stability and Retained Stability criteria.
- Mix design is provided by a private lab and approved by the M&R Division; field production is governed by method specifications.

Cold In-Place Recycling

Cold In-Place Recycling w/ Foamed Asphalt

Cold In-Place Recycling with Foamed Asphalt (CIR w/ Foam) is a rehabilitation technique used to create a stable base layer in pavements exhibiting significant stripping, pavement distresses, or substantial patching. The process involves injecting water into hot PG binder – maintained at a minimum temperature of 300ºF – causing it to foam and expand. This foamed binder coats and binds the RAP, improving cohesion and strength.

The equipment and recycling train used in CIR w/ Foam is similar to that of Hydrated Lime Slurry Stabilizations, consisting of a scalping shaker, crusher (to reduce oversized particles), pug mill, a material transfer vehicle or pickup machine, and a paver. This is followed by a compaction sequence involving pneumatic rollers and steel drum rollers. The recycling depth is typically between 4 inches and 5 inches.

General Guidance for Cold In-Place Recycling with Foamed Asphalt includes:

- Rarely used in Nebraska due to its sensitivity to environmental conditions.
- Used to create a stable base when significant stripping, pavement distress, and/or extensive patching is present.
- Typical asphalt overlay thickness is 3 inches.
- Uses a PG binder; during recycling, the binder is maintained at a minimum of 300ºF and foamed by water injection to promote RAP cohesion.
- Designed using Marshall Stability and Retained Stability.
- Mix design is provided by a private lab and approved by the Materials & Research Division.

Preventive Maintenance Strategies

Microsurfacing

Microsurfacing is a pavement preservation treatment designed to extend the life of existing asphalt surfaces. It's a fast-setting surface treatment that improves ride quality, restores surface texture, fills ruts and cracks, and seals the pavement to prevent further deterioration. This mixture is applied cold (without heating) in a slurry form and spreads evenly across the pavement surface using a specialized paving machine. Unlike slurry seals, microsurfacing sets and cures quickly, making it ideal for roads that need to reopen to traffic within a few hours.

Microsurfacing slurry as a mixture of:

- Polymer-modified Emulsified Asphalt
- Fine Aggregate (with Fine Angularity)
- Mineral Filler
- Water and Other Additives

General guidance for Microsurfacing includes:

- Intended for roads with minor surface distress (oxidation, raveling, minor cracking).
- Prevents oxidation, restores surface texture, seals minor cracks, and improves skid resistance.
- Typically applied between ⅜ to ½ inch thick.

Microsurfacing

Fog Seal

A Fog Seal is a pavement preservation treatment in which a diluted asphalt emulsion is lightly sprayed onto the surface of an existing asphalt road. It is primarily used to seal hairline cracks and protect the pavement from moisture infiltration and oxidation. Fog seals are often used in conjunction with other preventive maintenance strategies – for example, to seal shoulders during a Microsurfacing project.

Crack Seal

A Crack Seal is a preventive maintenance treatment that involves filling pavement cracks with a rubberized hot pour sealant to prevent water, debris, and other materials from penetrating the surface. By sealing these openings, the process helps extend the life of the asphalt pavement and minimize the need for more extensive repairs in the future.

Armor Coat or Chip Seal

An Armor Coat or Chip Seal is a pavement surface treatment used to seal and protect asphalt roads. Both methods involve applying a layer of asphalt emulsion followed by a layer of aggregate, which is then rolled into the binder to create a protective, wearing surface. These treatments are effective at sealing minor cracks and surface defects, protecting the underlying pavement from water infiltration and oxidation, and extending pavement life at a relatively low cost.

While Armor Coats and Chip Seals are similar in both application and purpose, key differences exist in the material specifications and intended use of the roadway. Aggregates used in Armor Coats must have a soundness loss of no more than 5% by mass after five cycles, compared to 7% for Chip Seal aggregates. Additionally, Armor Coat aggregates are typically a sand and gravel mix, while Chip Seals may use limestone, expanded shale, granite, or quartzite.

General guidance for Armor Coats and Chip Seals includes:

- Both Armor Coats and Chip Seals use the same polymer-modified asphalt emulsion.
- Armor Coats:
  - Use a sand and gravel aggregate blend with a maximum soundness loss of 5% after 5 cycles.
  - Typically applied at a thickness of ¼ to ½ inch.
  - Due to the polished, non-angular characteristics in gravels, Armor Coats tend to lose aggregate quicker than a Chip Seal.
- Chip Seals:
  - May use limestone, expanded shale, granite, or quartzite with a maximum soundness loss of 7% after 5 cycles.
  - Typically applied at a thickness of ¼ to ½ inch.
- A fog seal may be applied after installation to help reduce aggregate loss and lock in surface material, however, NDOT does not do this due to loss in friction numbers.
- Armor Coats and Chip Seals can typically be applied up to two times before a pavement rehabilitation strategy should be considered.

7.1 Introduction

Rigid pavement, also known as concrete pavement, is a type of road surface constructed with concrete to create a stiff, inflexible structure that offers high strength and a durable wearing surface – ideal for heavy traffic loads and harsh weather conditions. It typically consists of concrete slabs or panels placed over a drainable foundation course on a prepared or stabilized subgrade. These slabs may be reinforced with steel, depending on the design requirements and the pavement's intended use. Wire mesh reinforced concrete was used in the 1960s and earlier but current designs use a plain jointed concrete pavement, built with slipform pavers and dowel bars.

Rigid pavement is a common choice for highways and high-traffic areas due to its long-lasting durability and reduced maintenance needs compared to flexible pavement. However, it comes with higher initial costs and is generally more complex to install.

7.2 Admixtures

An admixture is a material, other than water, aggregates, cementitious materials, and fiber reinforcement, that is added to a cementitious mixture before or during mixing to modify its properties in the fresh, setting, or hardened state. Admixtures are used to make concrete more suitable for specific applications such as workability, set time, strength, permeability, freezing point, and curing. They have a significant impact on both fresh and hardened concrete properties and can act through chemical and/or physical mechanisms, influencing the hydration process of cement.

There are many different admixtures available for use in concrete construction. The following are the most common admixtures used in rigid pavement:

- Type A – Water-Reducing Admixture
  An admixture that reduces the water content of a given mix without affecting its consistency and/or increases the slump of the mix without increasing the water content.

- Type B – Retarding Admixture
  An admixture that extends the initial set time, thereby increasing the workability time and delaying early strength development.

- Type C – Accelerating Admixture
  An admixture that reduces the initial setting time and increases early strength.

- Type D – Water-Reducing and Retarding Admixture
  An admixture that reduces the required mixing water for a given slump while also slowing down the setting time of the concrete.

- Type E – Water-Reducing and Accelerating Admixture
  An admixture that reduces the required mixing water for given slump and accelerates both the setting time and early strength development.

- Type F – Water-Reducing, High Range Admixture
  An admixture that reduces the required mixing water by 12% or more while maintaining the same slump.

- Type G – Water-Reducing, High Range and Retarding Admixture
  An admixture that reduces the required mixing water by 12% or more while delaying the setting time of the concrete.

- Air-Entraining – An admixture that introduces a controlled system of small air bubbles during mixing, which remains in the hardened concrete and improves durability.

- Lithium Nitrate – An admixture used to control the Alkali-Silica Reaction (ASR) in concrete.

The Nebraska Department of Transportation utilizes a variety of these admixtures in concrete construction. ASTM C494/C494M provides the Standard Specification for Chemical Admixtures for Concrete, while ASTM C260/C260M provides the Standard Specification for Air-Entraining Admixtures for Concrete.

7.3 Cement Types

Cement is a binding agent that hardens when mixed with water and is a key ingredient in concrete. In Nebraska, several types of cement can be used depending on the intended application. Portland cement is the most used type, particularly for concrete pavements. It is a fine powder made from limestone, clay, and other natural materials, and primarily composed of lime, silica, and alumina, with a small amount of gypsum added during manufacturing to control setting time.

To enhance concrete performance, Portland cement can be blended with other materials to form blended cements. These are hydraulic cements made by uniformly mixing Portland cement with Supplementary Cementitious Materials (SCMs), such as slag, fly ash, and natural clay pozzolans. Blended cements are designed to improve durability, sustainability, and specific properties such as sulfate resistance. The ASTM provides both prescriptive and performance-based specifications for various types of blended cements.

Definitions:

- **Supplementary Cementitious Materials** – Supplementary Cementitious Materials (SCMs) are materials used as a partial replacement of Portland cement to improve both fresh and hardened concrete properties. The most commonly used SCMs in concrete mixtures are natural pozzolans, fly ash, slag, and silica fume.

- **Pozzolan** – A pozzolan is a material that reacts chemically with calcium hydroxide and water to form cementitious compounds, enhancing the concrete's resistance to chemical attack. Natural pozzolans include calcined clay, calcined shale, volcanic ash, volcanic glass, and husk ash. Artificial (or industrial) pozzolans include fly ash and silica fume. The Nebraska DOT commonly incorporates pozzolans in blended cements for concrete pavements.

- **Ternary** – A ternary is a mixture blended with three different cementitious materials. This combination typically consists of Portland cement and two other cementitious materials. In general, ternary mixtures perform in a manner that can be predicted by knowing the characteristics of the individual ingredients. A benefit of ternary mixtures is that negative properties of one SCM can be offset by positive properties of another. Overall, the combined use of SCMs in this manner allows for the reduction of Portland cement content and can lead to performance, economic, and environmental benefits.

- **Fly Ash** – Fly ash is a byproduct of the coal burning electric power generating plants. After the ignition of coal in the furnace, the ash residue is carried away by the exhaust gases and is then collected using electrostatic precipitators or in filter bag houses. As the fly ash travels to the collectors, the material cools and forms spherical, glassy particles.

  There are two classes of fly ash: Class F and Class C. Fly ash is categorized into these classes based on chemical composition. Class F fly ash contains higher amounts of silica and lower amounts of calcium, while Class C fly ash typically contains lower silica and higher calcium contents. Both classes are acceptable for use in concrete pavement.

  - **Class F Fly Ash** is generally pozzolanic, meaning it possesses little to no cementing properties on its own. However, in the presence of water and calcium hydroxide, it reacts to form compounds with cementing properties. Class F fly ash is excellent in improving the long-term durability of concrete. It is effective in reducing permeability and mitigating alkali-silica reactions (ASR), delayed ettringite formation (DEF), and external sulfate attack at relatively low replacement rates.

  - **Class C Fly Ash** has both pozzolanic and cementing properties. Due to the higher calcium content, it reacts with water and hardens. Class C fly ash is effective in reducing permeability but is less effective in mitigating alkali-silica reactions (ASR), delayed ettringite formation (DEF), and external sulfate attack. Higher replacement rates are required to mitigate ASR and DEF, and due to the chemistry of the glass phase, Class C fly ash is more susceptible to external sulfate attack and should not be used in sulfate-rich environments.

- **Slag Cement** – Slag cement is a hydraulic cementitious material made from iron blast-furnace by-products that are ground into a fine powder. Molten slag, diverted from the iron blast furnace, is rapidly chilled to form glassy granules. When these granules are ground to cement fineness, they develop the desired reactive cementitious properties.

- **Silica Fume** – Silica fume is an ultrafine powder by-product of producing silicon metal or ferrosilicon alloys. It consists primarily of silicon dioxide (SiO_2). The individual particles are extremely small, approximately 1/100th the size of an average cement particle. Due to its very fine particle size, large surface area, and high SiO_2 content, silica fume is a highly reactive pozzolan when used in concrete.

Table 7.3-1 – Properties of Concrete Mixtures and the General Effects of Each SCM Type on that Property

Property	Class C Ash	Class F Ash	Slag Cement	Silica Fume
Initial Set Time	+	+	+	-
Strength Gain (Early)	+	-	-	+
Strength Gain (Late)	0	+	+	< >
Setting Time	+	+	+	-
Heat of Hydration	-	-	-	+
Plastic Shrinkage Cracking	< >	< >	< >	+
Permeability	-	-	-	-
ASR Mitigation	< > or 0	+	< > or 0	+
Sulfate Attack Mitigation	0	+	< > or 0	+

Note: *+ = Indicates an Increase*
- = Indicates a Decrease
0 = Indicates No Change
< > = Indicates that the Effect Varies Depending upon the Characteristic of the SCM or the Replacement Level

The following are types of cement and cement blends for a range of applications.

- **Type I** – Normal Portland Cement
  Type I Cement is a General Construction Cement. With the introduction of pozzolans, NDOT no longer uses this type of cement for permanent concrete pavements. However, Type I Cement may still be used for concrete pavements intended as Temporary Surfacing.

- **Type II** – Moderately Sulfate-Resistant Portland Cement
  Type II Cement was used in water or soil with moderate sulfate levels, or where moderate heat buildup is a concern. Same as with the Type I Cement, NDOT no longer uses this type of cement for permanent concrete pavements. However, Type II Cement may still be used for concrete pavements intended as Temporary Surfacing.

- **Type III** – High Early Strength Portland Cement
  Used in precast operations or concrete repair work where early strength is needed. The cement is ground finer, so it hydrates quicker. Also beneficial in cold weather, where hydration is slower.

- **Type IL** – Portland-Limestone Cement
  A blended cement containing 5% to 12% limestone. Used for general construction, offering a reduced carbon footprint compared to Type I/II.

- **Type IP** – Portland-Pozzolan Cement
  A blended cement containing up to 25% pozzolan, typically fly ash. Offers improved performance such as increased compressive strength and better workability compared to Type I/II.

- **Type IS** – Portland Blast-Furnace Slag Cement
  Pre-blended or inter-ground cement containing between 25% and 40% slag. Typically, Nebraska maximum slag amount is 38%.

- **Type IT** – Portland-Ternary Cement
  A ternary blended cement that uses slag cement, limestone, and fly ash.

- **Type IPN** – Pozzolan-Blended Portland Cement (Class N)
  Pre-blended or inter-ground cement containing 25% ± 2% Class N pozzolan. A Class N pozzolan is a naturally occurring processed material, typically volcanic or siliceous material that conforms to ASTM C618 for use in concrete.

7.4 Current Nebraska Concrete Mix Designs

A concrete mix design is the process of determining the optimal proportions of ingredients – cement, water, aggregates, and admixtures – to achieve the desired concrete properties such as strength, workability, and durability for a specific application. In Nebraska, several different concrete mix designs are used, each tailored to produce a performance-based product suited to the conditions and environments the concrete will encounter. The selection of a mix design typically depends on factors such as the intended use, exposure conditions, structural dimensions, and required physical properties.

When specifying a concrete mix design for pavement or structural applications, the Nebraska Department of Transportation uses mix design name designators to indicate both the required mix type and the target 28-day compressive strength. For example, a specification may call for a Class 47B mix with an average 28-day compressive strength of 3,500 psi.

Mix Design Specification

Average 28-day Minimum Compressive Strength

Class 47B-3500

Nebraska DOT continually develops, refines, and improves concrete mix designs based on actual field performance to better address specific performance criteria. The following list outlines and describes the most recent updates and applications related to NDOT's rigid pavement mix designs.

- **Class 47B** – General Concrete Pavement
  This concrete mix design is primarily used for general concrete pavement applications and is the most commonly used mix by NDOT. Developed by the Nebraska Department of Roads in 1947, the 47B mix features a 70% fine aggregate with a 30% coarse aggregate blend. The typical minimum compressive strength requirement is 3,000 psi, though 3,500 psi is frequently specified depending on project needs.

- **Class 47BD** – Bridge and Structures
  This mix design follows the same general requirements as the Class 47B mix but is specifically used for bridge and structural applications. The typical minimum compressive strength of this mix is 4,000 psi. This concrete mix requires a 30% coarse aggregate and 70% fine aggregate blend.

- **Class 47B-HE** – High Early Strength
  This mix design follows the same requirements as the Class 47B concrete mix but is modified for situations where the pavement must be opened to traffic earlier than usual. A common example is a nighttime lane closure that requires reopening by morning to accommodate peak traffic volumes. To achieve high early strength, the mix includes additional cement to accelerate strength gain. This concrete mix requires a 45% coarse aggregate and 55% fine aggregate blend.

- **Class BX** – Sand & Gravel Mix
  This mix design is very similar to the Class 47B concrete mix, but it has a smaller aggregate gradation. Class BX does not have any ledge rock, just sand and gravel. This concrete mix can be used for Temporary Surfacing. Type I/II cement may be allowed. This concrete mix requires 100% fine aggregate blend.

- **Class 47B-OL** – Overlay
  This mix design is for concrete overlay applications. It uses a finer aggregate gradation with a water cement ratio of 0.36. This concrete mix requires a 45% coarse aggregate and 55% fine aggregate blend.

- **Class PR** – Pavement Repair
  This mix design is used for concrete repairs. Class PR concrete is a fast-setting concrete mix that is used for quick concrete repairs to open back up to traffic. Class PR concrete mix has several subdivisions of concrete mixes depending on the type of cement being used.

  - PR1 – Uses Type IL Cement

  - PR3 – Uses Type III Cement

  - PR1-4 – Blend of 75% Type IL and 25% Type IP or IT or IS. This concrete mix requires a 45% coarse aggregate and 55% fine aggregate blend.

The Nebraska Department of Transportation will allow the use of Tarantula Curve approach and incorporating performance testing methods for projects with at least 50,000 square yards of concrete pavement.

7.5 Evaluation of ASR in Concrete Pavement

Alkali-silica reaction (ASR) is a harmful chemical reaction that leads to cracking and deterioration in concrete. ASR occurs when reactive silica in certain aggregates reacts with alkalis (sodium and potassium) in the cement paste, forming an alkali-silica gel around the reactive aggregate particles. When this gel is exposed to moisture, it expands, generating internal pressure that causes tensile cracking around the aggregates. Once cracking begins, additional moisture can enter the concrete, further accelerating the ASR process and worsening the deterioration.

The ASR evaluation is based on the standard ASTM test methods listed below:

- ASTM C1260 – Standard Test Method for Potential Alkali Reactivity of Aggregates
- ASTM C1567 – Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate

ASTM C1260 determines and characterizes the reactivity of the aggregates within 28 days according to NDOT specifications and ASTM C1567 determines the mitigation of ASR with the use of Supplemental Cementitious Materials (SCM).

To assess the potential for ASR in concrete, the alkali metal content (sodium and potassium) in cement is evaluated using the "Equivalent Alkali" value. Because sodium and potassium have different molecular weights, their combined contribution to ASR potential must be adjusted. This is done with the following formula:

$$Na_2Oeq = Na_2O + 0.658(K_2O)$$

where:
Na_2Oeq = Sodium Oxide Equivalent (%)
Na_2O = Sodium Oxide (%)
0.658 = Factor Difference in Molecular Weights between Sodium and Potassium
K_2O = Potassium Oxide (%)

The Na_2Oeq value is a key indicator of cement's alkalinity and its potential to contribute to ASR. For Nebraska, the alkalinity of Type I or II cement is typically 0.60% Na_2Oeq or less. According to AASHTO PP65, when cement alkalis are below 0.70% Na_2Oeq, the required dosage of Supplementary Cementitious Materials (SCMs) – such as fly ash or slag – can be reduced by one prevention level, generally 5% to 10%.

Steps to Evaluate Concrete Susceptibility to ASR in Existing PCC
(Based on Halsey/Heyen 2013 Guidance)

1. **Conduct a Visual Evaluation**

   Assess the concrete pavement through a field visit or alternative tools (e.g., Google Earth, PathWeb). Look for visible signs of ASR-related staining or cracking.

2. **Research Project History**

   Collect documentation such as As-Built plans, Special Provisions, Specifications, Proportion Reports, and any archived project records.

3. **Analyze Cementitious Materials**

   Using the Special Provisions and Proportion Report, determine the following:
   - Type of Cement used
   - Type and Percentage of Fly Ash used
   - Percentage of Natural Pozzolans (if any)
   - Maximum allowable Cement Alkali Content (in lbs/yd^3)

4. **Confirm Cement Use**

   If the Special Provisions or Specifications do not match the cement type used in the field, check for Change Orders that may have modified the cement specification.

5. **Identify Aggregate Source**

   Determine the likely watershed source for the sand and gravel aggregates based on the project location (using Figure 7.5-1). Then use Table 7.5-1 to identify the aggregate type location.

6. **Compare SCM Use to Required Levels**

   Use Table 7.5-1 to compare the minimum required Supplementary Cementitious Material (SCM) replacement level to the actual level used in the concrete mix.

   Note: *The green column in Table 7.5-1 shows the appropriate Fly Ash (Type F) replacement percentage for Nebraska cements.*

   Reminder: *Nebraska's Type I or II cements typically have ≤ 0.60% Na_2Oeq. Cement Alkali contents < 0.70% Na_2Oeq allow for a reduction of SCM by one prevention level (usually 5% to 10%).*

7. **Conduct ASR Testing (Optional)**

   If a pavement is suspected to be ASR-affected, confirm by testing core samples. Use the AASHTO T 299-93 (2019) method.
   - The sample is treated with Uranyl Acetate.
   - Under black light, ASR gel will fluoresce, confirming presence.

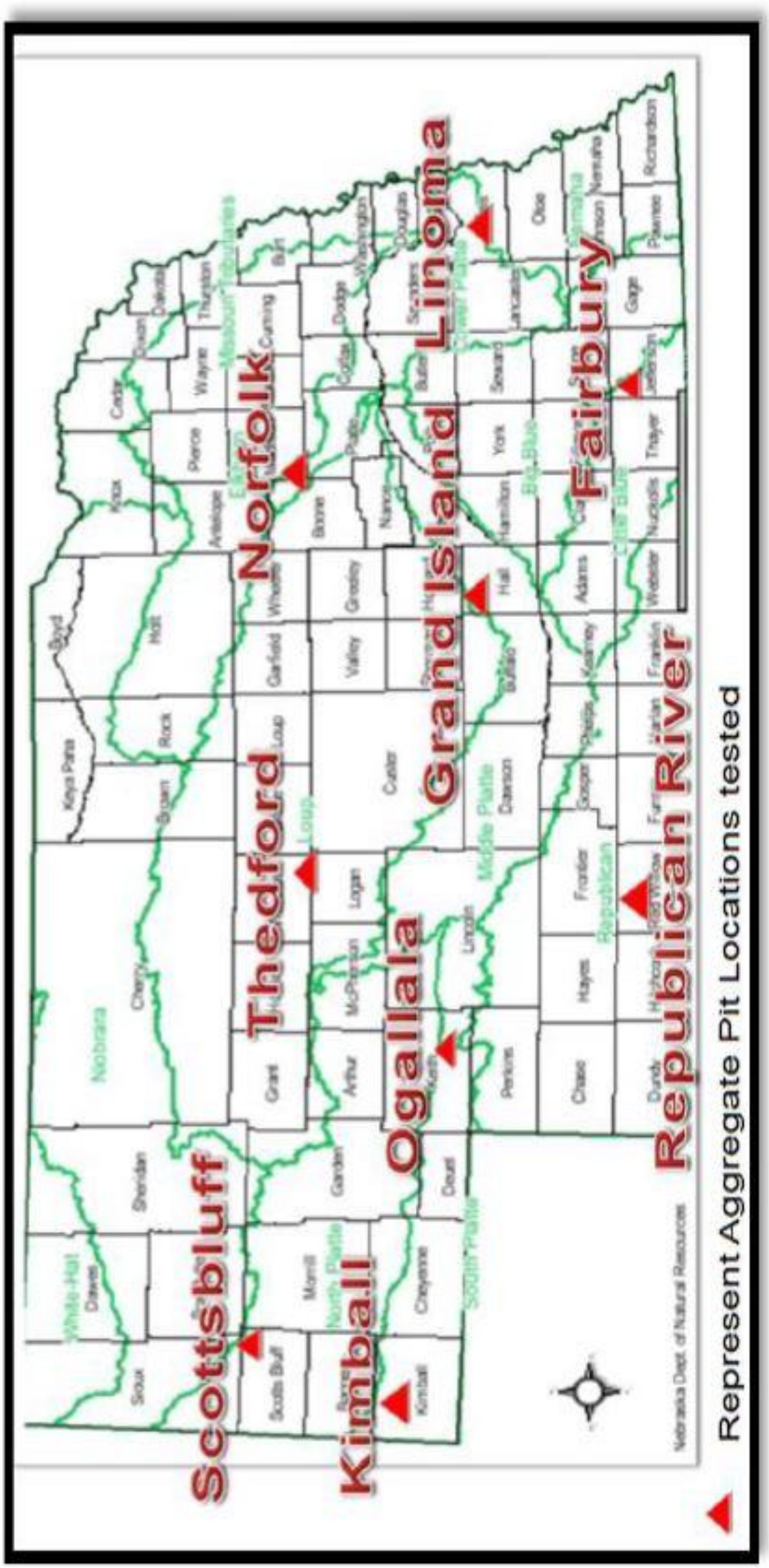


Figure 7.5-1 – Nebraska's Regional Aggregate Reactivity Study (December 2012)

Table 7.5-1 – Minimum SCM for Nebraska Aggregates

Aggregate Type & Location	Description of Aggregate Reactivity	Minimum Replacement Level of SCM	Minimum Replacement Level of SCM Mitigate ASR	NDOT's Specification Since Late 2004
		Table 6 of AASHTO PP65-10	Type I/II Cement Low Alkalinity	IP with 25% Class F
Platte River Grand Island	Moderately Reactive	20	15	✓
Dry Pit Kimball	Highly Reactive	25	20	✓
Republican River Indianola	Very Highly Reactive	35	25	✗
North Platte River Scottsbluff	Highly Reactive	25	20	✓
South Platte River Ogallala	Moderately Reactive	20	15	✓
Middle Loup River Thedford	Highly Reactive	25	20	✓
Little Blue River Fairbury	Moderately Reactive	20	15	✓
Elkhorn River Norfolk	Very Highly Reactive	35	25	✓
Platte River Linoma	Highly Reactive	35	20	✓

**The Republican River at Indianola, NE is a Non-Approved Aggregate source.*

NDOT General Guidance for ASR Prevention and Mitigation

Over the years, the Nebraska DOT has conducted extensive research on preventing and mitigating ASR in concrete. Table 7.5-2 below summarizes findings related to blending Fly Ash with Cement to reduce the susceptibility of concrete to ASR.

- NDOT no longer maintains Supplemental Cementitious Materials (SCMs) on the Approved Product List.
- NDOT permits the use of blended SCMs meeting ASTM C1697 – Standard Specification for Blended Supplementary Cementitious Materials. Suppliers must report the chemical composition of the final SCM blend, which NDOT verifies.
- NDOT allows the use of Type IP and IT cements in accordance with ASTM C595 – Standard Specification for Blended Hydraulic Cements. NDOT verifies both the chemical and physical composition of the final blended cement.
- Total cement replacement with SCMs typically ranges between 20% and 40%.
- Since 2007, Nebraska has used Type F Fly Ash at a 25% replacement rate as the Type IP cement component.
- More recently, NDOT allows for a combination of SCMs to mitigate ASR.

Additional information on ASR Prevention and Mitigation can be found in the PowerPoint Presentation below:

Recent Developments in the Preservation and Mitigation of ASR in Concrete:

PowerPoint Presentation

Nebraska's Aggregates Reactivity Evaluation According to AASHTO PP65-10:

\\dotfs\MR\In-House_Research\Presentations\2017 NC2 Salt Lake City

Table 7.5-2 – Cement Blended Results for ASR Susceptibility

Cement Strategy	Results
Type I/II Cement without SCMs	ASR Susceptible. Moderately reactive aggregates may develop ASR slowly.
Type I/II Cement with Fly Ash Type C	ASR Susceptible. Moderately reactive aggregates may develop ASR slowly.
Type I/II Cement with 17% Fly Ash Type F	Effective at mitigating ASR in moderately reactive aggregate. Development of ASR may be slow for highly and very highly reactive aggregate.
Type IPN Cement with 17.5% and 9% Fly Ash Type C	Effective at mitigating ASR in moderately reactive aggregate. Development of ASR may be slow for highly and very highly reactive aggregate.
Type IP Cement with 22% Fly Ash Type F	Effective at mitigating ASR for all but very highly reactive aggregate.
Type I/II Cement with Low Alkalinity and no SCMs	ASR susceptible. Deterioration may be slow with moderately reactive aggregate.
Slag	Effective at mitigating ASR for all but very highly reactive aggregate.

Additional information on ASR mitigation and identification can be found on the FHWA websites listed below:

- Research and Mitigation Guidance:
  https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/pccp/03047/02.cfm

- ASR Field Identification Handbook:
  https://www.fhwa.dot.gov/pavement/concrete/asr/pubs/hif12022.pdf

7.6 Rigid Pavement Strategies

The Nebraska Department of Transportation employs a variety of strategies to preserve, rehabilitate, and restore the performance of rigid pavements. Below are several approaches NDOT commonly uses for managing rigid pavement infrastructure.

New & Reconstruction

Concrete Pavement

Concrete pavement is used by the Nebraska Department of Transportation when constructing new roadways or replacing existing concrete surfaces that are no longer viable for repair. The appropriate thickness of the concrete pavement is determined using Pavement ME Design, but all concrete pavement design and construction must comply with the policy requirements outlined on Page 8 of this guide.

- Used when building new concrete or when existing concrete pavement is beyond repair and needs to be replaced.
- Thickness of concrete pavement is determined by Pavement ME Design.
- Follow the policy requirements found on Page 8 of this guide.

Resurfacing, Restoration, and Rehabilitation (3R)

Concrete Overlay

Concrete Overlays are used to rehabilitate existing bituminous or composite pavements by placing a new layer of Portland cement concrete directly over the existing pavement structure. This method enhances structural capacity and extends pavement life while reducing long-term maintenance. The overlays typically range from 6 to 8 inches in depth, although a minimum thickness of 5 inches may be used in certain applications.

Before the overlay is placed, significant milling is often required to control grade changes and maintain consistent elevation across shoulders, tie-ins, and adjacent infrastructure. Concrete overlays are best suited for thick HMA pavements in locations where full or partial detours can be implemented, as traffic disruptions during placement and curing must be carefully managed.

General guidance for Concrete Overlays includes:

- Applied over bituminous or composite pavements for structural rehabilitation.
- Minimum overlay depth: 5 inches. Typically, 6 to 8 inches thick.
- Pre-overlay milling is commonly performed to reduce grade raise and preserve existing geometry.
- Best suited for thick asphalt pavements with accessible detour routes.
- Provides durable, long-life pavement solution with reduced long-term maintenance needs.

Hot Mix/Warm Mix Asphalt Overlay

Hot Mix Asphalt (HMA) and Warm Mix Asphalt (WMA) overlays are commonly used by the Nebraska Department of Transportation to resurface and extend the life of existing concrete pavements. This technique involves placing a new layer of asphalt – typically about 4 inches thick – directly over the concrete, creating a smoother, more durable driving surface. Urban asphalt overlay thicknesses are often limited by the drainage constraints. Before overlaying, the concrete base is repaired to ensure it provides a stable foundation. A thin initial lift of asphalt, known as a leveling course, is usually applied first to correct surface irregularities and minimize bumps, especially at joints. After proper compaction of the leveling course, additional lifts of asphalt are added to form the final surface. This method improves ride quality, protects the underlying concrete from water infiltration and Alkali-Silica Reaction (ASR) damage, and adds structural strength. Asphalt overlays are ideal for concrete pavements showing minor to moderate surface distress and are considered a cost-effective, time-efficient alternative to full pavement replacement.

General guidance for Hot Mix/Warm Mix Asphalt Overlay includes:

- Most common resurfacing strategy for concrete pavements.
- Used when concrete has minor to moderate damage (e.g., cracking, faulting, surface wear).
- Involves repairing the concrete base and placing a new ~ 4" asphalt overlay.
- A leveling course is typically applied first to smooth the surface and joints.
- Adds structural capacity and extends pavement life.
- A 50-year concrete pavement design includes a 4" overlay at year 35.

Stress Absorbing Fiberglass Layer w/ Emulsified Asphalt

A Stress Absorbing Fiberglass Layer with Emulsified Asphalt (SAFLEA), also known as a Stress Absorbing Membrane Interlayer (SAMI), is a composite pavement treatment designed to enhance performance and extend pavement life. This specialized interlayer consists of chopped fiberglass strands placed between two layers of polymer-modified asphalt emulsion. Once applied, the fiberglass and emulsion are topped with Armor Coat aggregate and then covered by an asphalt overlay. SAFLEA serves multiple functions: it seals concrete surfaces against moisture infiltration and provides a cushioning layer that can help mitigate reflective cracking in asphalt overlays. This treatment is especially effective when used over cracked bituminous surfaces or as a protective barrier between existing concrete and a new asphalt layer, ultimately improving durability and ride quality.

General guidance for Stress Absorbing Fiberglass Layer with Emulsified Asphalt includes:

- Also known as a Stress Absorbing Membrane Interlayer (SAMI).
- Constructed with chopped fiberglass strands, polymer-modified emulsion, and aggregate.
- Placed beneath asphalt overlays to improve durability and performance.
- Seals PCC surfaces to protect against moisture infiltration.
- Helps reduce reflective cracking in asphalt overlays.
- Effective on badly cracked bituminous surfaces.

Crack and Seat

Crack and Seat is a specialized pavement rehabilitation technique used to prepare deteriorated concrete pavements for an asphalt overlay. The process involves fracturing the existing concrete pavement into smaller segments – typically about 3 feet wide in the transverse direction – using a truck-mounted guillotine hammer. These broken panels are then "seated" or pressed into the subgrade using an overweight single-axle cart to ensure firm contact and uniform support. This treatment creates multiple hairline fractures that help reduce reflective cracking in the overlay and minimize the need for extensive concrete repairs. A 3-inch to 4-inch asphalt overlay is placed over the seated concrete to restore the surface. Traffic can often be maintained during the process, and candidate roadways must have adequate drainage to prevent subgrade issues.

Crack and Seat is rarely used in Nebraska due to stability issues between the fractured concrete panels and the subgrade. It has been observed that these panels often act independently, leading to movement that causes cracking in the asphalt layers above. Saturated subgrades exacerbate this issue by increasing panel instability and must be addressed prior to construction. Additionally, Crack and Seat pavements present challenges for future concrete repairs and pavement widening projects. The reduced panel sizes created by fracturing prevent the installation of tie bars, making it difficult to properly connect new concrete sections to the existing pavement.

General guidance for Crack and Seat includes:

- Existing concrete pavement is broken into approximately 3' panels (transverse direction) by a truck mounted guillotine hammer. The small panels are then seated into the existing subgrade by an overweight single axle cart before a 3" – 4" thick asphalt overlay is applied.
- Multiple hairline fractures reduce reflective cracking of the original joints through the asphalt overlay and the amount of concrete repair work needed.
- Traffic is maintained throughout process.
- Rarely used in Nebraska due to stability concerns between panels and subgrade.
  - Fractured concrete panels may move independently, leading to cracks in asphalt layers.
  - Hinders future repairs and widenings – tie bars cannot be used with fractured panels.
- Requires good drainage to mitigate subgrade instability.

Cracking Concrete Panels

Seating Concrete Panels

Rubblization

Rubblization, also referred to as rubblizing, is a pavement rehabilitation technique used when concrete pavements have deteriorated beyond the point where methods like Crack and Seat or asphalt overlay are viable. This process involves fracturing the existing concrete slab into small, interconnected pieces using a resonant hammer. The resulting rubble remains in place and functions as a crushed concrete base for a new asphalt overlay. To support traffic loads, a substantial asphalt layer – typically 5 inches or more – is placed over the rubblized base. Rubblization requires detouring traffic during construction and is only suitable for pavements with adequate drainage. It is especially useful in cases of severe pavement distress, such as advanced Alkali-Silica Reaction (ASR), where other rehabilitation strategies are no longer effective.

General guidance for Rubblization includes:

- Existing concrete is reduced to a crushed concrete base by a resonant hammer.
- Significant asphalt overlay (≥ 5 inches) is required to carry traffic on top of the rubblization.
- Traffic must be detoured following rubblization.
- Best candidates are concrete pavements deteriorated past the point of rehabilitation by Crack and Seat or Overlay, such as concrete pavements with advanced ASR.
- Rarely used in Nebraska due to stability concerns between crushed concrete base and subgrade.
    - Crushed concrete may move independently, leading to cracks in asphalt layers.
    - Hinders future repairs and widenings – tie bars cannot be used with crushed concrete.
- Candidates must have good drainage.

Rubblization

Precast Concrete Panels

Precast Concrete Panels are prefabricated concrete slabs manufactured off-site and transported to the construction site for rapid installation. This method is particularly effective in high-traffic areas – such as ramps, intersections, and urban roadways – where minimizing lane closures and reopening the roadway quickly is critical. In Nebraska, especially in the Omaha area, precast panels are often used when repairs must be completed overnight to allow traffic to resume by the next morning. The process involves removing the damaged concrete, repairing the subgrade as necessary, and placing a subgrade leveling course to create a uniform platform. The precast slab is then set into position, and grout is used to fill any voids beneath the panel and encasing the dowel bars, ensuring full support and long-term performance.

General guidance for Precast Concrete Panels includes:

- Precast concrete panels are prefabricated off-site and installed quickly on-site.
- Used in urban areas where roadways must be reopened by the next morning.
- Ideal for high-volume traffic areas, ramps, and intersections.
- Damaged PCC is removed, and subgrade is prepared as needed.
- A subgrade leveling course is added to provide a uniform base.
- The precast slab is installed and grouted to fill voids and encase dowel bars to ensure support.
    - Bedding grout shall achieve a minimum compressive strength of 500 psi before opening to traffic, and a 28-day compressive strength of 2,500 psi.
    - Dowel Bar grout shall achieve a minimum compressive strength of 2,500 psi before opening to traffic, and a 28-day compressive strength of 4,000 psi.

Precast Concrete Panel

Preventive Maintenance

Joint Seal and Crack Seal

Joint Seal and Crack Seal treatments are preventive maintenance methods used to protect concrete pavements by sealing joints and cracks with rubberized hot-pour sealant. This process helps prevent the intrusion of incompressible materials – such as sand, gravel, or debris – that can become lodged in pavement openings. When concrete expands due to temperature changes, these materials can cause significant damage, including spalling or blowups. Sealing joints and cracks also prevents water and deicing chemicals from penetrating the pavement structure and causing additional damage, including freeze-thaw distresses. By sealing these openings, the process helps extend the life of the concrete pavement and minimize the need for more extensive repairs in the future.

General guidance for Joint and Crack Seal includes:

- Joint Seals
  - Require a reservoir to be sawcut into the pavement to contain the hot-pour sealant.
  - Pavement must be dry and clean for hot-pour sealant to adhere to the concrete surface.
  - Backer rods are not used in Nebraska due to water getting trapped and accelerating ASR deterioration.
  - Typically performed every 8 years to 12 years.
- Crack Seals
  - Surface must be dry and clean for proper sealant adhesion.
  - Typically performed every 8 years to 12 years.

For joint seals, a 12-year interval is ideal, resulting in only 2 resealing cycles over the pavements service life before an asphalt overlay. Frequent resealing can reduce sealant performance, as each sawing widens the joint and increases the volume that must be filled. This can lead to premature sealant failure and increased tire slap noise.

For more information on hot-pour sealers, please see Chapter 8 of this guide.

Diamond Grinding

Diamond Grinding is a corrective maintenance technique used to restore smoothness and improve the surface texture of concrete pavements. The process involves a specialized machine equipped with closely spaced diamond saw blades that remove a thin layer of the pavement surface. It is commonly used to correct surface irregularities such as faulting at joints, slab curling, and general roughness. In addition to enhancing ride quality, diamond grinding improves pavement friction and can help reduce traffic noise. However, this method is not recommended for concrete pavements with significant structural deficiencies or poor load transfer between slabs, as these issues can lead to the reoccurrence of faulting even after grinding. The process can be followed by a Penetrating Concrete Sealer to protect the treated surface and extend its service life.

Diamond Grinding

Dowel Bar Retrofit

Dowel Bar Retrofit (DBR) is a pavement preservation technique used to restore or enhance load transfer across transverse joints in existing concrete pavements. Over time, pavements can lose aggregate interlock, which results in faulting and reduced structural performance. DBR addresses this by cutting slots into the pavement at joint locations, inserting steel dowel bars, and backfilling the slots with a durable material such as epoxy. The embedded dowels help evenly distribute loads across adjacent slabs, reducing differential movement and prolonging pavement life. DBR is most effective when used on structurally sound pavements, but it is not recommended for pavements with advanced deterioration – such as those affected by Alkali-Silica Reaction (ASR) – as it may accelerate deterioration in such cases.

Dowel Bar Retrofit Blades

Dowel Bars Used in Dowel Bar Retrofit

Penetrating Concrete Sealer

Penetrating Concrete Sealer (PCS) is a surface-applied treatment designed to improve the durability and longevity of concrete pavements. Once applied, the sealer is absorbed into the concrete's pores, forming a water-repellent barrier below the surface. This barrier helps prevent the infiltration of moisture and chloride ions – especially from deicing chemicals – which can lead to the corrosion of reinforcing steel and contribute to Alkali-Silica Reaction (ASR) deterioration. By limiting moisture infiltration, PCS slows down these damaging processes and helps maintain pavement performance over time.

Currently, Silane sealers are the only type of approved Penetrating Concrete Sealers that the NDOT utilizes for concrete pavements and structures.

General guidance for Penetrating Concrete Sealer includes:

- Applied to concrete surface and absorbed into pores to form a protective barrier.
- Prevents water and chloride intrusion, reducing risk of steel corrosion and ASR.
- Help extend pavement life by limiting internal deterioration.
- Drying times may be an issue with different products and environmental conditions.
- Effective for 7 years to 12 years before reapplication is needed.

High Friction Surface Treatment

High Friction Surface Treatment (HFST) is a specialized pavement application designed to significantly increase surface friction, especially in wet conditions. It involves applying a layer of durable, polish-resistant aggregate – typically calcined bauxite – over a resin binder such as an epoxy polymer. HFST is commonly used on concrete pavements and bridge decks in areas with high friction demand, such as curves, intersections, and ramps.

General guidance for High Friction Surface Treatment includes:

- Uses calcined bauxite as the aggregate for optimal durability and skid resistance.
- An epoxy polymer is used to bond the aggregate to the pavement surface.
- Shot blasting is required on concrete surfaces before application to ensure proper bonding.

7.7 Joint Layout Practices for Rigid Pavement

Rigid pavements undergo expansion and contraction due to temperature fluctuations and differential curing, which can induce internal stresses and lead to cracking. To manage these stresses and prevent uncontrolled cracking, engineers incorporate “joints” into the concrete. A joint is a deliberately designed gap or weakened plane that controls where cracks occur and accommodates movement from thermal changes, thereby preserving the pavement's structural integrity and extending its service life.

There are 4 main types of joints used in concrete pavement design, each serving a specific function in controlling stress and maintaining performance:

- Contraction Joint – Also known as a Control Joint, this type of joint is typically created by sawing a groove into the concrete slab to form a weakened vertical plane. This plane is designed to control the location of cracking caused by internal stresses from moisture-related shrinkage, thermal contraction, temperature curling, and moisture warping. Contraction joints may be oriented transversely – with dowel bars to allow load transfer – or longitudinally – with tie bars to maintain alignment between adjacent slabs.

- Isolation Joint – This is a special-purpose joint placed between the concrete pavement and an adjacent pavement, fixed structure, or embedded object. Its function is to allow independent movement of the pavement in all directions without causing damage to either the pavement or the adjoining structure. Isolation joints typically contain a full-depth compressible material and do not include load transfer devices, tie bars, or other connections.

- Expansion Joint – This is a special-purpose joint constructed in new concrete pavements to accommodate potential excessive slab expansion or movement. Its primary role is to prevent the buildup of high compressive stresses that could otherwise lead to joint spalling, pavement blowups, or damage to adjacent structures. Unlike isolation joints, which allow full independent movement in all directions, expansion joints typically include dowels or other load transfer devices and permit movement primarily in the direction of expansion. Overuse of expansion joints should be avoided, as it can cause nearby contraction joints to widen over time, leading to sealant or filler failure, infiltration of water and incompressible materials, and a loss of load transfer efficiency.

- Construction Joint – These joints occur where concrete placement is stopped and later resumed, resulting in the interface between freshly placed and previously hardened concrete without special isolation measures. Construction joints provide a bond between the old and new concrete sections while allowing some movement. They can be oriented either transversely or longitudinally. Transverse construction joints are typically placed at the end of each day’s work but may also be used during paving stoppages caused by weather, project phasing, or equipment issues.

General Joint Layout Best Practices for Concrete Pavement

- The typical width of a concrete panel is 12’-0” for most roadways.
- The typical length of a concrete panel is 16’-6”.
- Longitudinal joints within the driving lanes should be avoided whenever possible.
- For 28’ wide roadways, a 14’-0” concrete panel is required.
    - Avoid placing a longitudinal joint between the 12’-0” wide driving lane and the 2’-0” wide paved shoulder.
- Lane width measured to the back of curb greater than 14’-0” will require a longitudinal joint at the edge of the 12’-0” driving lane.
- Outside lanes on Interstate and Expressway systems require a longitudinal joint between the 12’-0” driving lane and the outside paved shoulder, regardless of pavement thickness.
- Inside lanes on Interstate and Expressway systems do not require a longitudinal joint between the 12’-0” driving lane and the inside paved shoulder if the paved inside shoulder is 4’-0” wide or less.
    - Inside lanes on Interstate and Expressway systems with a concrete pavement thickness of 10” or less require a longitudinal joint between the driving lane and the paved inside shoulder.
- For thinner concrete pavement (< 8” thick), follow the general guidance provided by the following equation:

$$Concrete\ Panel\ Width\ and\ Length\ (ft) = 1.5 * Thickness\ of\ Concrete\ (in)$$

For more information on Concrete Pavement Joints, please see the FHWA Technical Advisory on Concrete Pavement Joints:

https://www.fhwa.dot.gov/pavement/ta504030.pdf

For more information on NDOT’s concrete joint layouts, please see the policy requirements found on Page 8 of this guide.

Roundabout Intersections

A roundabout intersection is a type of circular junction featuring channelized, curved approaches that reduce vehicle speeds, entry yield control that gives right of way to circulating traffic, and a counterclockwise flow around a central island that minimizes conflict points. Roundabouts promote continuous traffic flow and can replace standard stop-sign or signal-controlled intersections.

Due to a roundabout's geometry, the concrete pavement is laid out in a circular rather than linear pattern to avoid longitudinal joints within the driving lane. Transverse joints are also non-linear and are angled perpendicular to the direction of traffic flow. Because basic jointing rules for concrete pavements do not fully address the unique conditions found in roundabouts, the roadway designer should provide pavement design teams with detailed joint layout plans at least one month before the PS&E submission.

<u>General Joint Layout Rules for Optimum Roundabout Performance</u>

Typical Layouts

- Use a radial pattern for transverse joints in the circulating roadway and truck apron, with longitudinal joints arranged as concentric circles.

- Transverse joints in the roundabout legs should be perpendicular to slab edges.

- Longitudinal joints should follow the curved centerline alignment and connect to the nearest transverse joint on the circulatory roadway.

- Curb and gutter should be tied to the legs and the circulatory roadway. Do not tie the truck apron to the circulatory roadway curb and gutter.

- A non-doweled expansion joint is required between the circulatory roadway and the truck apron.

- Roundabout legs should be isolated from the circulating roadway pavement.

Slab Sizing

- The maximum width for both longitudinal and transverse joints is 14'-0".

- Avoid concrete slabs narrower than 2'-0".

Joint Types and Locations

- Whenever possible, match existing joints or cracks to avoid "T" intersections of joints.
- Joints should meet or intersect with in-pavement structures such as manholes or utility boxes.
- Avoid joint angles that are less than 60°. Best practice is to dogleg joints at approximately 90° through curve radius points.
- Avoid slabs with interior corners or "L"-shaped slabs.
- Minimize the use of irregularly shaped slabs.
- Longitudinal joints should be tied.

The American Concrete Pavement Association (ACPA) is an excellent resource for joint layouts in roundabouts. For additional information, see the link below:

Joint Layout - ACPA Wiki

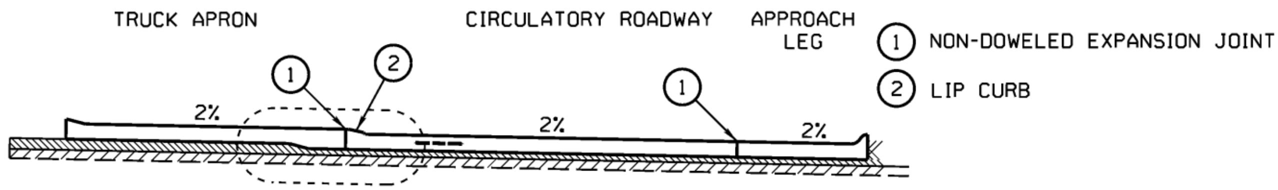
The Federal Highway Administration (FHWA) also offers a comprehensive technical brief on Jointed Concrete Pavement (JCP) in roundabouts. For additional information on roundabout layouts, visit the following link:

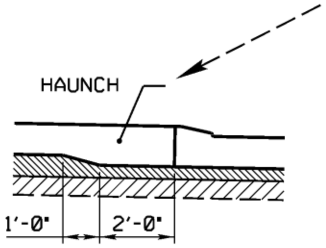
Tech Brief: Jointed Concrete Pavement (JCP) Roundabouts

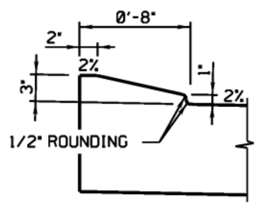
For geometry design and background information for roundabouts, please refer to:

Roundabouts: An Informational Guide

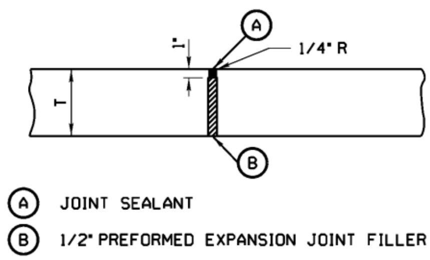
Roundabout Detail Examples



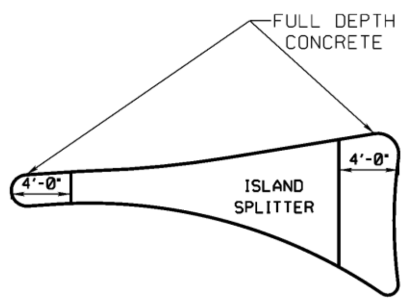




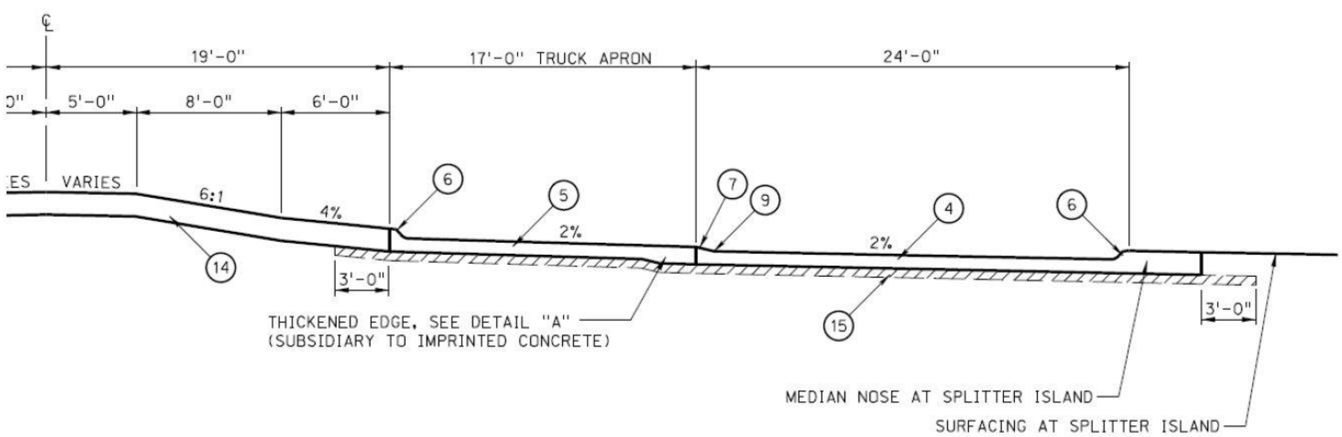
LIP CURB

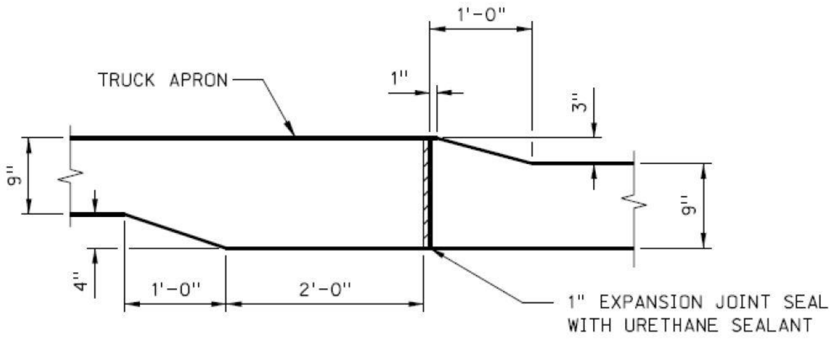


NON-DOWELED EXPANSION JOINT

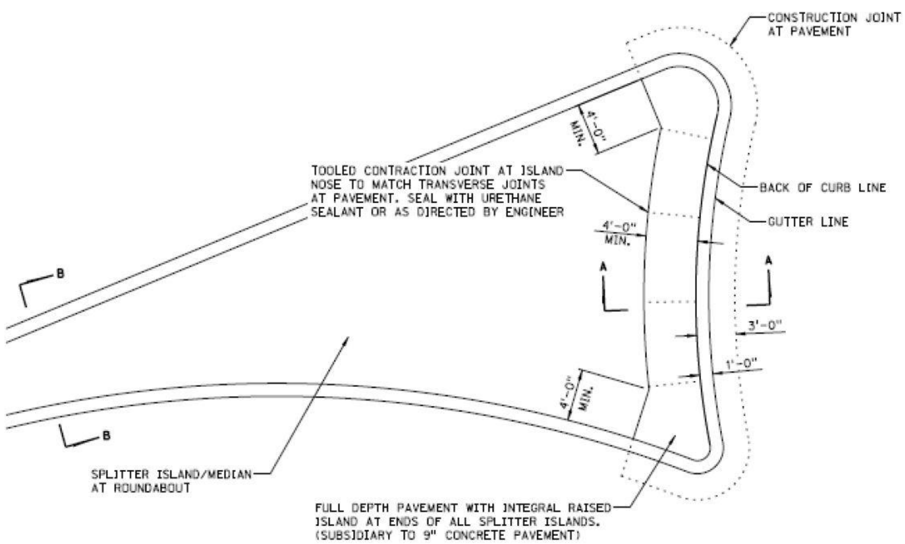


FULL DEPTH MEDIAN ISLAND NOSE





DETAIL "A"
THICKENED EDGE AT 3" HIGH INTEGRAL CURB



RAISED INTEGRAL ISLAND DETAIL AT SPLITTER ISLAND

8.1 Introduction

A hot-pour sealant is an asphalt-based material that is heated to a liquid state and poured into cracks and joints. Once cooled, the sealant hardens to form a flexible, durable, and weather-tight barrier. Hot-pour sealants prevent water and incompressible materials from entering the pavement, helping to maintain their performance over time. They are commonly used to fill cracks in asphalt pavements as well as joints and cracks in concrete pavements. Given that Nebraska is in a wet/freeze climate region, the use of sealants is especially important for preserving pavement integrity.

8.2 Nebraska Hot-Pour Sealants

The NDOT has conducted extensive research on hot-pour sealants and developed several formulations tailored for different scenarios and applications. Below are the four main hot-pour sealants that NDOT currently utilizes for joint and crack sealing in pavements.

- **NE-CR18B** – A specialty sealer developed by NDOT, specifically designed for sealing transverse and longitudinal cracks in asphaltic concrete that are greater than 3/8" wide. It contains 18% crumb rubber – hence its designation – and has the highest crumb rubber content among the four NDOT hot-pour sealants. This high rubber content provides superior resistance to pull-outs and wheel tracking. While all four NDOT sealants can be used for sealing cracks in asphaltic concrete, NE-CR18B is the preferred choice for addressing wide cracks. Additionally, a 2013 quality and specification review determined it to be the best option for use in the longitudinal intersplice joint between Portland cement concrete (PCC) mainline and asphaltic concrete shoulder.

- **NE-101** – A specialty sealer developed by NDOT, ideal for sealing transverse and longitudinal cracks in asphaltic concrete that are 3/8" wide or less. It contains 10% to 15% crumb rubber and has a higher polymer content than NE-CR18B. This composition makes it thinner and more adhesive, making it better suited for narrower cracks.

- **NE-3405** – Named after NDOT's use of the bond test criteria from the superseded ASTM D 3405 procedure, which is the basis for its designation. Aside from differences in bond testing methods, this sealer is more commonly known as 6690 Type II. It contains little to no crumb rubber and is widely used by DOTs nationwide as an all-purpose joint and crack sealer for both concrete and asphalt pavements. NDOT adopted this sealer for similar applications in 2013.

- **NE-3405LM** – NDOT uses different testing blocks in the bond test than those specified in ASTM D6690, which is why it retained the 3405 designation. This material is more commonly known as 6690 Type IV. Compared to NE-3405 (6690 Type II), it contains little to no crumb rubber but includes additional soft polymers. These polymers give it the LM designation, meaning Low Modulus. The added polymers make it better suited for colder climates – typically colder than Nebraska – by keeping the sealant softer and more flexible in low temperatures. Additionally, the low viscosity at application temperature allows it to penetrate more deeply into narrower cracks during cold weather. Some NDOT Maintenance Yards in the northwest portion of the state prefer using NE-3405LM, though it cost is approximately 27% higher than NE-3405, so cost versus practicality should be considered.

To access the Nebraska Department of Transportation's (NDOT) currently approved hot-pour sealant list, please visit the official NDOT Approved Products List:

Approved Products - NDOT

8.3 Hot-Pour Sealant Guidance

Asphalt Pavement

When crack sealing asphalt pavement, it is recommended to begin once 70% to 80% of the pavement surface exhibits cracking. This typically occurs between 5 and 6 years after an asphalt overlay has been completed depending on the lift thickness of the new asphalt. Cracks that are ¼ inch or wider should be filled or sealed prior to any maintenance surface treatments, such as fog seals, chip seals, or armor coats. Before applying hot-pour sealant, the cracks should be routed and cleaned using compressed air.

It was previously recommended to use a heat lance to warm the vertical edges of asphalt cracks; however, field observations revealed that this practice damages the surrounding pavement and prematurely ages the binder. NDOT now recommends limiting the use of heat lances solely to removing excess moisture from within the crack before sealing.

Concrete Pavement

Concrete pavement joints are sealed immediately after construction, with the sealing completed before the project finishes. Over time, hot-pour sealants age and weather, requiring resealing to maintain joint integrity. Typically, resealing occurs every 8 to 12 years throughout the concrete pavement's lifespan, until an overlay is applied.

When resealing, it is recommended to reface the existing joint up to a width of 1/8 inch greater than the original. This removes any remaining old sealant and creates fresh vertical surfaces for the new hot-pour sealant. These vertical surfaces must be thoroughly clean – dust, dirt, and sawing residue should be removed using compressed air, and the joint must be dry to ensure proper adhesion.

Heat lances may be used as needed to dry the concrete joint. However, the use of backer rods is no longer permitted. Field observations revealed that backer rods can trap water between concrete panels, leading to joint failures from freeze/thaw cycles.
Crack sealing for concrete pavement is typically performed alongside joint sealing or concrete repairs to minimize the duration of public lane closures. Before applying hot-pour sealant, the crack must be thoroughly cleaned and prepared. Any loose or detached concrete along the crack should be removed. Similar to asphalt crack sealing, the crack must be completely clean and dry before the sealant is installed.

Table 8.3-1 – Hot-Pour Sealant Selection Recommendation Charts

Bituminous Pavement	NE-101	NE-CR18B	NE-3405	NE-3405LM
Cracks	X	X	X	X
Best when majority of cracks are longitudinal (Tracking and Pullout Issues)	X	X		
Selection by crack width dimension. (Routing for cracks up to 3/8", rout to 1/2" wide x 3/4"-1" deep).	≤ 3/8"	> 3/8"	> 3/8"	>3/8"

Concrete Pavement	NE-101	NE-CR18B	NE-3405	NE-3405LM
Cracks and Joints For all cracks > 1/4" wide, remove old crack sealer and all foreign material by sand and air blasting. Full depth edge surfaces need to be dry and clean.			X	X
Intersplice Longitudinal Joint of Mainline to Bituminous Shoulder.		X		

Viscosity	NE-101	NE-CR18B	NE-3405	NE-3405LM
Low = Thin, High = Thick	Medium	Medium - High	Medium - Low	Low

Asphalt & Concrete Sealing Calendar Work Schedule

The optimal window for installing hot-pour sealant is limited and often debated. Nebraska is a wet/freeze region that receives snow during the winter months, making construction and maintenance activities difficult during that time. However, applying hot-pour sealant in colder weather can be beneficial for the pavement. As temperatures drop, the pavement contracts, causing joints and cracks to widen. This creates ideal conditions for sealant application, as the wider openings allow for better penetration. In contrast, warmer temperatures cause pavement expansion, which narrows the joints and cracks. Installing sealant under these conditions often requires excessive routing or saw cutting to achieve adequate joint width. For this reason, hot-pour sealant is typically applied in early spring or late fall, when cooler temperatures promote pavement shrinkage.

The following is a list of pavement maintenance treatments and the time of year in which the work is typically performed. Use this information as a guide to select the proper letting date for a project.

Concrete Pavement

Type of Work	Concrete Pavement Temperature
Sealing Joints & Cracks	40˚F – 100˚F

Table 8.3-2 – Concrete Pavement Temperatures for Hot-Pour Sealant

Hot-pour sealant should not be applied to concrete pavement when the pavement temperature is below 40°F, as low temperatures can hinder proper adhesion. Similarly, sealing should be avoided when pavement temperatures exceed 100°F, as joint openings may close, reducing the effectiveness of the seal. In Nebraska, the recommended timeframes for hot-pour sealing concrete pavement are from April 1st to June 30th and from September 1st to November 30th, when temperatures are more favorable for proper sealant performance.

Bituminous Pavement

Type of Work	Bituminous Pavement Temperature
Joint Sealing, Asphalt to Concrete	50˚F – 120˚F
*Crack Sealing Bituminous Surface	50˚F – 120˚F

Table 8.3-3 – Bituminous Pavement Temperatures for Hot-Pour Sealant

* On certain projects, both fog sealing and crack sealing of bituminous surfaces are specified. For these cases, a late spring or early summer letting is recommended to allow the fog seal to be completed between June 1st and September 1st, followed by crack sealing. See Chapter 6 of this guide for Fog Seal.

Chapter 9: Material Costs and Quantity Estimates

9.1 Introduction

In pavement design, selecting an appropriate project strategy is strongly influenced by material and construction costs. The initial phase involves a thorough analysis of available materials, construction methods, and their associated expenses to identify the most cost-effective and durable approach. By accurately estimating material quantities and understanding the costs of various construction techniques, designers can develop strategies that balance performance with budget constraints.

Quantity pay items, which reflect the actual units of work performed, offer a clear framework for tracking costs and ensuring fair compensation for contractors based on materials and labor used. Selecting the appropriate pay items for each project should align with the construction practices outlined in the Standard Specifications and any applicable Special Provisions.

This chapter presents cost estimates for various materials and construction practices used by NDOT to evaluate different pavement strategies. Each section indicates the date of the most recent cost update, based on the average unit prices of relevant pay items from let construction projects. For the most up-to-date information on quantities and cost estimates, please contact the Pavement Design Section.

Disclaimer

The pay items, units, and quantities provided herein are intended solely as general guidelines and rule-of-thumb estimates for planning, budgeting, and reference purposes.

Actual quantities, measurements, and pay items may vary based on project conditions, design requirements, field verification, agency standards, specifications, and final approved plans.

9.2 Estimated Cost Per Mile

Individual Pay Items

Asphalt – July 2025

Description		Cost/Mile
Asphaltic Concrete, Type SPR	Depth 2” x 24’ Wide Depth 3” x 24’ Wide Depth 4” x 24’ Wide	$198.2 K $295.3 K $389.5 K
Asphaltic Concrete, Type SRM	Depth 2” x 24’ Wide Depth 3” x 24’ Wide Depth 4” x 24’ Wide	$176.0 K $262.0 K $345.1 K
Asphaltic Concrete, Type SLX	Depth 2” x 24’ Wide Depth 3” x 24’ Wide Depth 4” x 24’ Wide	$200.2 K $298.4 K $393.7 K
Asphaltic Concrete, Type SPS	Depth 1” x 8’ Wide Depth 2” x 8’ Wide Depth 3” x 8’ Wide	$55.2 K $81.4 K $104.5 K

Notes:

(1) Includes the Pay Items Asphaltic Concrete Type __, Hydrated Lime/WMA, RAP Incentive, Performance Graded Binder, Tack Coat, Asphalt Pavement Smoothness Testing I/D, and Superpave Quality and Asphalt Smoothness I/D Payment.

(2) All Cost Estimates Above Includes 1.34 for Engineering & Construction, Mobilization, Traffic Control, Etc.

Concrete – July 2025

Description		Cost/Mile
Concrete Pavement	Depth 8” x 8’ Wide	$672.1 K
	Depth 8” x 24’ Wide	$2.016 M
	Depth 9” x 8’ Wide	$948.5 K
	Depth 9” x 24’ Wide	$2.846 M
	Depth 10” x 8’ Wide	$573.8 K
	Depth 10” x 24’ Wide	$1.721 M
	Depth 12” x 8’ Wide	$619.7 K
	Depth 12” x 24’ Wide	$1.859 M
Doweled Concrete Pavement	Depth 8” x 24’ Wide	$1.468 M
	Depth 9” x 24’ Wide	$1.998 M
	Depth 10” x 24’ Wide	$1.652 M
	Depth 12” x 24’ Wide	$1.968 M
	Depth 15” x 24’ Wide	$1.833 M
White Topping	Depth 5” x 24’ Wide	$600.7 K
	Depth 6” x 24’ Wide	$776.2 K

Notes:

(1) White Topping Includes the Pay Items Placement of Concrete Pavement, Concrete Pavement Class 47B-3500, Water, Earth Shoulder Construction, and Cold Milling, Class 3.

(2) All Cost Estimates Above Includes 1.34 for Engineering & Construction, Mobilization, Traffic Control, Etc.

Subgrade/Stabilizations/Recycling – July 2025

Description		Cost/Mile
Subgrade Preparation	24' Wide	$32.1 K
4" Foundation Course	24' Wide	$144.0 K
Stabilized Bituminous	Cement – 10" Deep x 24' Wide Fly Ash – 10" Deep x 24' Wide	$227.1 K $238.3 K
Stabilized Subgrade	Cement – 8" Deep x 30' Wide Fly Ash – 8" Deep x 30' Wide Lime – 8" Deep x 30' Wide	$177.3 K $240.7 K $210.9 K
Hydrated Lime Slurry Stabilization	4" Deep x 24' Wide	$139.6 K
Hot In-Place Recycling	2" Deep x 24' Wide 2" Deep x 28' Wide	$133.4 K $137.2 K
Cold In-Place Recycling with Foam Asphalt	4" Depth x 24' Wide – Paver Laid	$136.2 K

Notes:

(1) *Stabilized Bituminous Includes Pay Items Cement/Fly Ash Stabilized Bituminous, Cement/Fly Ash, Water, and Fog Seal.*

(2) *Stabilized Subgrade Includes Pay Items Stabilized Subgrade Type Cement/Fly Ash/Lime, Cement/Fly Ash/Hydrated Lime, Water, and Fog Seal.*

(3) *Hydrated Lime Slurry Stabilization Includes Pay Items HLSS, Emulsified Asphalt for HLSS, Fog Seal, Hydrated Lime for HLSS.*

(4) *Hot In-Place Recycling Includes Pay Items Hot In-Place Recycling and Emulsified Asphalt for Hot In-Place Recycling.*

(5) *Cold In-Place Recycling Includes Pay Items Cold In-Place Recycling (with Foam Asphalt), Performance Graded Binder (52-34), and Fog Seal.*

(6) *All Cost Estimates Above Includes 1.34 for Engineering & Construction, Mobilization, Traffic Control, Etc.*

Milling – July 2025

Description		Cost/Mile
Cold Milling Class 1	24’ Wide	$11.6 K
Cold Milling Class 2	24’ Wide	$12.0 K
Cold Milling Class 3	Depth 2” x 8’ Wide Depth 2” x 24’ Wide Depth 3” x 24’ Wide Depth 4” x 24’ Wide	$8.8 K $21.6 K $30.6 K $39.6 K
Cold Milling Class 4	Depth 2” x 24’ Wide Depth 3” x 24’ Wide Depth 4” x 24’ Wide	$25.8 K $36.6 K $47.4 K
Trenched Widening 1’		$9.7 K
Trenched Widening 3’		$15.2 K

Notes:

(1) Trenched Widening 1’ and Trenched Widening 3’ Includes Both Sides of the Pavement.

(2) All Cost Estimates Above Includes 1.34 for Engineering & Construction, Mobilization, Traffic Control, Etc.

Miscellaneous – July 2025

Description		Cost/Mile
Penetrating Concrete Sealer	24’ Wide	$36.5 K
Diamond Grinding	14’ Wide	$46.2 K
Dowel Bar Retrofit	12’ Wide (6 Dowel Bars/Lane) 24’ Wide (6 Dowel Bars/Lane)	$150.4 K $300.8 K
Joint Seal	24’ Wide	$108.7 K
Crack Sealing Bituminous Surfacing		
Armor Coat	24’ Wide	$59.2 K
Chip Seal with Light Weight Aggregate	24’ Wide	$66.1 K
Fog Seal	24’ Wide 16’ Wide	$9.8 K $6.5 K
Microsurfacing	24’ Wide	$52.5 K
High Friction Surface Treatment	(1-Layer) 24’ Wide (2-Layer) 24’ Wide	$566.0 K $1.264 M
Fabric Reinforcement Crack Repair	24’ Wide	$566.0 K
Earth Shoulder Construction		$20.5 K
Earth Shoulder Restoration		$11.1 K

Notes:

(1) *Penetrating Concrete Sealer Application Rate = 300 Gal/ft^2*

(2) *Joint Seal Includes Transverse Joints every 16.5’ and Longitudinal Joints at Centerline and both Shoulder Joints.*

(3) *Fog Seal Application Rate = 0.12 Gal/yd^2*

(4) *Crack Sealing Bituminous Surfacing*

(5) *Microsurfacing Includes Pay Items Microsurfacing, Emulsified Asphalt for Microsurfacing, Aggregate for Microsurfacing, and Mineral Filler for Microsurfacing. Microsurfacing Thickness is assumed to be ¼”.*

(6) *Chip Seal with Lightweight Aggregate Includes Pay Items Chip Seal Aggregate and Chip Seal Emulsified Asphalt.*

(7) *Dowel Bar Retrofit Includes 6 Dowel Bars per Transverse Joint at 16.5’ Joint Spacing.*

(8) *Armor Coat Includes Pay Items Armor Coat Aggregate and Armor Coat Emulsified Asphalt.*

(9) *Earth Shoulder Construction and Earth Shoulder Restoration Includes Both Sides of the Pavement.*

(10) *All Cost Estimates Above Includes 1.34 for Engineering & Construction, Mobilization, Traffic Control, Etc.*

Interstate – July 2025

Description	Cost/Mile
Microsurfacing 28' Wide	$78.3 K
Penetrating Concrete Sealer 27' Wide	$47.8 K
High Friction Surface Treatment (1-Layer) 27' Wide	$1.252 M
Joint Seal – Transverse Cracks – Assuming 320 Joints per Mile x 38.5' Wide – Longitudinal Cracks – Assuming 2 Joints x 5,280' Long – Total Length = (320 x 38.5') + (2 x 5,280') = 22,880' (One Direction)	$58.3 K
Crack Sealing Bituminous Surfacing – Transverse Cracks – Assuming 160 Cracks per Mile x 38.5' Wide – Longitudinal Cracks – Assuming 3,000' Long – Total Length = (160 x 38.5') + (3,000') = 9,160' (One Direction)	$24.5 K
Perform PCC Repair. Perform Diamond Grinding. Joint Seal. 27' Wide Apply Penetrating Concrete Sealer.	$271.2 K
Interstate Mainline – Perform PCC Repairs as needed. Mill 2" of existing asphalt by Cold Milling, Class 3 x 28' wide. Place 2" of SLX across 28'. (Used Performance Graded Binder 58E-34) Interstate Shoulder – Perform PCC Repairs as needed. Mill 2" of existing asphalt by Cold Milling, Class 3 x 10' wide. Place 2" of SPS across 10'. (Used Performance Graded Binder 58S-34)	$421.9 K
Interstate Mainline – Perform PCC Repairs as needed. Mill 4" of existing asphalt by Cold Milling, Class 3 x 28' wide. Place 4" of SLX. (Used Performance Graded Binder 58E-34) Interstate Shoulder – Perform PCC Repairs as needed. Mill 2" of existing asphalt by Cold Milling, Class 3 x 10' wide. Place 2" of SPS. (Used Performance Graded Binder 58S-34)	$673.7 K
Interstate – New Build (4-Lane) Mainline & Inside Shoulder – Build 13" Doweled Concrete Pavement on 5" Foundation Course on Stabilized Subgrade. Outside Shoulder – Build 13" tapered to 10" Concrete Pavement on 5" Foundation Course on Stabilized Subgrade.	$3.146 M
Interstate – New Build (6-Lane) Mainline & Inside Shoulder – Build 13" Doweled Concrete Pavement on 5" Foundation Course on Stabilized Subgrade. Outside Shoulder – Build 13" tapered to 10" Concrete Pavement on 5" Foundation Course on Stabilized Subgrade.	$4.095 M

Miscellaneous Project Scenarios – July 2025

Description	Cost/Mile
Perform Hydrated Lime Slurry Stabilization to a depth of 4” x 24’ wide. Place 3” of SPR across 24’ top.	$503.6 K
Trench Widen 2’ Left & Right. Perform Hydrated Lime Slurry Stabilization to a depth of 4” x 24’ wide. Place 3” SPR across 28’ top.	$582.0 K
Perform Hot In-Place Recycling to a depth of 2” from existing surface x 24’ wide. Place Armor Coat.	$192.6 K
Perform Hot In-Place Recycling to a depth of 2” from existing surface x 24’ wide. Place 1” of SLX.	$269.5 K
Mill 1” of existing asphalt x 24’ wide. Perform Hot In-Place Recycling to a depth of 2” from milled surface. Place 1.5” of SPR.	$315.3 K
Mill 1” of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 4” of SPR.	$407.3 K
Mill 1.5” of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 2” of SPR.	$222.0 K
Mill 2” of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 2” of SPR. Place 4” of SPR.	$227.3 K $440.5 K
Mill 4” of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 4” of SPR. Place 6” of SPR.	$420.2 K $600.8 K
Mill 4” of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 1” of SLX on 4” of SRM across 24’.	$561.4 K
Mill 4" of existing asphalt by Cold Milling, Class 3 x 24’ wide. Place 2” of SPR on 4” of SRM across 24’.	$674.8 K
Place 3” of SPR on 1” of LC across 24’.	$426.6 K
Trench Widen 2’ Left & Right from existing surface to a depth of 7”. Mill 4” of existing asphalt by Cold Milling Class 3 x 24’ wide. Fill trenches and place 4” of SRM across 28’ top. Place 1” of SLX across 28’ top.	$696.4 K
Perform Fly Ash Stabilized Bituminous to a depth of 10” x 24’ wide. Place 4” of SPR.	$694.3 K
Perform Cement Stabilized Bituminous to a depth of 10” x 24’ wide. Place 4” of SPR.	$691.9 K
Perform PCC Repair. Perform Dowel Bar Retrofit and Diamond Grinding in the Driving Lane and Joint Seal (14’ Wide).	$275.4 K
Build __ of Asphaltic Concrete, Type SPR x 24’ wide on Stabilized Subgrade, Type Lime. 6" Depth 8" Depth 9" Depth 10" Depth	 $743.5 K $913.7 K $988.2 K $1.074 M

Build __ x 24’ Wide Doweled Concrete Pavement with 4.5’ wide Inside Shoulder on 4” Foundation Course on Subgrade Preparation. 8” with 8” Thick Concrete Pavement Outside Shoulder (8’ wide) 9” with 9” Thick Concrete Pavement Outside Shoulder (8’ wide) 10” with 10” tapered to 8” Thick Concrete Pavement Outside Shoulder (10’ wide) 12” with 12” tapered to 10” Thick Concrete Pavement Outside Shoulder (10’ wide) 14” with 14” tapered to 11” Thick Concrete Pavement Outside Shoulder (12’ wide)	 $2.725 M $3.262 M $2.170 M $3.516 M $4.169 M
Expressway Place 1” x 37’ SLX on PCC Place 2” x 27’ SLX with 2” x 8’ SPS on PCC Place 3” x 27’ SLX with 3” x 8’ SPS on PCC Place 4” x 27’ SLX with 4” x 8’ SPS on PCC	 $230K $385K $559K $728K

9.3 Estimated Concrete Repair Cost

ESTIMATED COST FOR CONCRETE REHABILITATION
(NO E&C INCLUDED, November 2022)

CONCRETE PAVEMENT (TYPES A, B AND C) AND JOINT REPAIR, FULL DEPTH
(2 Lanes per Mile, yd^3)

No.	Description	Cost/Mile
1	Existing (Doweled or Plain) Concrete built After 2002 (No ASR)	$10,000 (No Overlay)
2	Existing (Doweled or Plain) Concrete built Before 2002 (No ASR)	$20,000 (No Overlay)
3	Existing (Doweled or Plain) Concrete w/very lite ASR but no spalling	$30,000 (No Overlay)
4	Existing (Doweled or Plain) Concrete w/lite ASR but limited spalling	$70,000/lane w/ASR (No Overlay)
5	Existing (Doweled or Plain) Concrete w/heavy ASR	$150,000/lane (No Overlay)
6	Existing Asphalt Overlay w/no blow ups	$25,000
7	Existing Asphalt Overlay w/blow ups	$50,000
8	Existing Asphalt Overlay w/fines and blow ups	$75,000
9	Existing Asphalt Overlay on the Interstate	$50,000 - $100,000 (Based on PathWeb Condition)

DIAMOND GRINDING AND TEXTURING CONCRETE PAVEMENT (yd^2)

No.	Description	Cost/Mile
1	24’ Concrete Mainline w/Asphalt Shoulders	$55,000 (Feather PCC Outside Shoulder = $58,000)
2	12’ Driving Lane w/Asphalt Shoulders	$28,000 (Feather PCC Outside Shoulder = $58,000)
3	Multilane in Omaha (Night Work)	$30,000/lane *(4 lanes w/PCC or AC IS/OS = $120,000/mile)
4	If previously Diamond Ground w/Concrete Shoulder, add $5,000/mile/needed side for extra feather for positive drainage.	

JOINT SEALING – ASPHALT TO CONCRETE (INTERSPLICE) (Station)

No.	Description	Cost/Mile
1	Exposed Concrete w/Asphalt Shoulders (per 'Splice')	$17,000 *(Exposed Concrete Mainline w/L+R AC Outside Shoulders = $34,000/mile)

SEALING JOINTS (STRAIGHT OR SKEWED) (Linear Foot)

No.	Description	Cost/Mile
1	24' Concrete Mainline w/Asphalt Shoulders (16.5' Joint Spacing)	$40,000
2	24' Concrete Mainline w/8' Asphalt Shoulders 40' wide (16.5' Joint Spacing)	$70,000
3	2-lane Divided 37' wide/side (15'+12'+10') x 2 = 74' wide (16.5' Joint Spacing)	$180,000
4	6-lane Divided 50' wide/side (5 – 10' panels) x 2 = 100' wide (16.5' Joint Spacing)	$300,000 *(10 miles of 6 lane = $3,000,000)

SEALING CRACKS (EXPOSED CONCRETE PAVEMENT) (Linear Foot)

No.	Description	Cost/Mile
1	1930's to 1980	$3,500 (If several cracks are noted, use $7,000)
2	1980's to 2000	$2,500 (If several cracks are noted, use $5,000)
3	2000's to 2022	$1,500 (If several cracks are noted, use $2,500)

CRACK SEALING BITUMINOUS SURFACING
(DUAL LANE INTERSTATE & SOME EXPRESSWAY) (Linear Foot)

No.	Description	Cost/Mile
1	4-lane (like I-80) for underlying Reinforced Concrete Pavement (46.5' Joint Spacing)	$30,000
2	4-lane (like I-80) for underlying Plain Concrete (16.5' Joint Spacing)	$60,000
3	Omaha – Use $17,000/traveled lane minus shoulders. Note that if the Asphaltic Concrete is not up to the Jersey Barrier or Face of Curb/in the gutter pan (I-480 at NB OFF Ramp at Martha Street.)	

EXAMPLE: <u>Concrete Repair, Grind and Seal</u>

$ 75,000	=	Plain Concrete Pavement Repair
$ 30,000	=	Grinding Driving Lane/1' Passing Lane/1'Outside Shoulder
$ 1,815	=	Sealing Cracks
<u>$ 84,000</u>	=	Sealing 2 Longitudinal Joints/Skewed Transverse Joints/3' & 10' Shoulders
$ 190,815	=	Total Cost/Mile

E & C = Engineering & Contingencies
FCR = Foundation Course Replacement

CONCRETE CURB REPAIR – $50/Lin. Ft.

EXAMPLE: <u>CN 42921 Wood River to Platte River Crack Seal I-80 RP 299.25 to 310.88</u>

61,406ft	=	Longitudinal Joint Length, Original RCP under AC (11.63 miles x 5,280ft/mile)
1,320 Joints	=	Joint Spacings under AC (61,406ft / 46.5ft per joint)
50,160ft	=	Transverse Joint Length (1,320 Joints x 38ft wide)
122,812ft	=	Longitudinal Joint at Centerline and Outside Shoulder (61,406ft x 2 joints)
245,624ft	=	Total Longitudinal Joint Length (122,812ft x 2 (Both Sides of the Interstate))
100,320ft	=	Total Transverse Joint Length (50,160ft x 2 (Both Sides of the Interstate))
345,944ft	=	Total Joint Length (245,624ft + 100,320ft)
207,566ft	=	Total Joint Length needing Sealed (345,944ft x 0.60). Assuming 60%
$31,233/mile	=	Total Cost/Mile for Crack Sealing ((207,566ft x $1.75/foot) / 11.63 miles)

9.4 Quantities Worksheet

Estimating Quantities

Reference to 2017 NDOT Standard Specifications for Highway Construction

General Information:

- Items are listed in alphabetical order.
- Conversion factors are in English units.
- RAP is an acronym for Reclaimed Asphalt Pavement. Also known as Bituminous Millings.
- Weight of RAP = 144 lbs/ft^3
- One gallon of emulsified asphalt or water = 8.333 lbs.
- Beveled edges in asphalt roadways are included in the asphalt tons. In concrete pavements, the bevel footprint is measured and added to the surface top square yards.

Measuring Units:

Length

- ft = Foot
- lf = Linear Foot
- Sta = Station (1 Station = 100 Feet)
- Mile = Mile (1 Mile = 5,280 Feet)

Area

- yd^2 = Square Yard
- ft^2 = Square Foot
- Acre = Acre (1 Acre = 43,560 Square Feet or 4,840 Square Yards)
- lbs/yd^2 = Pounds per Square Yard
- ft^2/Gal = Square Feet per Gallon

Volume

- Gal = Gallons
- MGal = Mega Gallons (1 MGal = 1,000 Gallons)
- yd^3 = Cubic Yard
- ft^3 = Cubic Foot
- yd^3/Sta = Cubic Yards per Station
- Gal/Sta = Gallons per Station
- Gal/yd^2 = Gallons per Square Yard
- MGal/Sta = Mega Gallons per Station

Weight/Density

- lbs = Pounds
- lbs/yd^2 = Pounds per Square Yard
- lbs/ft^3 = Pounds per Cubic Foot
- Ton = Ton (1 Ton = 2,000 Pounds)
- Ton/Sta = Tons per Station
- Ton/Acre = Tons per Acre (1 Acre = 43,560 Square Feet)

Asphaltic Concrete Projects

Add the following Equipment Rental Pay Items and Hours:

Rental of Loader, Fully Operated	=	15 Hours
Rental of Motor Grader, Fully Operated	=	15 Hours
Rental of Dump Truck, Fully Operated	=	15 Hours
Rental of Skid Loader, Fully Operated	=	15 Hours

Armor Coat – Section 515		
Pay Item	**Unit**	**Design/Estimation Basis**
Armor Coat Aggregate	yd^3	23 lbs/yd^2 Conversion Factor 1.3 Tons = 1 yd^3
Armor Coat Emulsified Asphalt	Gal	0.34 Gal/yd^2

Asphaltic Concrete – Section 503, Section 1028 & Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Asphaltic Concrete, Type "___"	Ton	Use 148 lbs/ft^3 for all Asphaltic Concrete Mix Types. Include Material Required for Beveled Edge.
Hydrated Lime/WMA	Each	See Table on Page 183
RAP Incentive Payment	Each	Asphaltic Concrete, Type "___" x 1.7

Asphaltic Concrete Curb – Section 505		
Pay Item	**Unit**	**Design/Estimation Basis**
Constructing Asphaltic Concrete Curb	lf	Factor for 3" Curb = 1.35 Ton/Sta Factor for 4" Curb = 2.00 Ton/Sta Factor for 6" Curb = 2.10 Ton/Sta Factor for Tack Coat = 1.00 Gal/Sta

Asphaltic Concrete for Patching – Section 516		
Pay Item	**Unit**	**Design/Estimation Basis**
Asphaltic Concrete for Patching, Type "___"	Ton	

Asphalt Pavement Smoothness Testing I/D – Section 502 & Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Asphalt Pavement Smoothness Testing I/D	Mile	
Bituminous Patching of Concrete Pavement – Section 520		
Pay Item	**Unit**	**Design/Estimation Basis**
Bituminous Patching	Ton	

Bituminous Sand Base Course – Section 509		
Pay Item	**Unit**	**Design/Estimation Basis**
Bituminous Sand Base Course	Sta	
Bituminous Sand Base Course Asphaltic Oil	Gal	1,000 Gal/Sta for (5" x 24')
Bituminous Sand Base Course Emulsified Asphalt	Gal	1,200 Gal/Sta for (5" x 24') (6% Residual)
Mineral Filler for Bituminous Sand Base Course	yd^3	*10 yd^3/Sta for (5" x 24')
Mineral Aggregate	yd^3	Do not use for estimate
Water	MGal	1 MGal/Sta
Fog Seal	Gal	0.15 Gal/yd^2

**Quantity of Mineral Filler will vary depending on type of soil.*

Bituminous Surface Course – Section 512		
Pay Item	**Unit**	**Design/Estimation Basis**
Bituminous Surface Course	yd^2	
Fog Seal	Gallon	0.6 Gal/yd^2

Bridge Items		
Pay Item	**Unit**	**Design/Estimation Basis**
Placement of Asphaltic Concrete for Bridges	yd^2 or ft^2	
Saw and Seal Joint	lf	See Bridge Plans.
Bridge Preparation	yd^2 or ft^2	Use for Type 1 or Type 2 Membranes
Preformed Waterproofing Membrane, Type 1	yd^2	
Preformed Waterproofing Membrane, Type 2	yd^2	
Preformed Waterproofing Membrane, Type 3	yd^2	Use for New Bridges
Hot Liquid – Applied Membrane Waterproofing	yd^2	
Cold Liquid – Applied Membrane Waterproofing	yd^2	Item No Longer Available

Note: If the Saw and Seal Joint Pay Item is not shown in plans, add this Pay Item if bridge is adjacent to Concrete Pavement or Composite Pavement.

Calcium Chloride, Applied – Section 309		
Pay Item	**Unit**	**Design/Estimation Basis**
Calcium Chloride, Applied	Ton	3 lbs/yd^2

Cement Stabilized Bituminous – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Cement Stabilized Bituminous	Sta	
Cement	Ton	5% Weight of RAP
Water for Cement Stabilization	MGal	5% Weight of RAP & Cement (Convert to MGal)
Cold Milling, Class 2	Sta	Use if required in the Pavement Determination
Fog Seal	Gal	*0.24 Gal/yd^2

**One application after the "CSB". Second application after the Cold Milling, Class 2, if required.*

Chip Seal – Section 515		
Pay Item	**Unit**	**Design/Estimation Basis**
Chip Seal Aggregate	yd^3	25 lbs/yd^2 Normal Aggregate Weight 1.4 Tons = 1 yd^3 12 lbs/yd^2 Light Weight Aggregate 1,500 lbs/yd^3 or 0.75 Tons = 1 yd^3
Chip Seal Emulsified Asphalt	Gal	0.36 Gal/yd^2

Cold In-Place Recycling (w/Foamed Asphalt) – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Cold In-Place Recycling with Foamed Asphalt	Sta	
Preformed Graded Binder (58–28)	Ton	2% RAP (4" x 24' = 1.15 Ton/Sta) (4" x 28' = 1.34 Ton/Sta)
Fog Seal	Gal	0.10 Gal/yd^2

Cold Milling – Section 510		
Pay Item	**Unit**	**Design/Estimation Basis**
Cold Milling, Class "____"	Sta or yd^2	

Concrete Pavement Repair, Flexible Polymer Modified		
Pay Item	**Unit**	**Design/Estimation Basis**
Concrete Pavement Repair, Flexible Polymer Modified	yd^2	

Note: Special Provision describes depth of repair.
Preparation of Concrete, Primer, Bulking Aggregate, and Surfacing Aggregate are subsidiary.

Concrete Sealer – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Penetrating Concrete Sealer	Gal	300 ft^2/Gal at 100% Silane 150 ft^2/Gal at 40% Silane

Note: Due to BABA requirements, during pre-letting estimate, assume that the total material costs are 35% of the total contract cost. Multiply that number by 5% to get the threshold we cannot exceed for non-compliant BABA material.

Concrete Surface Milling – Section 510		
Pay Item	**Unit**	**Design/Estimation Basis**
Concrete Surface Milling	Sta or yd^2	

Cracking & Seating Concrete Pavement – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Cracking & Seating	yd^2	

Diamond Grinding and Texturing Pavement – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Diamond Grinding and Texturing Pavement	yd^2	

Earth Shoulder Construction – Section 304		
Pay Item	**Unit**	**Design/Estimation Basis**
Earth Shoulder Construction	Sta	Shoulders are Measured Separately
Water	MGal	0.25 MGal/Sta

Earth Shoulder Restoration – Section 304		
Pay Item	**Unit**	**Design/Estimation Basis**
Earth Shoulder Restoration	Sta	Shoulders are Measured Separately
Seeding, Type B	Acre	Use 8' Wide x Length
Mulch (Hay or Straw)	Ton	2.25 Ton/Acre

Note: Use this item unless the Pavement Determination specifies differently or if Earth Shoulder Construction is requested in the Plan-In-Hand.
Use this item when the project has Trenched Widening 1' and 1" grade raise or less.
Use this item when the project has Trenched Widening 3' and 2" grade raise or less.

Fabric Reinforcement Crack Repair – Section 518		
Pay Item	**Unit**	**Design/Estimation Basis**
Fabric Reinforcement Crack Repair	lf or yd^2	

Note: See Special Provision for type of Tack Coat specified and application rate.

Fly Ash Stabilized Bituminous – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Fly Ash Stabilized Bituminous	Sta	
Fly Ash	Ton	12% Weight of RAP
Water for Fly Ash Stabilization	MGal	5% Weight of RAP & Fly Ash (Convert to MGal)
Cold Milling, Class 2	Sta	Use if required in the Pavement Determination
Fog Seal	Gal	*0.24 Gal/yd^2

**One application after the "FASB". Second application after the Cold Milling, Class 2, if required.*

Fog Seal – Section 513		
Pay Item	**Unit**	**Design/Estimation Basis**
Fog Seal	Gal	0.12 Gal/yd^2 Mainline & Shoulder
		0.16 Gal/yd^2 Open Graded Friction Course
		0.07 Gal/yd^2 Milled Surface of Asphaltic Concrete
		0.10 Gal/yd^2 Milled Surface of Bituminous Sand

Note: Use CSS-1 & CSS-1H.

Foundation Course – Section 307		
Pay Item	**Unit**	**Design/Estimation Basis**
Foundation Course "____"	yd^2	Use this item for estimates.
Bituminous Foundation Course "____"	yd^2	123 lbs/ft^3 or 1.66 Ton/yd^3 In-place Weight for 4" + 1/4" Trimming
		1.43 Ton/yd^3 Stockpiled Bituminous Millings
Crushed Concrete Foundation Course "____"	yd^2	0.190 Ton/yd^2 In-place Weight for 4" + 1/4" Trimming
		1.35 Ton/yd^3 Stockpiled Crushed Concrete
		yd^3 x 1.94 Ton/yd^3 x 90% (10% loss) Concrete Pavement In-place (Tons of Crushed Concrete Available)
Aggregate Foundation Course "D" "____"	yd^2	
Aggregate Foundation Course "____"	yd^2 or Ton	(yd^2 x 0.2222 Ton/yd^2) = Tons In-place Weight for 4" + 1/4" Trimming
Crushed Rock Foundation Course "____"	yd^2	Separation Layer is Subsidiary

Note: Foundation Course calculated as total pavement footprint including bevel. Water calculated for pavement footprint plus 3' beyond. Plans show Foundation Course 3' beyond pavement footprint.

Gravel Embedment – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Gravel Embedment	Sta	
Gravel	yd^3	Designer's Item

Note: Design is usually 2" gravel embedded in the upper 4" & capped with 1" Gravel Surface Course.

Granular Subdrains – Section 915		
Pay Item	**Unit**	**Design/Estimation Basis**
Granular Subdrains	Each	

Note: Add patching tons for Retrofit. (0.5 Ton Each).

High Friction Surface Treatment – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
High Friction Surface Treatment (1-Layer)	yd^2	
High Friction Surface Treatment (2-Layer)	yd^2	

Hot In-Place Recycling – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Hot In-Place Recycling	Sta	
Emulsified Asphalt for Hot In-Place Recycling	Gal	1.0% of RAP – (2" x 24' = 69 Gal/Sta) – (2" x 28' = 81 Gal/Sta)

Note: If the width is anything other than 24' wide, then the Special Provision Tape needs to be edited.

Hydrated Lime Slurry Stabilization – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Hydrated Lime Slurry Stabilization	Sta	
Hydrated Lime	Ton	1.50% Weight of RAP (4” x 24’ = 0.9 Ton/Sta) (5” x 24’ = 1.1 Ton/Sta)
Emulsified Asphalt for HLSS	Gal	1.75% Weight of RAP & Lime (4” x 24’ = 245 Gal/Sta) (5” x 24’ = 307 Gal/Sta)
Fog Seal	Gal	0.10 Gal/yd^2

Note: Growth Factor approximately ¾” for a depth of 3” to 5”.
Growth Factor of 1” for a depth of 6”.

Intersections and Driveways – Section 302 & Section 503		
Pay Item	**Unit**	**Design/Estimation Basis**
Preparation of Intersections and Driveways	yd^2	
Placement of Asphaltic Concrete for Intersections and Driveways	yd^2	

Note: Asphaltic Concrete paid for by Roadway Tonnage.

Joint Sealing Asphalt to Concrete – Section 508		
Pay Item	**Unit**	**Design/Estimation Basis**
Joint Sealing – Asphalt to Concrete	Sta	Measured Separately (One Side)

Mailbox Turnouts – Section 912 & Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Preparation of Intersections and Driveways	yd^2	
Placement of Intersections and Driveways	yd^2	

Microsurfacing – Section 514		
Pay Item	**Unit**	**Design/Estimation Basis**
Microsurfacing Placement	Sta	
Emulsified Asphalt for Microsurfacing	Gal	12.0% of Total Tons 240 Gallons = 1 Ton
Aggregate for Microsurfacing	Ton	83.8% of Total Tons
Mineral Filler for Microsurfacing	Ton	1.7% of Total Tons

Note: Weight Factor is 6.6 Ton/100 ft^3.
Lift thicknesses are ¼" and calculate rut depth, if applicable.

Milling Concrete for Inlays – Section 510		
Pay Item	**Unit**	**Design/Estimation Basis**
Milling Concrete for Inlays	Each	

Non-Woven Pavement Overlay Fabric – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Non-Woven Pavement Overlay Fabric	yd^2	

Pavement Lifting and Stabilization with Polyurethane Foam – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Pavement Lifting and Stabilization	lbs	500lbs/Panel (lifting less than 3") 1,000lbs/Panel (lifting 3" or more)

Performance Graded Binder (**-**) – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Performance Graded Binder (**-**)	Ton	See Table on Page 182 to Estimate Tons

Perforated Pipe Underdrain – Section 914		
Pay Item	**Unit**	**Design/Estimation Basis**
____ Perforated Pipe Underdrain	lf	
____ Non-Perforated Pipe Underdrain	lf	

Note: Granular Material and Filter Fabric is subsidiary.

Shoulder Subgrade Preparation – Section 302		
Pay Item	**Unit**	**Design/Estimation Basis**
Shoulder Subgrade Preparation	Sta	Measured Separately (One Side)
Water	MGal	0.5 MGal/Sta

Note: Earth Shoulder Construction is subsidiary.

Special Surface Course – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Special Surface Course	yd^2	
Fog Seal	Gal	0.20 Gal/yd^2 Application Rate for Soil 0.30 Gal/yd^2 Application Rate for Surfacing

Note: Use Special Surface Course if placing millings on driveways or under guardrail.
Use 2 Applications for both Soil and Surfacing situations.

Stress Absorbing Fiberglass Layer with Emulsified Asphalt (SAFLEA) – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Stress Absorbing Fiberglass Layer with Emulsified Asphalt	yd^2	
Armor Coat Emulsified Asphalt	Gal	0.44 Gal/yd^2
Armor Coat Aggregate	yd^3	32 lbs/yd^2 Conversion Factor – 1.3 Tons = 1 yd^3

Subgrade Preparation – Section 302		
Pay Item	**Unit**	**Design/Estimation Basis**
Subgrade Preparation	Sta or yd^2	
Water	MGal	1.0 MGal/Sta or 0.003 MGal/yd^2

Note: Subgrade Preparation calculated as total pavement footprint including beveled edge.
Water calculated for pavement footprint plus 3' beyond.
Plans show Subgrade Preparation 3' beyond pavement footprint.

Subgrade Preparation for Widening – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Subgrade Preparation for Widening	Sta	One Side
Water	MGal	0.5 MGal/Sta

Note: Use for Concrete Pavement Widening.

Subgrade Stabilization – Section 303		
Pay Item	**Unit**	**Design/Estimation Basis**
Subgrade Stabilization	Sta or yd^2	One Side
Soil Binder	yd^3	12.5 yd^3/Sta for (6" x 30')
Water	MGal	1 MGal/Sta or 0.003 MGal/yd^2

Note: Subgrade Stabilization calculated as total pavement footprint including beveled edge.
Soil Binder and Water calculated for pavement footprint plus 3' beyond.
Plans show Subgrade Stabilization 3' beyond pavement footprint.

Superpave Quality and Asphalt Smoothness Testing I/D – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Superpave Quality and Asphalt Smoothness Testing I/D	Each	6 x Mainline Asphalt Tonnage

Surfacing – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Surfacing "____"	yd^2	

Note: Contractor's choice for pavement type, Asphaltic Concrete or Portland Cement Concrete.

Surfacing Under Guardrail – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Surfacing Under Guardrail	yd^2	

Note: Contractor's choice for pavement type, Asphaltic Concrete or Portland Cement Concrete.
Subgrade Preparation is subsidiary.

Stabilized Subgrade (8" Depth) – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Stabilized Subgrade Type Cement	yd^2	Use if Granular Content is high for a variety of PIs.
Cement	Ton	27 lbs/yd^2 Cement quantity is **4% of soil tons.
Stabilized Subgrade Type Fly Ash	yd^2	Use if PI of soil is 19 or less.
Fly Ash	Ton	66 lbs/yd^2 Fly Ash quantity is **10% of soil tons.
Stabilized Subgrade Type Lime	yd^2	Use if PI of soil is 20 or more.
Hydrated Lime	Ton	33 lbs/yd^2 Hydrated Lime quantity is **5% of soil tons.
Water	MGal	1 MGal/Sta or 0.003 $MGal/yd^2$

Note: Stabilized Subgrade Type "____" calculated as total pavement footprint including beveled edge.
Cement, Fly Ash, Hydrated Lime, and Water calculated for pavement footprint plus 3' beyond.
Plans show Stabilized Subgrade Type "____" 3' beyond pavement footprint.

***Soil weight compacted in place, 110 lbs/ft^3*

Tack Coat – Section 504		
Pay Item	**Unit**	**Design/Estimation Basis**
Tack Coat	Gallon	0.17 Gal/yd^2 Factor for Existing Surface
		0.07 Gal/yd^2 Factor for Between Asphalt Lifts
		0.20 Gal/yd^2 Milled Hydrated Lime Stabilized Bituminous
		0.36 Gal/yd^2 Milled Fly Ash Stabilized Bituminous
		0.36 Gal/yd^2 Milled Cement Stabilized Bituminous

Note: Application Rates include applying a second Tack Coat at longitudinal joints.

Temporary Surfacing – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Temporary Surfacing "____"	Station or Square Yard	

Note: Contractor's choice for pavement type, Asphaltic Concrete or Portland Cement Concrete.
Subgrade Preparation, Earth Shoulder Construction, Water Applied, and Removal is subsidiary.

Trench Widening 1' – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Trench Widening 1'	Station	Measured Separately (One Side)

Note: Include Earth Shoulder Construction or Earth Shoulder Restoration.

Trench Widening 3' – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Trench Widening 3'	Station	Measured Separately (One Side)

Note: Include Earth Shoulder Construction or Earth Shoulder Restoration.

Ultra-Thin Bonded Asphalt Wearing Course – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Ultra-Thin Bonded Asphalt Wearing Course	Ton	
Performance Graded Binder (**-**)	Ton	See Table on Page 182 to Estimate Tons

Note: UTBAWC will be an Asphaltic Concrete, Type SLX, SLX Super-Mod, or HPS4 as noted in the Pavement Determination.
Do not pay for Tack Coat.

Widening – Special Provision		
Pay Item	**Unit**	**Design/Estimation Basis**
Widening	Station	Measured Separately (One Side)

Performance Graded Binder Table

Asphaltic Concrete Mix Type	RAP Content (Min % – Max %) Section 1028.02	Performance Graded Binder Type	With RAP	Without RAP (Virgin Mix)
LC	0% – 25%	58E-34	5.8%	6.3%
LC-S	0%	58E-34	–	6.5%
SLX	20% – 35%	58V-34	4.2%	5.8%*
		58E-34	4.2%	5.8%*
SLX Super-Mod	0% – 35%	58E-34	4.2%	5.8%
SPR	0% – 45%	58H-34	3.4%	5.4%
		58V-34	3.4%	5.4%
SPR35	0% – 35%	58V-34	3.9%	5.9%
SPS	0% – 50%	58S-34	3.2%	5.4%
SRM	35% – 50%	58H-34	3.0%	–
HPS4	0%	58E-34	–	7.2%

Notes:

(1) Add 15% to Asphalt Tons applied to the Top Lift of Mainline Asphalt for Slope and Profile Correction.

(2) "–" indicates that the percent of Performance Graded Binder specified for the applicable Asphaltic Concrete Mix Type and condition is not applicable.

(*) Asphaltic Concrete, Type SLX may have a minimum RAP content of 0% if permitted by Special Provision for projects where no RAP source is available.

Hydrated Lime/Warm Mix Asphalt

Asphaltic Concrete Type	"Hydrated Lime/WMA" Pay Item is "Each" Multiply Tons of Asphalt by:
LC	1
LC-S	1
SLX	1
SLX Super-Mod	1
SPR	1
SPR35	1
SPS	N/A
SPH	1
HPS4	1

Example:

If there are 10,534 Tons of Asphaltic Concrete, Type SPR, there will be 10,534 Each for the Pay Item "Hydrated Lime/WMA".

Asphaltic Concrete, Type SPS will not be included with the Pay Item "Hydrated Lime/WMA".

Appendices

List of Appendices

A.1 Abbreviations and Definitions

Abbreviations

AC	–	Armor Coat / Asphaltic Concrete
ACSC	–	Asphaltic Concrete Surface Course
ADT	–	Average Daily Traffic
ADTT	–	Average Daily Truck Traffic
ASR	–	Alkali-Silica Reaction
BSBC	–	Bituminous Sand Base Course / Bituminous Stabilized Base Course
BM	–	Bituminous Material
BMSC	–	Bituminous Material Surface Course
BR	–	Bridge
CAA	–	Coarse Aggregate Angularity
CIR	–	Cold In-Place Recycling
CONC	–	Concrete Pavement
CRBC	–	Crushed Rock Base Course
CRCP	–	Continuously Reinforced Concrete Pavement
CSS	–	Cationic Slow Set
ESAL	–	Equivalent Single Axle Load
FAA	–	Fine Aggregate Angularity
FC	–	Foundation Course
FWD	–	Falling Weight Deflectometer
GR	–	Grading
G.R.	–	Guard Rail
HIR	–	Hot In-Place Recycling
HLSS	–	Hydrated Lime Slurry Stabilization
HMA	–	Hot Mix Asphalt
JRCP	–	Jointed Reinforced Concrete Pavement
JPCP	–	Jointed Plain Concrete Pavement
MR	–	Resilient Modulus
PC	–	Prime Coat
PCC	–	Portland Cement Concrete

9”-7”-9” Concrete	–	Parabolically Crowned Concrete with 7” thickness at center and 9” thickness at edge
PDM	–	Pavement Design Manual
RAP	–	Reclaimed Asphalt Pavement (Millings)
RAS	–	Recycled Asphalt Shingles
RDM	–	Roadway Design Manual
SABC	–	Soil Aggregate Base Course
SSBC	–	Stabilized Sand Base Course OR Stabilized Soil Base Course
SSHC	–	State Specification for Highway Construction (2017 latest version)
TSB	–	Tar Stabilized Base
UTBAWC	–	Ultra-Thin Bonded Asphalt Wearing Course

Definitions

Subgrade Preparation – Topsoil removed and top 6” of soil compacted to Optimal Moisture and Stiffness.

Stabilized Subgrade – Lime, Fly Ash, Cement, Cement Kiln Dust, etc. added to upper 8” of cohesive soil to Optimal Moisture and Stiffness.

Subgrade Stabilization – Soil Binder added to upper 6” of granular.

Aggregate Foundation Course – Clean, Crushed Aggregate Layer, Gradation and Angularity requirements

Aggregate Foundation Course with Binder – Gravel, Sand and Soil Binder. Older Specifications refer to this as Aggregate Foundation Course (Regular)

Bituminous Foundation Course – Bituminous Millings

Bit Sand Base Course – Oil mixed with granular material, historically used in the western half of the state, (Low truck count Highways) Sec. 509

Soil Aggregate Base Course – Defined in older Spec. books, no longer in Current Spec. Book.

Existing Stabilized Fill – Sand + Gravel + Cohesive Soil (10-15%)

A.2 Project Numbering

Date: May 2022
Source: Project Management Division

The purpose of project numbering is to provide policy for numbering highway construction projects. The Project Management Division is responsible for issuing project numbers to construction projects.

All project numbers consist of three major parts:

- Part 1 – Prefix

  The prefix indicates the appropriation type or the highway system. See Table A.2-2.

- Part 2 – Route Number/Zone

  The Route Number/Zone indicates the location of the construction project.

  For projects on the state highway system, the first three characters are the state highway route number.

  The final character is the zone of the route in which the project begins. Zones are established for the state from west to east and from south to north. Each state highway is assigned a direction for zoning purposes. Zones for the interstate system differ from those on the rest of the highway system. See Image A.2-1 and Image A.2-2.

  Projects off the highway system, but on a federal-aid route, use the four-character federal-aid route number as the second part of the project number.

  For projects off the state highway system, all four characters are in a single entity and have no relationship to highway route numbers or zones. They instead refer to the county or indicate that the project is statewide.

- Part 3 – Unit Number

  The unit number is a number that is sequential within each zone by highway route number.

  Projects not on the highway system and federal-aid interstate projects begin their sequential series with number 1. Other federal-aid projects on the highway system begin their series with number 101.

  Highway system projects not using federal funds (including interstate) begin their series with 1001. Projects that contain four characters in part three of their project number do not involve federal funds.

See Table A.2-1 for examples of project numbering.

Table A.2-1 – Project Number Examples

Prefix	Route Number	Zone	Unit	Project Number
STP	84	6	106	STP-84-6(106)
BRO	7084		5	BRO-7084(5)
HSIP	STWD*		30	HSIP-STWD(30)

**Note that the STWD references the location of the project (statewide) and is not part of the prefix.*

The general types of project numbers include:

- Projects on the state highway system are numbered using the highway number, zone and sequential number (in parenthesis).
  - Example – NH-2-3(112)

- Projects on links and spurs use the state spur/link number and a sequential number.
  - Example – STP-S55G(102)

- Urban system projects in cities of 5,000 or more population use the urban system (5,000 – 6,000 series) and the sequential number.
  - Example – URB-5044(3)

- Federal-aid secondary system projects off the state highway system (major and minor collectors) use the system number and a sequential number. Major collectors are numbered from 2,000 – 3,000, and minor collectors have a 7000 series number.
  - Example – RUR-2755(4)

- Projects not on a federal-aid system use the county number and a sequential number. Off-system county bridge projects use the county number preceded by “70” and a sequential number.
  - Example – TAP-55(110)
  - Example – BRO-7055(125)

There are other miscellaneous projects, particularly those with federal-aid special funding, which use project numbers not included in these general guidelines. These numbers are assigned by FHWA and have no correlation to our numbering system.

Table A.2-2 – Federal-Aid Project Prefixes and Descriptions

Prefix	Description	Federal Participation Rate
NH	**National Highway Performance Program** Resurfacing, rehabilitation or reconstruction of highways designated as part of the National Highway System, including the Interstate.	80% (90% on the Interstate if not used to add capacity)
BR / BH	**Federal-Aid Bridge – On System** Replacement (BR) or rehabilitation (BH) of bridges on the federal-aid highway system.	80%
BRO / BHO	**Federal-Aid Bridge – Off System** Replacement (BRO) or rehabilitation (BHO) of bridges not on the federal-aid highway system.	80%
STP	**Surface Transportation Program – Any Area** Construction, reconstruction, rehabilitation, resurfacing, restoration of federal-aid highways. These funds are generally used on non-NHS highways.	80% (90% on the Interstate if not used to add capacity)
LCLC / MAPA	**Surface Transportation Program – Urban Attributable** STP funds set aside for use in Nebraska's two metropolitan areas with a population over 200,000, Lincoln (LCLC) and Omaha (MAPA). These funds can be used for any of the purposes outlined under STP funds above.	80%
URB	**Surface Transportation Program – Urban** STP funds set aside for use in Nebraska's first-class cities (population between 5,000 and 50,000). These funds can be used for any of the purposes outlined under STP funds above.	80%
TAP	**Transportation Alternatives Program** Used for various activities such as bicycle/pedestrian trails, landscaping, rehabilitation of historic structures, and environmental mitigation.	80%
HSIP	**Highway Safety Improvement Program** Used to carry out safety improvements on any public road or publicly owned bicycle or pedestrian trail.	90%
RRZ	**Rail Highway Hazard Elimination Program** Used to construct new grade separation structures.	90%

Table A.2-2 – Federal-Aid Project Prefixes and Descriptions (Continued)

Prefix	Description	Federal Participation Rate
HRRR	**High Risk Rural Road Program** Used to carry out construction on roadways functionally classified as rural collectors and local roads.	90%
RRX	**Rail Crossing Protection** Used to improve rail highway crossings.	90%
SRTS	**Safe Routes to School Program** Used for projects to improve the ability of students to walk or bike to school.	100%
SPR	**State Planning and Research** Used by NDOT for planning and research activities.	80%
PL	**Metropolitan Planning** Allocated to metropolitan areas to carry out transportation planning processes required by federal law.	80%
PLH / FLH	**Public Lands Highways / Forest Lands Highways** Projects within, adjacent to or providing access to public lands or forest highways.	100%
DPS / DPU / EM	**Earmarks** Used for specific projects designated in federal legislation.	Varies
ER	**Emergency Relief** Emergency repairs and restoration of federal aid highways damaged by natural disasters or catastrophic failures.	80% – 100%

Table A.2-3 – State Funded Project Prefixes and Descriptions

Prefix	Description	State Participation Rate
S	Resurfacing, rehabilitation or reconstruction of state highways.	100% State Highway Cash Fund
SRR	Resurfacing, rehabilitation or reconstruction of roads into or within state parks and recreational areas.	100% SRR funds for roads within parks. 90% SSR / 10% Local for exterior roads.
NFG	State grade crossing funds used for rail crossing protective devices and closures.	100% NFG funds
TMT	**Train Mile Tax** State tax on rail traffic used for constructing, rehabilitation, relocating or modifying railroad grad separation structures.	Up to 100% TMT funds
RD	Restoration and rehabilitation projects such as armor coat, fog seal, joint and crack seal, asphalt and concrete patching.	100% State Highway Cash Fund
STR	Minor structure work such as bridge or box culvert repair.	100% State Highway Cash Fund
MISC	Minor projects such as culvert repair, landscaping or minor grading.	100% State Highway Cash Fund
ELEC	Minor electrical projects such as lighting and traffic signals.	100% State Highway Cash Fund

Table A.2-4 – Highway Numbers by Direction

Highway No.	Direction	Highway No.	Direction	Highway No.	Direction
1	West – East	38	West – East	83	South – North
2	West – East	39	South – North	84	West – East
4	West – East	40	West – East	85	South – North
5	South – North	41	West – East	87	South – North
6	West – East	43	South – North	88	West – East
7	South – North	44	South – North	89	West – East
8	West – East	45	South – North	91	West – East
9	South – North	46	South – North	92	West – East
10	South – North	47	South – North	94	West – East
11	South – North	50	South – North	95	West – East
12	West – East	51	West – East	97	South – North
13	South – North	52	South – North	98	West – East
14	South – North	53	South – North	99	South – North
15	South – North	56	West – East	103	South – North
16	South – North	57	South – North	105	South – North
17	South – North	58	South – North	109	South – North
18	West – East	59	West – East	110	South – North
19	South – North	61	South – North	112	South – North
20	West – East	62	West – East	116	South – North
21	South – North	63	South – North	121	South – North
22	West – East	64	West – East	128	West – East
23	West – East	65	South – North	131	South – North
24	West – East	66	West – East	133	South – North
25	South – North	67	South – North	136	West – East
25A	South – North	68	South – North	137	South – North
26	West – East	69	South – North	138	South – North
27	South – North	70	West – East	159	West – East
29	South – North	71	South – North	183	South – North
30	West – East	73	South – North	250	South – North
31	South – North	74	West – East	275	West – East
32	West – East	75	South – North	275B	West – East
33	West – East	77	South – North	281	South – North
34	West – East	78	South – North	283	South – North
35	South – North	79	South – North	370	West – East
36	West – East	81	South – North	385	South – North

Image A.2-1 – Non-Interstate Zone Map

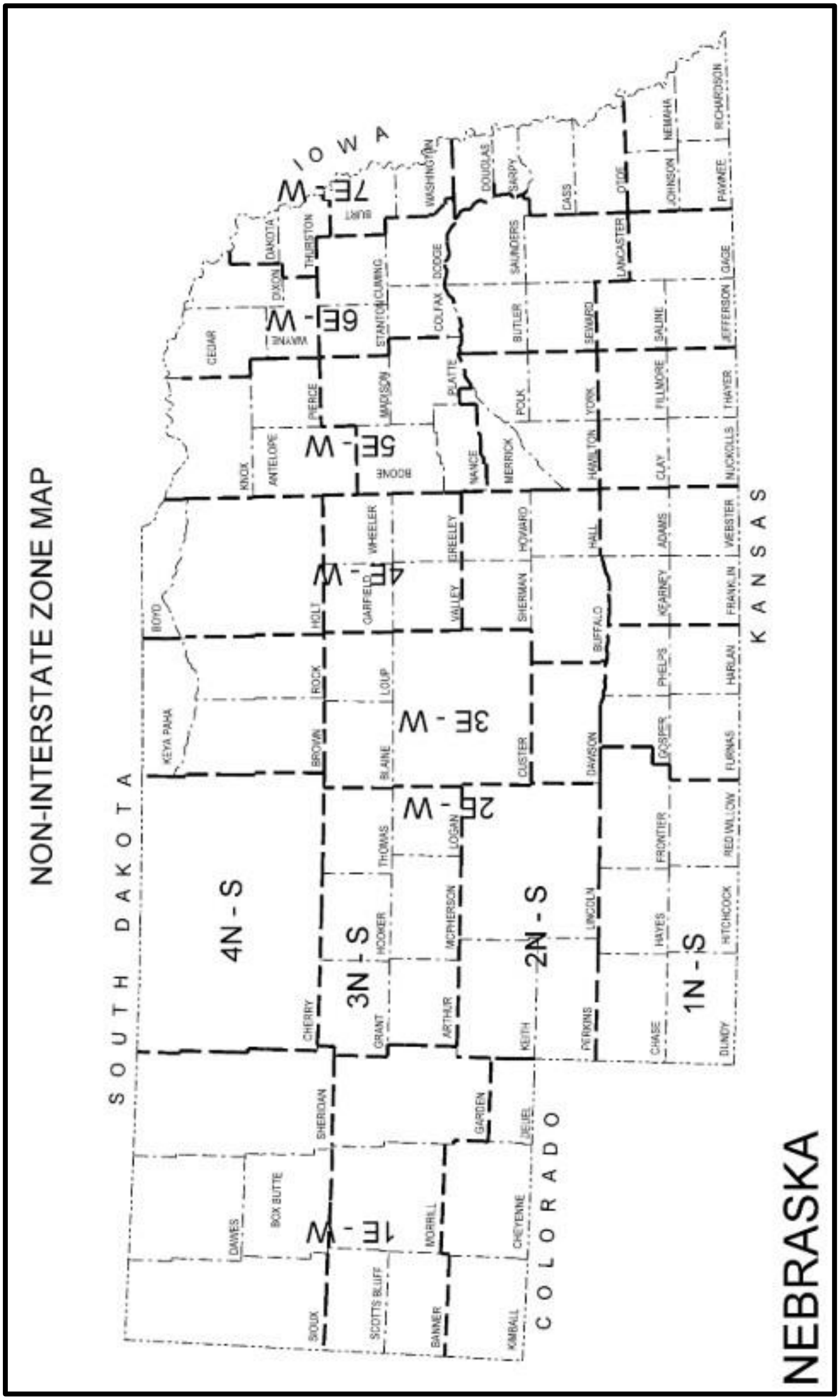
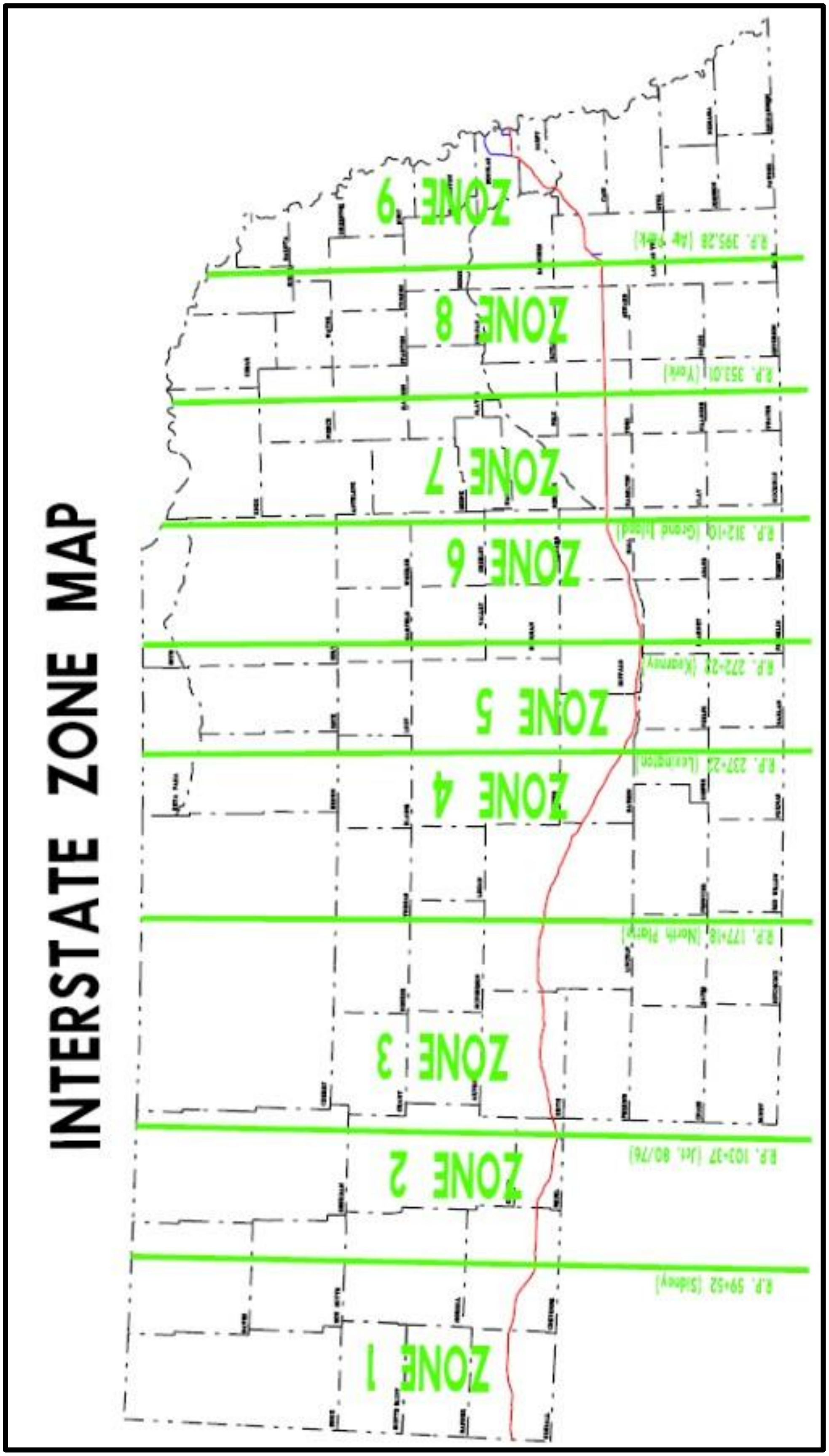


Table A.2-5 – Interstate Zones

Zone No.	Location	Interstate Route No.
1	Wyoming State Line – Sidney	80
2	Sidney – I-76	80
3	I-76 – North Platte	76, 80
4	North Platte – Lexington	80
5	Lexington – Kearney	80
6	Kearney – Grand Island	80
7	Grand Island – York	80
8	York – West Lincoln	80
9	West Lincoln – Omaha	80, 180, 480, 680
1	South Sioux City Spur – Iowa Line	129

Image A.2-2 – Interstate Zone Map



Date: January 2007, Revised 2023
Source: Syslo, Barrett

TESTING AND SAMPLE PREPARATION FOR:

- Lime Modified Subgrades.
- CKD Modified Subgrades.
- Fly Ash Modified Subgrades.
- Full Depth Reclamation with Fly Ash or Cement.
- Full Depth Pavement Pulverization using subbase material.

LIME MODIFIED SUBGRADES (using Pebble Quicklime)

1. Perform Eades and Grim test on soil to find target lime content (12.40 pH).
2. Perform soluble sulfates test on soil (<0.2% soluble sulfates in 10:1 H_2O to Soil).
3. Prepare specimens at 4% over optimum moisture (virgin soil).
4. Prepare specimens at target lime content and 1% over and 1% under.
5. Compact specimens.
6. Cure in sealed container at 75 degrees near 100% humidity for 6 days.
7. Cure in exposed atmosphere at 75 degrees for 24 hours.
8. Perform unconfined compression tests.
9. Report: virgin soil PI.
   virgin soil compressive strength.
   virgin soil optimum moisture & density.
   modified soil PI.
   modified soil compressive strength.
   modified soil density.

CKD MODIFIED SUBGRADES (Minimum 20% free Lime Material)

1. Perform soluble sulfates test on soil (<0.2% soluble sulfates in 10:1 H_2O to Soil).
2. Prepare specimens at 2% over optimum moisture (virgin soil).
3. Prepare specimens at 4, 6 & 8% CKD.
4. Compact specimens.
5. Cure in sealed container at 75 degrees near 100% humidity for 6 days.
6. Cure in exposed atmosphere at 75 degrees for 24 hours.
7. Perform unconfined compression tests.
8. Report: virgin soil PI.
   virgin soil compressive strength.
   virgin soil optimum moisture & density.
   modified soil PI.
   modified soil compressive strength.

FLY ASH (Class C Fly Ash) OR CEMENT MODIFIED SUBGRADES

1. Prepare specimens at 2% over optimum moisture (virgin soil).
2. Prepare specimens at 10%, 12% & 15% Fly Ash or 3%, 4% & 5% Cement.
3. Compact specimens.
4. Cure in sealed container at 75 degrees near 100% humidity for 6 days.
5. Cure in exposed atmosphere at 75 degrees for 24 hours.
6. Perform unconfined compression tests.
7. Report: virgin soil PI.
   virgin soil compressive strength.
   virgin soil optimum moisture & density.
   modified soil PI.
   modified soil compressive strength.
   modified soil density.

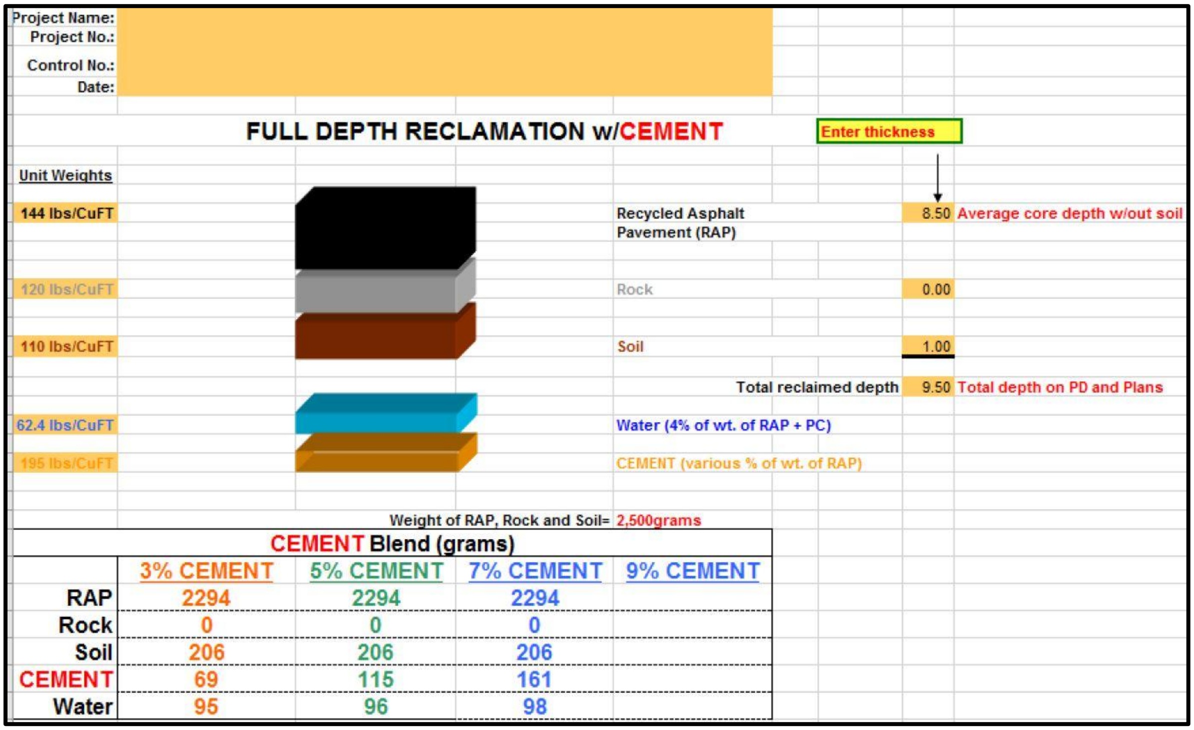
FULL DEPTH RECLAMATION WITH FLY ASH (Class C Fly Ash) OR CEMENT

1. Prepare samples by adding water to make the sample friable (millings and soil).
2. Prepare specimens at 10% & 12% Fly Ash or 3%, 4% & 5% Cement.
3. Add water 4% by weight of RAP + Ash.
4. Add soil based on thickness of soil incorporated in reclamation process.
5. Dry back small sample of blended material to determine total moisture content.
6. Compact specimens.
7. Cure in sealed container at 75 degrees near 100% humidity for 6 days.
8. Cure in exposed atmosphere at 75 degrees for 24 hours.
9. Perform unconfined compression tests.
10. Report: compressive strength, moisture & density.

Pavement Designers give this spreadsheet to the lab with updated pavement thickness and project information.

Source: Shared Folder - …Pavement Design\Stabilization Design\FDR w PC Lab Comps\

Figure B.1-1 – Cement Stabilized Bituminous Spreadsheet Example



Project Name:
Project No.:
Control No.:
Date:

FULL DEPTH RECLAMATION w/CEMENT

Enter thickness

Unit Weights

144 lbs/CuFT	Recycled Asphalt Pavement (RAP)	8.50	Average core depth w/out soil
120 lbs/CuFT	Rock	0.00	
110 lbs/CuFT	Soil	1.00	
	Total reclaimed depth	9.50	Total depth on PD and Plans
62.4 lbs/CuFT	Water (4% of wt. of RAP + PC)		
195 lbs/CuFT	CEMENT (various % of wt. of RAP)		

Weight of RAP, Rock and Soil= 2,500grams

CEMENT Blend (grams)

	3% CEMENT	5% CEMENT	7% CEMENT	9% CEMENT
RAP	2294	2294	2294	
Rock	0	0	0	
Soil	206	206	206	
CEMENT	69	115	161	
Water	95	96	98	

C.1 Historical Data of NDOT Asphalt

Date: 9/12/07 (edited 11-2018)
Source: Koves

ASPHALT THROUGH THE YEARS IN NEBRASKA (1950-2006)

FIFTIES TO EARLY SIXTIES

During the fifties and early sixties, a Low Type asphaltic concrete was produced. It consisted of gravel, sand and limestone dust filler. The mix design was made by the Bituminous Engineer and in the lab after mixing, a 4” x 4” asphalt cylinder was made on a compression machine. It consisted of a 4” mold with a double plunger. With the bottom plunger in place and the molding cylinder supported temporarily on the two steel bars, the hot mixture was added to the mold. The mixture was spaded two or three times around the inside of the mold with a heated spatula to reduce surface “honeycomb.” It was then compressed between the top and the bottom plungers under an initial load of about 150 psi to set the mixture against the sides of the mold. The pressure was then released, and the support bars removed to permit full double plunger action and the entire load of 3,000 psi was applied and maintained for two minutes. After removal from the mold specimens were cooled and a density was run by weighing in air and then weighing in water. The special provisions provided that the mixture be compacted to a percentage of the control density. During production a 2” mold was used to control density and had to be a certain percentage of the original 4 x 4 puck. These densities were made every 500 tons and everything was molded at 255°F ± 5.

EARLY SIXTIES TO MID SEVENTIES

As more earth roads were converted to gravel, more counties are showed interest in bituminous surfacing. The early 60's saw the culmination of many efforts, and high type bituminous road construction reached nearly fifty miles, more than half the total of gravel roads constructed with Federal funds in the same period.

July 1, 1964 to June 30, 1965, 144.8 miles of asphaltic concrete was contracted for work. The specifications and testing of asphaltic concrete was getting under way and by June 30, 1966 another 156.7 miles was let.

Testing of asphalt materials had come a long way. The first mix designs of the sixties and early seventies were created by the Flexible Pavement Assistant Engineer (FPAE). The mixes were based on the amount of traffic.

For higher traffic, a Type "A" mix was used. It contained crushed limestone, crushed gravel and limestone dust for filler with about 15% retained on the 3/8" sieve. For lower traffic roads the mix was Type "B" and contained mostly river gravels, sand and limestone dust filler and retained about 22% on the #4 sieve. And finally, a mix used for leveling courses and bridge wedges called Type "C". It contained about 100% crushed road gravel, had about 8% retained on the #4 sieve and an asphalt content of around 6.0 to 7.0%. It worked very well for leveling courses and also from keeping moisture from getting to the surface from below. The contractor would submit the materials for use and the aggregate was tested for quality and gradation. The FPAE would design the mix and measure the gradations of the aggregate. After figuring out the percentages, it was sent to the lab for mixing and testing. At least 2 to 3 designs were always made, one with high asphalt content and one about a percent lower. On each design, a variety of testing was done. The testing done on these 2 to 3 designs would verify that a mix would meet specification. The AC contents, gradations and densities controlled the project.

The asphalt cement used was penetration graded (hardness) and normally was 85 - 100. All designs were mixed at 300°F and all Marshall specimens were compacted at 250°F ± 5. Three 4" x 2 ½" specimens were molded using a Marshall hammer. This test was a 10 lb., slide hammer attached to a 4" round, slightly angled foot. The heated material was placed into the mold and then the whole assembly was placed on a rotating base. The 10 lb. slide hammer was inserted into the mold and the hammer would pound the asphalt a certain number of times, usually 50 blows. The sample was then flipped over and the routine repeated. After slight cooling, the samples were extruded and set aside to cool. When samples were at room temperature (approx. 1 hour wait), samples were weighed in air, then weighed in water, and then the saturated surface dried with a damp towel and weighed again. The densities were then figured and an average was obtained. All three samples were then placed in a 140° F water bath for 30 ± 5 minutes and stability and flow was obtained.

From the design, a Voidless Density (zero percent air voids) was obtained. It consisted of a sample approximately 800 – 1000 grams, which was cooled and broken into individual pieces. When cooled, it was placed into a calibrated glass container and weighed, covered with water at least an inch over the surface of the mix and placed under a vacuum of about 28 mm Hg. After about 10 minutes, the pressure was released and the sample was carefully placed into a water bath and weighed again. A maximum specific gravity was then figured.

Next, an extraction sample of about 1000 grams was weighed and a placed into an aluminum bowl. Perchloroethylene (a very strong degreaser) was then added, and the sample was stirred until broken down. From there, the sample was lidded and placed into the Rotorex (a centrifuge) where the liquid and asphalt was spun off into a calibrated flask. Perchloroethylene was then added in small portions and spun until liquid became straw colored. The clean sample was then scraped from the bowl and placed into an oven to dry. The liquid in the flask was also weighed and the temperature was taken. After the aggregate was dried and weighed, an asphalt content could be calculated. The oven dried sample was then washed, dried again and the gradation was obtained to ensure the specification of design.

The last test run was a Dry Displacement on the combined virgin aggregate. With the results from the Dry Displacement and a similar test called a Volumetric test, determined how the production was controlled in the field. A 1000 mL flask was used and a 1000 gram sample of the combined virgin aggregate was added. Perchloroethylene was added to a pre-determined line on the flask and the flask was then corked, rolled and bounced on a rubber pad for 10 minutes to remove all the air. After 10 minutes, the flask was filled back to the line and a siphon was used to remove solvent to a calibrated limit, weighed and a temperature was taken. The volume displaced by the virgin aggregate was then figured. During production in the field, they used the same test, only with the asphalt coated roadway material. A random sample was taken and a 1000 gram sample was split out from that. The testing was done the exact same way called a Volumetric and when completed, the two numbers were algebraically compared and an asphalt content was determined. The aggregate was then washed with solvent and a gradation was run.

After all the tests were run and the results were all figured, the engineers from the Flexible Pavement and the lab supervisor would all gather and look at the results to decide the asphalt content for production. They looked at the stability and flow of the Marshalls, how the mix looked, air voids, voids filled with asphalt (VFA) and then each voted on a percent binder to be added and the results were averaged. The required asphalt content, aggregate proportions and combined gradation to be maintained was then sent to the contractor and construction could begin. During production the contractor furnished a lab for a state employee to be on the job. The state employee ran all Volumetric tests and gradations out of that lab. Production samples were also sent to the Branch lab closest to the job sight for testing but the only pay factor items were for asphalt content, gradation, and density.

Trying new things and experimenting with different materials in asphalt was also big during this time period. We had already experimented using crushed glass as a replacement for aggregate and in the late 60's, the first of a few asbestos roads were built, using approximately 2% asbestos to replace the mineral filler. In the early 70's, we tried to use crushed Bakelite and there are even a couple of roads that contain shredded asphalt shingles. It seemed like everyone thought that waste products could be used in asphalt.

The asphalt cement (AC) during this era was penetration (hardness) and viscosity graded and the penetration most used was an 85 - 100. Voids in the asphaltic concrete surface (field density) varied from 3.5% to 12.0% depending on how much AC it contained and there were no real minimum or maximum requirements. Laboratory voids on Marshall specimens were running about 1.2% to 4.5%.

One other thing that should be noted during this time period is the crushing of our river gravels. During the 60's and 70's and even some into the 80's, the specification for the crushing of gravels was gradation limits before and after crushing. Most notably, the crushing specification for gravels was 70 ± 30% retained on the #4 sieve before crushing and after crushing the specification was 8 ± 8% retained on the #4 sieve. This made a highly angular material and worked quite well in our Type "A" mix designs for durability on our higher traveled roadways with the tire pressures and truck traffic at the time.

MID SEVENTIES THROUGH THE EIGHTIES

Prior to 1977, Limestone Dust had been used exclusively as mineral filler for asphaltic concrete. That changed in 1977 as soils and Fly Ash were tried and then used as mineral filler, both of which were cheaper to use. Soil was readily available everywhere and Fly Ash was a waste product left from burning coal and came from coal-fired, electrical plants. Also tried, but with not much success, was Stack Dust, Beet Lime and Volcanic Ash. Soil seemed to work quite well as filler if clay deposits were avoided. Light Peorian soil worked best and was easily broken down into a fine dust. If the clay content was too high; it would ball up and leave pock marks in the surface after a rain.

Around 1977, the Department started to read about the highways in Europe and how well they were performing. The Europeans were using an open-graded mix on their high-speed roadways. Nebraska's first attempt at this was placed on East "O" Street from 84th Street to the Lancaster County line in 1978. It contained Platte River gravel graded mostly to be retained on the 3/8" and #4 sieve and Fly Ash for filler. It was called "M-1", laid 1" thick and contained an AC of 4.70%. It was laid on top of a 2" mix called "Stone-filled", which contained about 60% large Limestone (mostly + ½"), some crushed gravel and Fly Ash filler with an AC of 3.0%. This design worked quite well for several years and being so open was also very drainable. The only problem was that the rounded river gravels did not have much skid resistance. In 1979, this mix was redesigned on the Alvo Spur to N-50 project. To increase skid resistance, Crushed Limestone, Crushed Gravel and Fly Ash were added. All of the round river gravel was removed and the mix was mostly retained on the #4 sieve.

Another new technique in the early 80's was milling of the roadway and using the millings back into the mix as aggregate. The first full-fledged design of this nature was F-281-1(101) Cowles Spur North and 50% of the design was the milled material. The rest was made up of Platte River gravel. This design was a little different because the aged binder, already in the millings, had to be accounted for. The millings had to be extracted using Trichloroethylene instead of Perchloroethylene, and the amount of asphalt figured into the total. During the early designs, the lab would run a Penetration on the aged asphalt. This was done by taking the liquid from the extracted material and boiling the solvent off until just the raw asphalt was left. The raw asphalt was poured into a small tin and cooled. After cooling, the sample was placed in a 77° F water bath for one hour and then a Penetration was run. This told us how hard the old asphalt was and what grade of asphalt cement to use. In the 80's, we went to viscosity graded asphalt and AC-10 was comparable to an 85 – 100, which is what was used for most virgin mixes. Since the asphalt was a lot harder in the millings, it was thought that using a softer grade would blend with the aged asphalt and create the desired grade. An AC-5 was used, which when pen graded, would be like a 120 - 150. For this design, 2.5% of new asphalt was added for a total of 5.1%. By introducing the millings into the design, it was a great cost savings to the State because of owning the millings. The project special provisions allowed the contractor to select the method for removal and pulverization of the old bituminous material. The only two requirements were that all of the removed material had to be reduced in size to pass a 2" sieve and that including any of the underlying base course should be avoided. No major problems were encountered during the production and lay down of the recycled mixture. Actually, the material appeared to be somewhat more stable than a design using virgin materials of the same gradation.

In the 70's and early 80's, the mix designs were still made by the Department. Field testing was still done by the State and the asphalt cement was still tested for penetration and viscosity, but the Department was moving forward. We were always looking at new technologies, test methods and designs around the country. As trucks got heavier, tire pressure increased, and traffic got higher, the designs had to get more structurally sound also.

LATE EIGHTIES TO MID-NINETIES

During the 80's, the Interstate was being overlaid and needed high performing designs that would withstand the increase in traffic. A modification of the Alvo to N-50 mix was tried. Limestone was replaced with Quartzite, a ledge rock from South Dakota. The Quartzite material was pink, very hard, and angular, and the "MQ" was born. "MQ" was open-graded with a thick coating of asphalt, very drainable and laid in a thickness of 1". This meant that during a rainstorm, the water would drain off of the pavement and not be thrown onto the windshield of the vehicle behind. The "MQ" contained about 65% Quartzite, 25% Crushed Gravel and about 5 - 10% Fly Ash. Eventually the "MQ" covered the Interstate and performed very well for many years.

During the 80's, more recycling work was done, this time with Crushed Concrete. Stockpiles of milled Crushed Concrete showed up around the state. Recycled Asphalt Pavement (RAP) jobs were working well, why not try this also. The problems encountered were minimal but there were things to be worked through. Crushed Concrete was very absorptive and no matter how much asphalt was added, the mix always looked dry. One other problem encountered throughout the years was that the piles of Crushed Concrete would set up and harden again over the winter and in the spring would have to be broken again and re-crushed. Recycled Crushed Concrete was tried for a few years, but never really took off for asphalt use.

In 1988, the FHWA issued a Technical Advisory (TA) about the asphalt design and field control of the mixes. The TA's purpose was to set forth guidance and recommendations relating to asphalt concrete pavement, covering the areas of material selection, mix design and mixture production and placement. The TA was directed primarily toward developing quality asphalt concrete pavements for high-type facilities. It covered such things as different materials, quality of the aggregates, how crucial dust to asphalt was, film thickness, properties of the binder, stripping, proper mix design and the control limits, etc.

In 1993, 1994 and 1995, a consultant was hired by the Department to conduct training on mix designing, properties of the mixes, what to look for, and how to get the desired volumetric properties with Nebraska aggregates. Voids, voids in the mineral aggregate (VMA), minimum AC and many other things were learned that needed to be done to conform to what the FHWA's TA deemed necessary for better roadways. New designs were initiated, crushing values of materials were looked at, target field voids were put at 3.5 - 4.0% and different Marshall blows for higher traffic roads. Any millings that were used in the designs were given crushed values. Our new designs were as follows:

- Type 1 — 80% crushed value for combined mineral aggregate
75 blow Marshall design
A maximum of 60% Limestone in the mix
4.0% target field air voids

- Type 2 — 60% crushed value for combined mineral aggregate
75 blow Marshall design
A maximum of 60% Limestone in the mix
4.0% target field air voids

- Type 3 80% crushed value for the mineral aggregates
  75 blow Marshall design
  A minimum of 50% Quartzite, Granite, or Crushed Gravel meeting 100% crushed value criteria.
  4.0% target field air voids

- Type 4 60% crushed value for the combined mineral aggregate
  50 blow Marshall design
  A maximum of 60% Limestone in the mix
  4.0% target field air voids

- Type 5 80% crushed value for the combined mineral aggregate
  50 blow Marshall design
  A minimum of 50% Quartzite, Granite, or Crushed Gravel meeting 100% crushed value criteria.
  4.0% target field air voids

- Type 7C Roadway mix constructed under traffic and parking areas
  20% crushed value for the combined mineral aggregate
  50 blow Marshall design
  A maximum of 60% Limestone in the mix
  3.5% target field air voids

- Type 7 Roadway mix when closed to traffic or shoulder mix
  0% crushed value for the combined mineral aggregate
  50 blow Marshall design
  A maximum of 60% Limestone in the mix
  3.5% target field air voids

Voids are the spaces between asphalt coated aggregate after molding of the Marshall specimens or after the rollers in the field. Voids are necessary for the longevity of the roadway. Too high of voids will tend to compact and ravel and if the voids are to low there is no place for the asphaltic concrete to go but to push and shove. After lay down and the finish rollers, the goal was 6% – 8% voids. After 6 - 10 years of traffic, the air voids should stabilize at 3% – 5% and remain for a few more years. When the roadway gets to 2% voids or less, the pavement is said to be at the end of its life.

Voids in the Mineral Aggregate (VMA) are the air voids between the virgin aggregate if you could mold a specimen of just the aggregate. VMA is important for design so that there is room for the asphalt cement. VMA varies from 13% – 15% and is dependent on the nominal aggregate size.

By 1994, the mix design and field testing was the contractor's responsibility with the Department verifying all results. Thus, the Quality Assurance/ Quality Control (QA/QC) program was initiated. The Department of Roads had 4 Branch laboratories (North Platte, Grand Island, Norfolk and Omaha) with the main lab in Lincoln. All five labs were furnished the same equipment so that correlation of testing between state labs was not a problem. Also, a list of equipment was made for the contractor that was needed for their testing. The contractors began buying trailers and equipping them with the necessities. Marshall machines, rice apparatus (voidless density), ovens, sieves, shakers, sample splitters, running water, air conditioning, computers, fax machines, etc. were all included in what the contractors needed to include in their labs. Unfortunately, our consultant made the mistake of saying that the sands of Nebraska were "unique". These sands, unlike the rest of the country, were great builders of VMA and the cost for the material was minimal. Our new mix designs, though having better mix and field specifications, ended up not being exactly the product that we wanted. Although we had a specification for crushed value on the design, it seemed like after a couple of years that more and more of our "unique" sand was showing up in our mixes. We had given contractor crushed values for their aggregates which we thought were reasonable. For example, crushed ledge rock was given a value of 100%. Crushed Gravel was given 80% crushed value and plain River Gravels and Sands were 0%. If a design contained 25% Crushed Rock and 65% Crushed Gravel and 10% Gravel, its crushed value was 77%.

$$(25 \text{ x } 100\%) + (65 \text{ x } 80\%) + (10 \text{ x } 0\%) = 77\%.$$

If the design criteria for this mix had 60 % crushed value, it looked like a good design. Somehow though, more and more of our VMA building sands were entering the designs and our mixes ended up becoming very tender. The Department ended up with designs that would rut or fail even before the job was finished. We had taken a big step with our specifications during this time, even if the roads ended up not quite where we wanted them. The contractor was running their own samples with our verification. Field samples were now being controlled, not only density and binder content, but voids, VMA, minimum asphalt contents, gradations and dust to asphalt content. Even though some mix designs left a lot to be desired, some worked quite well and we had learned quite a bit that helped us get into the next phase of building better roadways.

In the late 80's, the Strategic Highway Research Program (SHRP) developed the "Superpave" program. The program consisted of new ways to test asphalt cement (now called Performance Graded Binders) and to check the asphaltic concretes properties during design and field testing. Most testing at SHRP was finished by the early 90's and the Federal Government was looking for states to try the new test methods. In 1996 and 1997, the Feds offered states money to buy new Superpave equipment and build roads to the new specifications. Superpave design methods are based on Equivalent Single Axle Loads (ESAL). This is a means of equating various axle loads and configurations to the damage done by a number of 18,000lb single axles with dual tires, on pavement of specified strength, over the design life of the pavement. Originally, 7 designs were created with SP-1 being the road with the lowest ESAL and SP-7 the highest.

Testing and equipment was quite different, especially on the binder side. New equipment was purchased and new test methods were learned. The asphalt cements went from 85 - 100 and AC-10 to PG 58-28, which were climate and temperature graded binders. The numbers were based on records from the National Weather Service and several different weather stations around the United States from the last ten years. The first number (58°C) being the average high temperature of the roadway during the summer months and the last (-28°C) being the one time low during the winter. Higher grades of binder were also better suited for highways with more ESAL's such as PG 70-28 (polymer modified) may be used on the Interstate system because of the higher tire pressures and larger trucks.

Binder testing changed with testing at high temperature, low temperature, before aging and after aging, checking phase angles and elastic properties. The Dynamic Shear Rheometer (DSR) was used to report phase angles and the dynamic shear of the binders. Phase angles indicated whether polymer modifications were present. Dynamic shear was an indication of the binder stiffness at the upper grade temperature and indicates the "viscous behavior" at a lower temperature, after aging.

The Rolling Thin Film Oven (RTFO) simulated the aging of an original binder after going through the field hot mix plant during production. This material could be re-run through the DSR to measure aging occurs during production.

The Pressure Aging Vessel (PAV) took the RTFO material through a timed process of controlled heat and oxidation. The PAV simulated the long term aging of the binder before it was run through the DSR for the purpose of Dynamic Shear (lower temperature viscous behavior) testing again. The Bending Beam Rheometer (BBR) and Direct Tension (DT) gave test data at the lower temperatures. The BBR and DT were used to determine the low temperature stiffness and tensile properties of the binder. Stiffness correlates with brittleness at low temperatures and brittle materials are more likely to crack (BBR) or fracture (DT).

The Elastic Recovery Apparatus worked in conjunction with the DSR phase angle for modified binders. It indicated whether adequate polymer modification was present by measuring its "elastic" properties.

The changes on the mix design were not quite so drastic. In place of the Marshall, which molded a 2 ½"x 4" specimen, was a Gyratory Compactor which molded a 4 ½" x 6" specimen. Instead of the Slide Hammer pounding the sample a certain number of times on each side, a plunger would be hydraulically inserted into the mold with 600Kps of pressure, an angle of 1.25° placed on the sample and a set number of gyrations would all be started and stopped automatically. Each time the mold rotated, a height was obtained and printed out. All the new designs were figured for N initial, N design and N maximum and density were figured at each height. From this puck, a density was run and that density was N maximum or end of the life of the pavement. N design and N initial were back figured with a simple algebraic formula.

The Rice test (maximum gravity) was basically run the same way as always and with this number and the gyratory densities, air voids at each level were figured. N design should be between 3% - 5% air voids and N maximum should be somewhere around 2%.

Superpave design changed the way that the asphalt content was obtained. The use of toxic chemicals and centrifuges were eliminated. The new method involved an ignition oven where temperature was kept at 538°C and when the asphaltic concrete sample was weighed and placed into the ignition oven, the weight was entered on the oven. As the asphalt was burned off, the asphalt content was printed out and automatically shut off when burn off was complete. After cooling, this burned off sample could then be washed and a gradation obtained. Perhaps the greatest innovations that SHRP developed, was the technique for finding the angularity of the fine materials. The method obtained a void content and the device was very simple but effective, involving -8 /+100 material. A mason jar with no bottom was inverted and screwed to a calibrated funnel on a tripod. Below the funnel was a calibrated brass cylinder. A finger was then placed over the hole in the funnel and the sample was poured into the mason jar and leveled. The finger was removed and the sample free fell into the cylinder. The cylinder was carefully scraped off with a straightedge and weighed. After calculating, a person could tell how angular the fines were by the void content. The higher the number the more angular the fine material was. This test was very important to roadway longevity.

Other aggregate tests included the Coarse Aggregate Angularity which was a visual count of materials above the #4 sieve. Flat and Elongated, which used a device at 5:1 ratio to determine the amount of flat pieces compared to normal crushed material. To many flat pieces in a roadway surface can cause early failure. The last test, Sand Equivalent showed the relative proportions of fine dust or claylike material in graded aggregate.

LATE NINETIES TO MID 2000'S

The first 2 Superpave jobs were let in 1997. The contractors were just getting gyratory compactors, the design was ran with both gyratory compactor and Marshall hammers as a comparison. Both designs were called SP-97. The first project, let in February, was constructed by U.S. Asphalt from Omaha was RD-50-1(1006), In Tecumseh. It was an SP-4(3/4") containing 28%-5/8" Crushed Rock, 32%-¼" Limestone Chips, 15% Limestone manufactured sand and 25%-Crushed Gravel. The binder used was PG 64-22 and the percent added was 4.65% (by weight of mix). Superpave mix specifications used were: Gyratory % air voids @ Ndes = 4.0 ±1.0%, VMA = 13%, Void filled with Asphalt = 65% – 78 % and field Marshall air voids = 3.5 ±1.0% was subject to change based on the Gyratory results. The job was only ½ mile long and was produced during early to mid June. During the Test Strip, the voids barely reach 2.0% and VMA never got over 11.5%. Binder and aggregates were adjusted slightly to get the design into specification and production continued. The new result was fairly consistent but still had some highs and lows. The mix was quite open and in some spots was placed between curb and gutter.

The second project, let in May, was constructed by Henningsen Construction from Atlantic, IA was EACSTPDSTPP-50-2(120) Louisville to Springfield. It was an SP-5(1/2") containing 5%-5/8" Limestone Chips, 25%-¼" Limestone Chips, 30%-Crushed Gravel, 20% Limestone manufactured sand, 20%-¾" Crushed Gravel. The binder used was PG 64-22 and the percent added to the design was 4.90%. Superpave mix specifications that were used were: VMA = 14%, Voids Filled with Asphalt = 65% - 75%, Gyratory air voids @ Ndes = 4.0±1.0 and initial field Marshall air voids of 4.0±1.0% subject to change based on gyratory results. The seven miles of construction took place the end of August and finished in early September. The production Gyratory pucks at Ndes ran very close to the specifications with the voids at about 4.0% and VMA running about 14.3%. Marshall results ran slightly lower on both. This project was built along a rock quarry with very large, heavy truck traffic and seemed to perform quite well.

In 1998, seven projects were Superpave. The department went from one end of the ESAL spectrum to the other. We made two SP-1's, two SP-2's, one SP-3, one SP-4 and one SP-5 on Interstate 680 in Omaha. All of the designs contained between 17 and 25% millings with the exception of the 680 project, where no millings was used at all. The department bought Gyratories for all the branch labs and the contractors were gearing up with all the necessary Superpave equipment too. It was quite a costly project, but there was a significant increase in the performance of the asphaltic concrete over time.

By 1999, thirty-six Superpave projects were let and the Marshall equipment was being used less and less. The contractors were designing mixes using the gyratory compactor and using Superpave volumetric and consensus properties to control the mix in the field. Three 10,000 gram batches of their design were submitted to the NDOR lab for verification along with 6 gyratory pucks prepared for moisture susceptibility. The department was verifying all mix designs and correlating well with the contractor design and field samples.

In the 1960's, 1970's and 1980's, designs were controlled with field density, asphalt content and gradation. In the 1990's, design controls were added for Voids, VMA, minimum asphalt content and a certain percent of crushed materials. Superpave added even more control. By 1999, the department was looking at plant produced gradations, binder content, air voids, VMA, VFA, FAA, CAA, dust to asphalt and even whether the design had a tendency to strip or not. Better grades of binder were used for higher traffic roadways. At least one QC sample was tested for each 750 ton of mix produced. That random sample was split by the contractor and half was sent to the NDOR lab for correlation. During construction, if two consecutive points were outside the Specification limits, production was stopped until the problem was fixed.

By the end of 1999, it was decided that certification of the contractor's test technicians, was necessary and another consultant was hired for technician training, mix design and certification. This consultant also trained NDOR personnel in the new methods of testing and ways to help control mixes during production. The end result was the contractor's responsibility and generally produced better roadways.

By the year 2000, Superpave was the only mix type specified for asphalt surfacing, including rebuilds and overlays. In May of 2000, three tied projects were started using an SP-2(0.5) mix design. The three tied projects were EACSTPD-43-2(106) Adams to Bennet, RD-S55G (1007) Hickman Spur and RD-S34B (1002) Firth Spur with the Adams job starting first. The project was started using a PG 58-28 binder from Koch Material at 5.00% (by weight of mix). The roadway surface was milled and the new asphalt was to be laid in 2 lifts of 2" each. The first and second lifts went down smoothly with air voids between 3.2% to 4.0% and VMA of about 14.4% to 15.0%. FAA on the original design was 43.5 and during construction, using the burn-off, it still ran in the 42's. Just after July 4, 2000, the Firth Spur was started, using the same design. During this time period, we had very hot and humid weather and things began to change. The Adams project started flushing and by the 8th of July, the Firth project had been stopped to see what the problem was, so it didn't continue. Cores were taken, valuated and NDOR could find nothing out of the ordinary except that now, where the top lift had originally been about 5.00% binder, it was now between 6.50% and 10.00%. After splitting the cores on the lift line, the bottom still contained about 5.00%. When looking at some places on the project, a person could take a spade and scrape off about 1/8" to ¼" of pure binder for thirty or forty feet at a time. In other spots no flushing was noted. A letter was sent to the contractor to ask what course of action he was going to take to alleviate the problem. An upgrade in PG binder was suggested and the job was switched to PG 64-22 from Trifinery. The project resumed August 22nd with the new binder and shut down again with the same problem on the Firth job August 24th. The mix was now totally redesigned, pulling the Limestone screenings and replacing with millings from the project. The project resumed September 8 and no further flushing was found on the rest of that project, nor the Hickman Spur project.

Over the next couple of years, more projects with SP-2 designs were found to be flushing. During the Bennet project several cores were taken and kept in storage. Since the department had done all the testing it could do and really found nothing, some of the cores went to Western Research Institute in Laramie, WY. and some to the North Central Superpave Center at Purdue University in West Lafayette, IN. to see what they could find. Nobody could come up with anything conclusive as to why our SP-2's was flushing. In 2003, the Department decided that the flushing possibly was result of our Fine Sands. Since the SP-2's FAA was only 40, the specification was changed to 43 and seemed to alleviate some of the problems with these mix designs. In 2003, a new mix was tried and later used exclusively, for all low volume roadways. It had all the properties of an SP-4 with the exception of the gyrations which were like our SP-2 at 117. It was called an SP-4 Special, tried on a few projects in 2003 and from then on has taken the place of our SP-1's, 2's and 3's. During 2002, the University of Nebraska (UNL) was developing an asphalt research program in conjunction with the Department of Roads and their first project was the SP-2 flushing project. The UNL project was finished in the year 2005 and their conclusions were about like everyone else's. There was not a clear-cut answer as to why the SP-2 mixes flushed.

Over time, Superpave mixes started to show stripping problems. A liquid anti-strip was added to the mix but the quality and variability between producers was great. Contractors had been adding their own anti-strip at about 0.5% to percent total binder (The quality of the anti-strip varied.) About 2002, the department found that some anti-strips were not compatible with the binders being used. At that time, we made it the responsibility of the binder producer to add and certify that the correct amount was added before the contractor received the binder for a project.

The Department also found that binder producers were using polyphosphoric acid to modify the upper temperature of their PG binders, to reduce the cost of real polymers. The acid was a lot less expensive and the upper temperature specifications could be met using the acid. The problem was, they didn't produce the highly modified binders were specified. The other concern was that acid modification would react with Limestone and increase stripping over time. The modified binder specification was changed and producers could only incorporate a blend of base asphalt and elastomer modifiers of styrene-butadiene (SB), styrene-butadiene-styrene (SBS) or styrene-butadiene-rubber (SBR). No acid could be used.

In 2004, the department decided that liquid anti-strips did not meet moisture sensitivity requirements. The industry had been using hydrated lime, as an anti-stripping agent, for quite a while. Nebraska had done some experimentation with Hydrated Lime and Type 2 Cement on a couple of earlier projects and it seemed to perform well. In late 2004, several projects were let with the option of using 1% Hydrated Lime in their mix designs as an anti-stripping agent and a specification was written. Originally the virgin aggregate was moistened and the Hydrated Lime was pug milled onto the aggregate, mixed thoroughly and dried, and then the % binder was added. By 2005, all mainline surface designs contained at least 1% Hydrated Lime and could be added by pug mill, Lime Slurry or premixed and stockpiled for use during the project.

Also in 2005, some contractors asked the Department if they could verify their designs during the construction process instead of submitting verification samples to the Lincoln Lab. After some discussion, it was decided that it would be tried on a few projects and the mix design would be verified in the 1000-ton test strip by the NDOR Branch Lab closest to the project. By 2006, all projects were handled in this way, and it seemed to work well.

C.2 SuperPave Asphalt Mix Design Requirements

Date: 1999
Source: Rea

Table C.2-1 – SuperPave Requirements for Type SP1, SP2, SP3, SP4, SP5, SP6 & SP7

COARSE AGGREGATE ANGULARITY
ASTM D5821

	TRAFFIC, ESAL's	DEPTH FROM SURFACE	
		<100 mm	>100 mm
1	< 300,000	55	----
2	< 1,000,000	65	----
3	< 3,000,000	75	50
4	< 10,000,000	85/80	60
5	< 30,000,000	95/90	80/75
6	<100,000,000	100/100	95/90
7	≥100,000,000	100/100	100/100

FINE AGGREGATE ANGULARITY
AASHTO TP33, METHOD A

	TRAFFIC, ESAL's	DEPTH FROM SURFACE	
		<100 mm	>100 mm
1	< 300,000	----	----
2	< 1,000,000	40	----
3	< 3,000,000	40	40
4	< 10,000,000	45	40
5	< 30,000,000	45	40
6	<100,000,000	45	45
7	≥100,000,000	45	45

FLAT, ELONGATED PARTICLES
ASTM D4791

	TRAFFIC, ESAL's	PERCENT, MAXIMUM
1	< 300,000	----
2	< 1,000,000	----
3	< 3,000,000	10
4	< 10,000,000	10
5	< 30,000,000	10
6	< 100,000,000	10
7	≥ 100,000,000	10

CLAY CONTENT
AASHTO T176

	TRAFFIC, ESAL's	SAND EQUIVALENT, MINIMUM
1	< 300,000	40
2	< 1,000,000	40
3	< 3,000,000	40
4	< 10,000,000	45
5	< 30,000,000	45
6	< 100,000,000	50
7	≥ 100,000,000	50

Table C.2-2 – SuperPave Requirements for Type SP1, SP2, SP3, SP4, SP5, SP6 & SP7

	GYRATORY COMPACTION EFFORT			
	DESIGN ESAL's (MILLIONS)	Nini	Ndes	Nmax
1	< 0.3	7	68	104
2	0.3 – 1	7	76	117
3	1 – 3	7	86	134
4	3 – 10	8	96	152
5	10 – 30	8	109	174
6	30 – 100	9	126	204
7	> 100	9	142	233

MIXTURE DESIGN REQUIREMENTS

VMA CRITERIA	
NOMINAL MAXIMUM AGGREGATE SIZE,	MINIMUM VMA, PERCENT
9.5 mm	15.0
12.5 mm	14.0
19.0 mm	13.0
25.0 mm	12.0
37.5 mm	11.0

	VFA CRITERIA	
	TRAFFIC, ESAL's	DESIGN VFA, PERCENT
1	< 300,000	70 – 80
2	< 1,000,000	65 – 78
3	< 3,000,000	65 – 78
4	< 10,000,000	65 – 75
5,6,7	≥ 10,000,000	65 – 75

C.3 Historical Layer Coefficient Data

Date: 7/28/71, 1998, 2023
Source: Inghram, Rea, Pavement Design

Image C.3-1 – Structural Coefficients for Flexible Pavement (1971)

July 28, 1971

Donald Inghram

L. J. Bryant

DESIGN COEFFICIENTS FOR FLEXIBLE PAVEMENT

In connection with setting up a computer program for design, we feel that it would be desirable to have a list of standard coefficients for various pavement components which would be acceptable to everyone concerned. Consequently, Rosecrans has prepared the following list for consideration. These values are based on Table 8, page 438, of the AASHO Road Test report SR-73.

Surface Course	New Surface	Existing Surface
Asph. Conc. Type A	0.44	0.24
Asph. Conc. Type B	0.40	0.24
Asph. Conc. Type C	0.44	0.24
Asph. Conc. Type D	0.40	0.24
Asph. Sand	0.30	0.18
Armor Coat	--	--
Bit. Sand Base	0.20	0.12
Mixed-in-Place	0.15	0.09

Base Course	New	Existing
SABC	0.14	0.14
Bit. Sand Base	0.20	0.20
Lime Treated Subgrade	0.14	0.14
Mixed-in-Place	0.15	0.15
Granular Subbase	0.11	0.11
Crushed Rock	0.14	0.14
P. C. Concrete	0.50	0.40

It is recognized that some of the figures are somewhat controversial and that changes will probably have to be made before agreement is reached. AASHO states that the 0.44 for high stability asphaltic concretes and the 0.11 for subbase are the only values based on data and that all the others are assumptions.

BIT MILLINGS BASE	.20	50,000 PSI
SABC	.14	22,000 PSI
EXISTING AC	.24	450,000 PSI
NEW AC	.44	450,000 PSI
BIT SAND	.20	150,000 PSI
FOAMED ASPHALT	.18	

Image C.3-2 – Structural Coefficients for Flexible Pavement (1971)

L. J. Bryant
Page 2
July 28, 1971

The 0.40 for Type B and Type D is a departure from our usual practice, but this is based on the fact that it usually has a Marshall Stability between 1000 and 1200. On the other hand, the only asphaltic concrete given a value of 0.44 is that with a stability of 2000 or over, and for Type A and C this may be stretching it somewhat. In any event these and several other factors will need to be reviewed.

Bituminous Materials Engineer

DIInghram:mf

cc: OLLund
CRosecrans

Image C.3-3 – Structural Coefficient for Bituminous Millings Base (1998)

Nebraska
Department of Roads

Memorandum

Date: November 18,1998

To: Bob Rea

From: Dan Nichols DDN

Subject: Structural Coefficient for Bituminous Millings Base
(Pavement Design)

Initial results from the studies conducted at Kansas State's accelerated testing laboratory indicate that .26 is a reasonable value to use for the structural coefficient for a bituminous millings base. We expect the final report to be available within the next two to three months and we will provide you with a copy of that report.

xc: G.V. Woolstrum

ANOTHER KANSAS STUDY SHOWED FOAMED ASPHALT HAS STRUCTURAL COEFFICIENT OF 0.18.

Figure C.3-1 – Recalibration of the Asphalt Layer Coefficient (2003)

National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

Recalibration of the Asphalt Layer Coefficient

RESEARCH SYNOPSIS 09-03

Background

Although many highway agencies are exploring the use of new mechanistic-empirical pavement design methods, many currently still use the pavement design guide based on the AASHO Road Test in Ottawa, Illinois, from 1958 to 1960. This test established an empirical relationship between traffic loading and pavement thickness. One of the key inputs to this method is the layer coefficient for the hot mix asphalt (HMA) layers. This HMA layer coefficient has not been updated in more than 50 years despite numerous improvements in mix design methods, quality control and construction of HMA.

Objective

The primary objective of this study was to recalibrate the asphalt layer coefficient based on current paving materials and construction using data collected at the NCAT Test Track accelerated pavement testing facility (Figure 1).

Figure 1. NCAT test track

Description of Study

In the first phase of the study, a sensitivity analysis was performed to determine the influence of design inputs on the resulting HMA thickness. A total of 5,120 design iterations were conducted, as the inputs were varied. Analysis of these data showed that the layer coefficient was the most influential parameter on the resulting HMA thickness.

The second phase of the study involved recalibrating the asphalt layer coefficient using traffic and performance data from the structural sections of the 2003 and 2006 NCAT Test Track. Using the 1993 AASHTO Design Guide flexible pavement design equation, the predicted amounts of traffic in equivalent single axle loads (ESALs) were calculated to reach given levels of pavement serviceability. The predicted ESALs were compared to actual traffic on the sections. Least squares regression was performed to determine new asphalt layer coefficients for each section.

Key Findings

Figure 2 shows the computed layer coefficients for each section individually, as well as the average HMA layer coefficient of 0.54. Two test sections resulted in layer coefficients lower than the AASHTO recommended value of 0.44; however, forensic investigations showed that both sections had poor bonds between asphalt layers. No trends were observed relative to overall pavement cross-section, HMA layer thickness or binder grade.

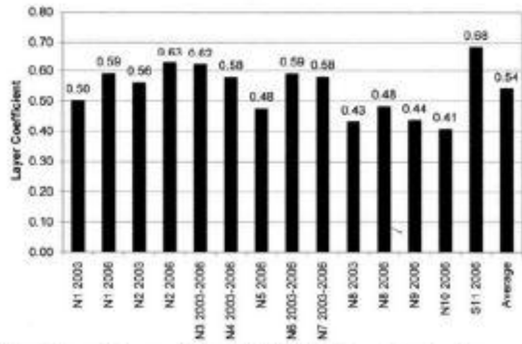


Figure 2. Computed layer coefficients for NCAT Test Track structural sections

The relationship between HMA layer coefficient and HMA modulus at 68°F is given graphically in the 1993 AASHTO Design Guide. This relationship can be modeled using a natural logarithmic function, allowing extrapolation for higher modulus values. When using the average HMA modulus value (backcalculated using falling weight deflectometer data) for the structural sections of the 2003 NCAT Test Track, this extrapolation yielded a corresponding layer coefficient of 0.543, which serves as additional validation of the 0.54 value in Figure 2.

Recommendations for Implementation

This study recommends an asphalt layer coefficient of 0.54 for flexible pavement designs using the AASHTO Design Guide. Increasing the coefficient will result in approximately 18 percent thinner HMA cross-sections. This translates directly into annual cost savings and/or more efficient use of HMA material to pave more highway mileage.

Care should be exercised when applying this coefficient in other states, as the recommended layer coefficient is based on the environmental conditions and materials used in this study.

For the Full Report, see NCAT Report 09 - 03 at www.NCAT.us

Acknowledgements and Disclaimer

This study was sponsored by the Alabama Department of Transportation. This Research Synopsis provides a brief summary of the study's final publication. This document is for general guidance and reference purposes only. NCAT, Auburn University, and the listed sponsoring agencies assume no liability for the contents or their use.

Use these Structural Coefficients when designing a project using the 93 AASHTO Method.

Table C.3-1 –Asphalt Layer Structural Coefficient (2023)

Type of Pavement	Structural Coefficient
New Asphalt	0.54
Existing Asphalt	0.24 – 0.35
Existing Bituminous Sand	0.20
Bituminous Millings	0.20
Cold-In-Place Recycle	0.25
Full Depth Reclamation w/PC or Fly Ash	0.25
Full Depth Reclamation w/water only	0.14
Hydrated Lime Slurry Stabilization	0.25
Foundation Course	0.20
Soil Aggregate Base Course	0.14
Lime or Fly Ash Stabilized Subgrade	0.22

Note: PCC does not have a layer coefficient. However, a value of 0.50 – 0.75 has been used by some researchers for comparison purposes only.

C.4 Typical RAP Percentages of Past Mix Designs

Date: Nov 2008
Source: Koves

Year	Project	Type	AC	Content Recycle
1984	80-5(42)	A Sp CC	4.6%	45% Crushed Concrete
1985	80-7(75)	RCC	5.3%	40% Crushed Concrete, 50% Millings, 10% 3A
1986	80-3(96)	A Sp CC	5.3%	47% Crushed Concrete
1993	19-2(1002)	CCF	7.0%	50% Crushed Concrete
1992	12-6(1008)	RB	?	30% Millings
1993	11-2(110)	RB	4.0%	15% Millings
1993	15-2(104)	RB	4.6%	30% Millings
1983	2-3(106)	R	?	50% Millings
1983	6-3(105)	R	?	50% Millings
1984	2-4(103)	R	?	50% Millings
1984	2-6(111)	R	?	50% Millings
1984	34-4(104)	R	?	40% Millings
1985	6-3(108)	R	?	50% Millings
1990	20-3(1004)	AX	4.8%	32% Rock, 65.5% 3A, 2.5% Fly Ash
1992	15-3(109)	RAX	3.5%	25% Millings
1992	20-6(1008)	RAX	4.3%	50%
1993	14-3(1008)	RAX	4.2%	20% Millings
1993	20-4(116)	RAX Sp.	5.6%	15% Millings

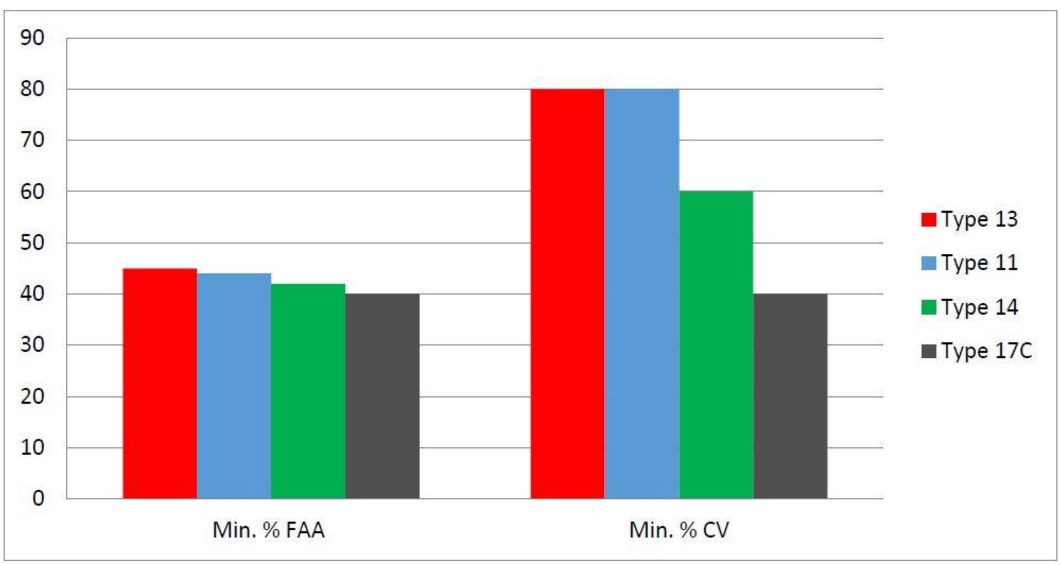
C.5 Typical Crushed Values and/or FAA (Fine Agg. Angularity) of Past Mix

Date: Data 6/14/95
Source: Koves/Varilek See 1970s Spec. Book & PDM Sec 12.3

Asphalt Type	Material or Use	Crushed Value[1]	FAA
A, AX, or RAX	3A Gravel & Ledge Rock	80%	-
A Special	3A Gravel & Ledge Rock	85%	-
B or RB	Mainly 2A Gravel, Some Sand	20%	-
C	3A Gravel & Sand	70%	-
MQ	Quartzite & 3A Gravel	100%	-
Q or RQ	Quartzite	100%	-
13	Interstate/Expressway Surface	80%	45%
11	Interstate/Expressway Base	80%	44%
14	Medium Volume Roadways	60%	42%
17C	Low Volume[2]	40%	40%

Note: 2A & 3A Gravel – See 1970's Spec. Book and Sec. 12.3 (PDM) for more info.

[1] Crushed Value *approximates* percentage of crushed material. Experience showed that although the material passed through a crusher, not all was crushed, resulting in lower actual percentages.
[2] Mix exhibited severe rutting.



Note: FAA and Crushed values are not directly related.

C.6 Evolving Rehabilitation Strategies for Asphalt Pavement Research

Evolving Rehabilitation Strategies for Asphalt Pavement

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Strategies for rehabilitation of flexible pavements have traditionally included mill and overlay or reconstruction. Three new strategies for flexible pavement rehabilitation currently being utilized by the Nebraska Department of Roads include partial depth in-place recycling, full depth in-place recycling and full-depth reclamation. Cost differences between the new and the traditional strategies were evaluated and conditions under which the new strategies are appropriate for use are discussed in this paper. A comparison based upon cost of rehabilitating a section of pavement using the two conventional and three new strategies found that the three recycling strategies provide flexible pavement which costs more than mill and overlay but less than complete reconstruction.

Keywords: Pavement rehabilitation, recycling, reclamation.

1. Introduction

When flexible pavement has deteriorated to the point where rehabilitation or reconstruction is necessary, pavement engineers have traditionally used either the mill and overlay strategy or complete reconstruction. Today numerous other alternatives are available, each of which is characterized by a different level of cost and performance.

Newer evolving strategies include several variations of partial or full depth milling and full depth stabilization/reclamation. This paper explores the Nebraska Department of Roads (NDOR) experiences with several alternatives to mill and overlay or complete reconstruction and analyzes these alternatives with regards to initial cost.

State transportation agencies are being required to maintain a larger inventory of pavement with little or no increase in budget, causing engineers to seek solutions that deliver a sustained level of performance coupled with cost-effectiveness. The procedures discussed in this paper provide effective alternatives to the constraints imposed by many of the more traditional strategies.

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2. Conventional Approaches

The conventional, least expensive approach to correcting flexible pavement distress has been mill and overlay. This process typically involves milling 25-100 mm from the existing pavement to remove surface distress (i.e. rutting, cracking, raveling, etc.). A tack coat is applied to the milled surface and a layer (or two) of new asphalt is added as a leveling/wearing course. Nebraska has many sections of highway where the mill and overlay strategy has been employed for forty or more years, resulting in flexible pavements with a total thickness of 300 mm or more. Items and costs associated with the mill and overlay strategy are shown in Table 1. Cost is estimated using the average unit prices for the August 2005 bid lettings.

Table 1. Cost/km of mill and overlay.

Item	Unit	Price ($/Unit)	Quantity	Total Cost
Asphaltic Concrete, Type SP4	Mg	25.50	1,300	$33,150.00
Performance Graded Binder (64-28)	Mg	290.00	70.2	$20,358.00
Tack Coat	L	0.35	3,350	$1,172.50
Hydrated Lime for Asphalt Mixtures	Mg	308.70	13	$4,013.10
Milling	Sta M	400.00	10	$4,000.00
Total Cost for 1 Km				$62,693.60

The mill and overlay strategy is very cost effective, which has been the primary reason for its widespread and long-term use. The costs shown in Table 1 reflect the cost of placing a 75 mm layer of 7.3 m wide asphalt over a distance of one kilometer. The principal reason for not using the mill and overlay strategy to solve flexible pavement problems under all conditions is that surface distresses often reflect stability problems originating in lower pavement layers or in the subgrade. Under these conditions, distresses in the lower layers will soon propagate upward through a new overlay to the surface. Milled and overlaid asphalt pavements with problems in the subgrade or lower pavement will generally remain in good condition for two to five years before significant deterioration in performance can be measured at the surface.

The second conventional approach, commonly used where long-term performance is more important than cost, is total reconstruction of the asphalt pavement. When using this strategy, existing pavement is removed by milling, subgrade layers are prepared to a depth of approximately 150 mm, asphalt millings are placed and stabilized as a base material and a new asphalt layer is compacted on top. Items and costs associated with the reconstruction strategy are shown in Table 2. Cost is estimated using the average unit prices for the August 2005 bid lettings.

Table 2. Cost/km of Reconstruction.

Reconstruction

Item	Unit	Price ($/Unit)	Quantity	Total Cost
Asphaltic Concrete, Type SP4	Mg	25.50	2,440	$62,220.00
Performance Graded Binder	Mg	290.00	131.76	$38,210.40
Tack Coat	L	0.35	1,680	$588.00
Hydrated Lime for Asphalt Mixtures	Mg	308.70	24.4	$7,532.28
Water	Kl	2.60	102	$265.20
Bituminous Foundation Course	m^2	1.95	7,300	$14,235.00
Milling of Existing Asphalt	m^2	2.25	7,300	$16,425.00
Subgrade Preparation	m^2	1.20	7,300	$8,760.00
Total Cost for 1 Km				$148,235.88

The principal advantage of reconstruction is that all distresses in the lower layers of the existing pavement are removed during the reconstruction process. Reconstruction results in pavement that, if properly designed, will provide high quality performance for ten or more years before significant distress becomes apparent. Table 2 itemizes the cost of placing a 137 mm layer of 7.3 m wide asphalt over 100 mm of asphalt millings base and 150mm of prepared subgrade for a distance of one kilometer.

As can be seen from Table 2, removal of problems in the lower layers of pavement and/or subgrade is not inexpensive. The cost of complete reconstruction per kilometer is more than twice the cost of mill and overlay. However, pavement life expected from reconstruction more than doubles that expected from mill and overlay, resulting in lower annual cost. Pavement engineers at the Nebraska Department of Roads have been engaged for several years in developing repair and rehabilitation strategies that can provide longer pavement life than the mill and overlay strategy without incurring the cost associated with complete reconstruction.

3. Evolving Strategies

Three evolving strategies currently being utilized on highways in Nebraska include partial depth milling with in-place recycling, full-depth milling with in-place recycling and full depth reclamation. Which strategy is more appropriate for a specific pavement depends upon a number of factors, including the condition and thickness of the existing pavement and the stability of the subgrade.

The first strategy currently being utilized by the NDOR is partial depth milling with in-place recycling of the flexible pavement. This option can be used only for highways with subgrades and/or lower asphalt layers that possess sufficient bearing capacity to resist the stress imposed by the recycling train as it passes overhead. A recycling train commonly consists of a milling unit, a screening unit and a pug mill. The recycling train mills existing asphalt and leaves behind a windrow of material that is subsequently picked up and placed by a paving machine. Pneumatic and steel wheel rollers follow the paving machine. Detailed descriptions of this pavement recycling process are available from a number of sources (Epps 1990; Wood et al. 1988).

Unstable subgrades pose a major problem for partial depth milling operations, as the recycling train is supported only by the flexible pavement remaining (after an upper layer has been milled off) and the subgrade. Subgrade stability problems caused a cold in-place recycling project to be abandoned at Pleasant Creek State Park in Iowa in 1997 (Jahren et al., 1999). The method used to define the minimum soil bearing capacity necessary to support a recycling train becomes very important. Cross and Ramaya (1995) used a dynamic cone penetrometer to determine the bearing capacity of subgrade soils for cold in-place recycling (CIR) in Kansas. Jahren et al. (1999) recommend a modified procedure using the same instrumentation based upon studies of Iowa soils.

The Nebraska Department of Roads uses a falling-weight deflectometer (FWD) to measure subgrade capacity and thus assess whether or not a subgrade can support a recycling train. FWD testing is conducted using a towed Kuab 2m-FWD. Data is recorded electronically and processed using the software provided with the instrument. Differences (in deflection) of 600 μm or less between the D_o and D_2 sensor have been found to indicate that the soil has sufficient capacity to support a paving train.

A diagram showing the pavement structure before and after partial depth, in-place recycling of flexible pavement is shown in Figure 1. Depending upon the vertical grade alignment, placing new leveling and wearing courses can increase overall pavement depth by 62-100 mm or more if no material is removed.

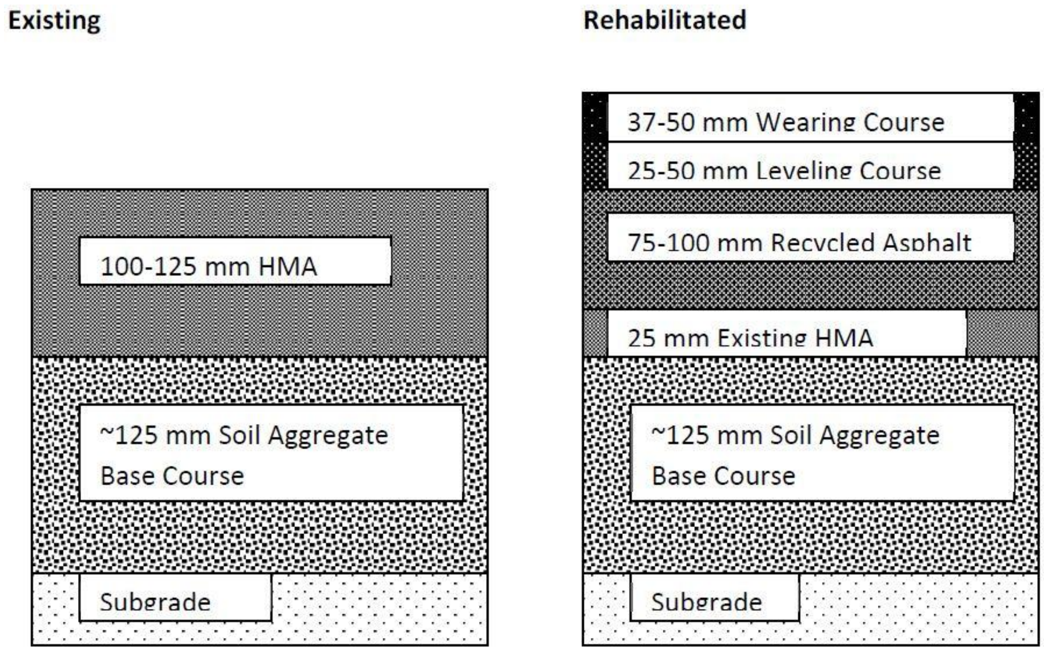


Figure 1. Typical partial depth in-place recycling pavement section

Partial depth milling with in-place recycling can be completed with minimum delays since most of the material required is already located on site. Asphalt is milled and windrowed by a milling unit, picked up from the roadway surface by a screening unit, screened, crushed if necessary, mixed with lime and/or emulsion, and deposited back on the milled asphalt surface. A paving machine then distributes the millings for subsequent compaction by a roller. The milled asphalt is often compacted to form a new surface less than an hour after first being disturbed. A section of roadway being rehabilitated during the day can often be re-opened to traffic that evening. Traffic moves across the recycled asphalt until a wearing/surface course is added seven to twenty-eight days later.

A second strategy currently used by the NDOR is full depth, in-place recycling. Full depth recycling is used primarily where the lower layers of pavement have significant distresses and/or the pavement and subgrade below have insufficient strength to support the recycling train. These conditions are defined by differences of more than 600 µm between the Do and D2 sensors on the FWD.

Reclaimers/soil stabilizers are specialized machines specifically designed for deep (150-300 mm) mixing of soil and/or aggregate. Most have the ability to incorporate solid or liquid binding agents during the mixing process. Reclaimers can be utilized to mechanically stabilize deteriorated asphalt pavement by pulverizing and mixing pavement with an asphalt emulsion or sometimes with a thin layer of soil and binder. Reclaimers are normally equipped with floatation tires or tracks to decrease surface pressure.

When working on full depth in-place recycling projects, Nebraska contractors are required to pulverize and mix all but a thin (~25 mm) layer of asphalt at the base of the pavement structure. This thin layer of asphalt is left in place as a working platform to protect the subgrade during construction. The upper surface of the recycled asphalt is normally bladed with a motor grader or other equipment capable of slope control before being compacted by pneumatic and then by steel wheel rollers. A diagram showing the pavement structure before and after full depth, in-place recycling is shown in Figure 2.

As with partial depth in-place recycling, placing new leveling and wearing courses can increase overall pavement depth by 62-100 mm or more if no material is removed.

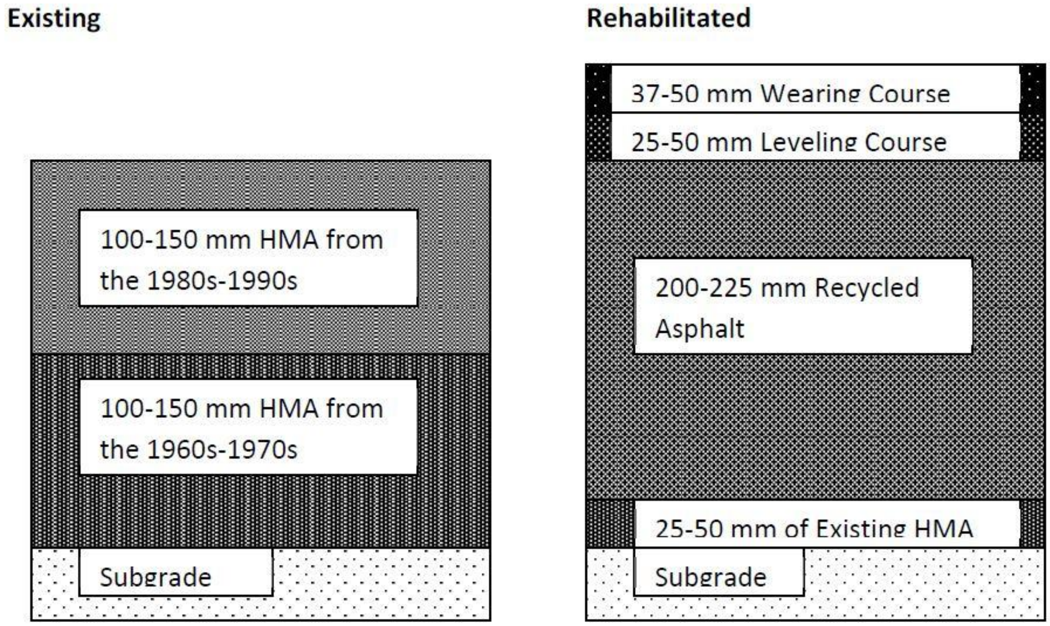


Figure 2. Typical full depth in-place recycling pavement section.

One alternative used with full depth, in-place recycling is partial depth milling (usually to a depth of 100-150 mm) followed by full depth recycling. During this process, asphalt is milled and then windrowed onto the milled surface where it is spread by a blade. Additives may be sprayed onto the milled material or added later during the pulverization/mixing process. Milled material is pulverized and mixed with the asphalt beneath using a reclaimer. Compaction is completed in a manner identical to that used for full depth, in-place recycling.

An exception to the use of full or partial depth recycling is recommended in situations where thermal cracking penetrates through the full depth of the flexible pavement layers. In this situation, subgrade material has often been weakened or even locally removed by water traveling downward through the crack. The subgrade as a platform, however, may still exhibit sufficient strength (when tested by falling weight deflectometer) to support a recycling train, so partial depth or full depth in-place recycling is often contemplated. Use of partial depth or even full depth recycling does not eliminate the zone of weakness in the vicinity of where the thermal crack exits the pavement base, which results in a reflection crack that will propagate upward to the pavement surface through the new asphalt. When full depth thermal cracking is the problem, full depth reclamation offers the only permanent solution short of pavement reconstruction.

The final strategy now being utilized is full depth reclamation. This process is used only when the subgrade and/or lower pavement offers significantly less than adequate capacity to support a recycling train or when the existing asphalt pavement has deteriorated to the point that recycling is no longer feasible. When using full depth reclamation, the objective is to thoroughly pulverize and mix all existing asphalt layers and to incorporate 25-150 mm of the underlying subgrade into this mix. Fly ash is commonly incorporated by spreading it on top of the pavement ahead of a reclaimer. Mixing cohesive soil and 8-12% fly ash with the pulverized asphalt provides a mixture that, when compacted into a subgrade and covered by pavement, will be of sufficient strength to support heavy traffic loads. A diagram showing the pavement structure before and after full depth reclamation is shown in Figure 3.

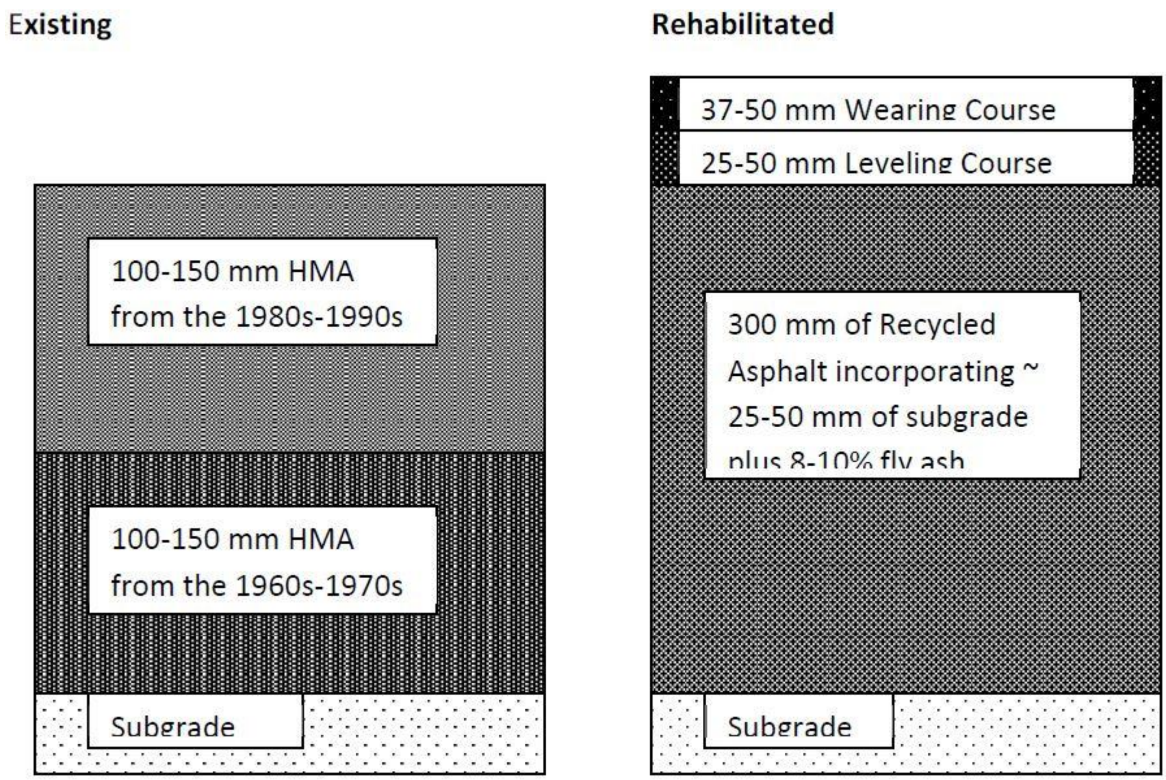


Figure 3. Typical full depth reclamation pavement section.

As with the recycling strategies described earlier, placing new leveling and wearing courses can increase overall pavement depth by 62-100 mm or more if no material is removed. Increase in pavement depth results from the 'fluff' that occurs whenever existing pavement is milled and/or pulverized and a stabilizing agent is added. The NDOR has noted that, for asphalt pavements in Nebraska, fluff typically averages about 20%. If the fluff is needed within the new pavement structure and an increase in finished pavement elevation is acceptable, fluff is simply incorporated into the finished pavement.

If fluff is not needed within the pavement structure or if an increase in grade is not acceptable, the NDOR specifies that fluff be used to widen the pavement section. There are currently two approved methods for widening pavement sections. The first method is used with partial depth strategies and involves milling a 600 mm wide by 150 mm deep trench adjacent to the outside edge of existing pavement. The NDOR specifies the trench must be milled with its outside edge vertically aligned to insure a uniform depth of asphalt at that location. The entire width of existing asphalt is milled to the appropriate depth, processed and replaced using an asphalt distributor set to a 4.27 m width for each lane (NDOR CIR Paver Layered 2006). The second method involves using a grader, with its moldboard canted to 600 mm effective width, to create a trench adjacent to the existing pavement (NDOR CIR Blade Layered 2006). This method is used primarily for full-depth reclamation due to the depth of material being processed and the fact that full depth reclamation is often completed without the use of milling equipment.

The NDOR requires contractors to meet strict specifications regarding time and sealing constraints for asphalt pavements processed using different recycling methods. Time and sealing constraints for various processes are outlined in Table 3. One final consideration is weather. The NDOR's specifications require that the air temperature remain at or above 15°C while pavement is being recycled and that work is halted when precipitation occurs.

Table 3. Time and sealing constraints for various pavement recycling strategies.

Strategy	Overlay Within	Comments
Partial Depth Using Lime and Emulsion	Twenty-eight calendar days	Contractor must wait seven days for lime to cure before overlaying. Sealing can be completed within these seven days.
Full Depth In-Place Using Emulsion	Twenty-eight calendar days	Aeration is required after initial processing to obtain uniform moisture content.
Full Depth Reclamation Using Fly-Ash	Twenty-eight calendar days	Sealing is required for curing of the Fly-ash and reducing surface raveling.
Full Depth Reclamation Using Water and Aggregate	Seven calendar days	Use on low volume roadways only. Process is susceptible to moisture damage since no stabilizing agent is used; sealing is critical.

Information in Table 3 was obtained from various sources including:
NDOR Specifications for “Cold in-place recycling-Internal liquidated damages” 2006.
NDOR Specifications for “Fly ash stabilized bituminous” 2006.
NDOR Specifications for “Fly ash stabilized bituminous-Internal liquidated damages” 2006.
NDOR Specifications for “Full depth pulverization” 2006.
NDOR Specifications for “Full depth pulverization-Internal liquidated damages” 2006.
NDOR Specifications for “Hydrated lime slurry stabilization” 2006.
NDOR Specifications for “Hydrated lime slurry stabilization-Internal liquidated damages” 2006.

4. Analysis and Results

Costs per kilometer for the mill and overlay and reconstruction strategies were shown in Tables 1 and 2. When attempting to compare the cost of conventional flexible pavement with evolving rehabilitation strategies, pavement sections with equivalent traffic capacity and expected lifespan must be analyzed for the comparison to be valid. Figure 4 shows four pavement sections designed by Robert Rea, a co-author of this paper, which are typical of the rehabilitation strategies outlined herein. Each was designed for a different project, using a specific rehabilitation strategy based upon condition and thickness of existing pavement and subgrade conditions at the site. A partial depth in-place recycling pavement section was designed for Friend to Milford (NDOR CN 12581, 2002), the full depth in-place recycling section was designed for Niobrara River N-S (NDOR CN 80648, 2003), and the full depth reclamation section was designed for Brule to Ogallala (NDOR CN 60893, 2005). The reconstructed pavement section, with its costs shown in Figure 2, was designed by Robert Rea for Ogallala-West (NDOR CN 60750, 1999). All pavement sections were designed for similar traffic loading conditions (using an almost identical structural number) and identical expected life span (ten years), so long-term performance of these pavement sections should be very similar.

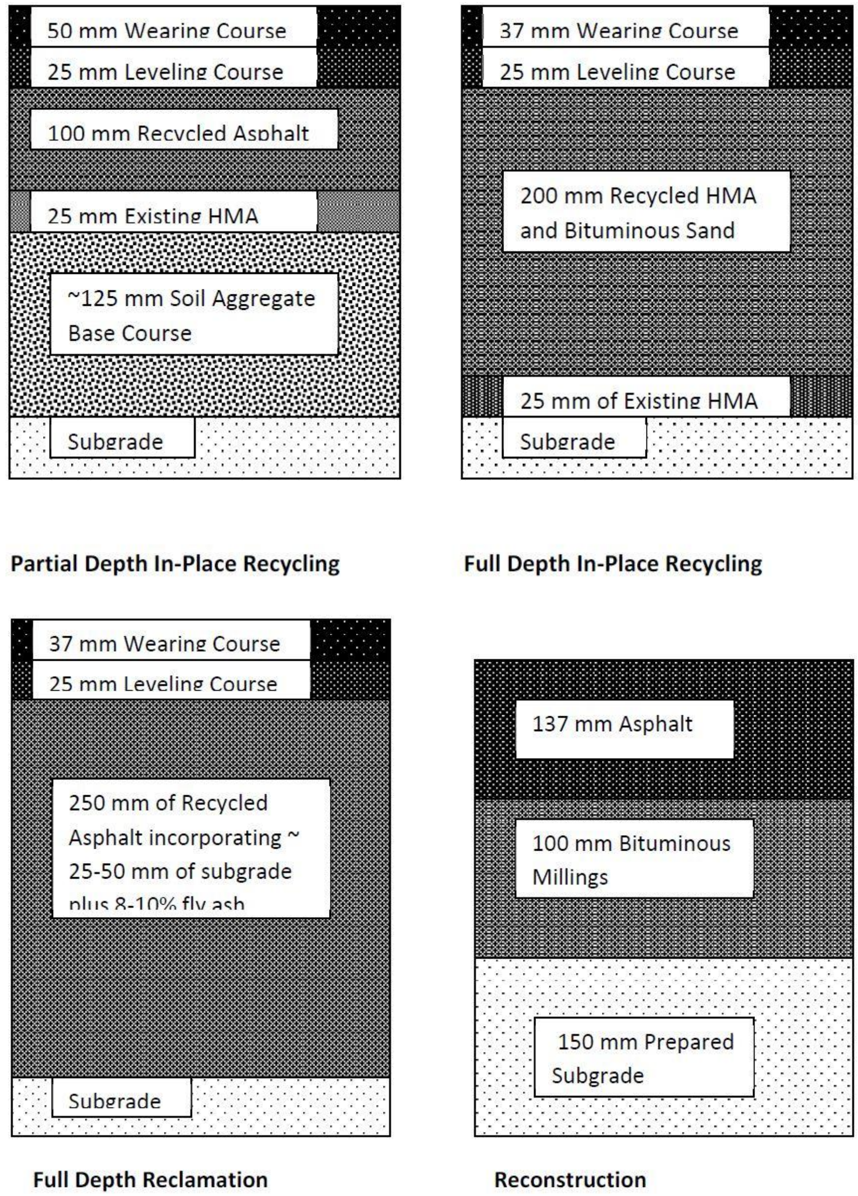


Figure 4. Equivalent sections of pavement using different rehabilitation strategies.

The pavement sections shown in Figure 4 were then transferred to the NDOR Pavement Design section, where the items and quantities of construction materials for a 7.3 m wide by one kilometer section of each pavement were calculated. Items and quantities were subsequently delivered to the Estimating Unit of the NDOR Construction Division, where each was priced out using the average unit prices for the August 2005 bid lettings. This process ensured identical costs were compiled for identical items of work on each of the five different projects. This process enabled a more accurate comparison of the relative cost for each alternative than if the actual costs from each contract had been compared. The cost for an identical section of pavement using each rehabilitation strategy in August 2005 dollars per kilometer is shown in Table 4.

Table 4

Partial Depth In-Place Recycling

Item	Unit	Price ($/Unit)	Quantity	Total Cost
Asphaltic Concrete, Type SP4	Mg	25.50	1,300	$33,150.00
Performance Graded Binder (64-28)	Mg	290.00	70.2	$20,385.00
Tack Coat	L	0.35	3,350	$1,172.50
Hydrated Lime for Asphalt Mixtures	Mg	308.70	13	$4,013.10
Hydrated Lime for Slurry Stabilization	StaM	1,476.00	10	$14,760.00
Hydrated Lime	Mg	121.30	25	$3,032.50
Emulsified Asphalt	L	0.30	42,730	$12,819.00
Total Cost for 1 Km				$89,305.10

Full Depth In-Place Recycling

Item	Unit	Price ($/Unit)	Quantity	Total Cost
Asphaltic Concrete, Type SP4	Mg	25.50	1,070	$27,285.00
Performance Graded Binder (64-28)	Mg	290.00	57.78	$16,756.20
Tack Coat	L	0.35	3,350	$1,172.50
Hydrated Lime for Asphalt Mixtures	Mg	308.70	11	$3,395.70
Cold In-Place (CIP) Recycling	StaM	820.00	10	$8,200.00
Repulverization and Aeration	StaM	525.00	10	$5,250.00
Emulsified Asphalt for CIP	L	0.25	117,950	$29,487.50
Water for CIP Recycling	Kl	2.25	70	$157.50
Total Cost for 1 Km				$91,704.40

Full Depth Reclamation

Item	Unit	Price ($/Unit)	Quantity	Total Cost
Asphaltic Concrete, Type SP4	Mg	25.50	1,070	$27,285.00
Performance Graded Binder (64-28)	Mg	290.00	57.78	$16,756.20
Tack Coat	L	0.35	3,350	$1,172.50
Hydrated Lime for Asphalt Mixtures	Mg	308.70	11	$3,395.70
Fly Ash Stabilized Bituminous	StaM	2,790.00	10	$27,900.00
Fly Ash	Mg	35.80	420	$15,036.00
Water for Fly Ash Stabilization Bit	Kl	2.25	230	$517.50
Fog Seal	L	0.32	8,030	$2,569.60
Total Cost for 1 Km				$94,632.50

Recycling material to greater depths is obviously more expensive. From the information shown in Table 4, partial depth in-place recycling is slightly more economical ($2400/km) than full depth in-place recycling. If the lifespan of full depth in-place recycled pavement exceeds that of partial depth in-place recycled pavements by even a small margin, the latter strategy will prove to be more economically advantageous. Full depth in-place recycling is correspondingly less expensive per kilometer than full depth reclamation. All pavement recycling alternatives fall between mill and overlay and complete reconstruction in terms of cost. With the full range of rehabilitation options considered, mill and overlay remains the most economical while complete reconstruction remains the most expensive strategy.

5. Conclusions

Mill and overlay is the least expensive solution ($62,693/Km), but its cost advantage is offset by its relatively short lifespan. A shortened lifespan is much more probable if pavement distresses originate in the lower levels of the pavement or in the subgrade. Partial depth in-place recycling is the next most economical solution ($89,305/Km), but it has the limitation that the subgrade and/or lower pavement layers must retain sufficient strength to support the recycling train during construction.

Full depth in-place recycling ($91,704/Km) and full depth reclamation ($94,632/Km) provide pavements that are essentially equivalent to that provided by complete reconstruction. Either option could be used in lieu of complete reconstruction ($148,236/Km), resulting in significant savings.

Long term performance of recycled asphalt pavement on medium volume roads is anticipated to be satisfactory or better. McKeen et al. (1998) report that New Mexico has completed over 120 cold, in-place recycling (CIR) projects since 1984 and condition surveys indicate that most have or will exceed their ten year design life. Sebaaly et al. (2004) report that Nevada utilized CIR for three flexible pavement projects on US-50 and US-95. All are currently performing in a satisfactory manner. Morian et al. (2004) collected data from forty-four pavement sections in Northwestern Pennsylvania rehabilitated using CIR between 1983 and 1995 and discovered that many had documented service lives up to 160% of the ten year design life typically provided by the conventional mill and overlay strategy. All recycled pavement in Nebraska appears to be performing in a satisfactory manner, showing little distress beyond what was anticipated.

The NDOR has used one of the three rehabilitation strategies on more than sixty paving projects during the past five years. These pavements are currently being monitored to document actual performance and maintenance costs throughout their lifespan. Within a few years, sufficient data will have been collected so that a life cycle cost analysis can be completed for each pavement section. Comparison of life cycle costs among the different recycling alternatives and conventional strategies will be the subject of a future paper.

An additional advantage is that the two recycling strategies and one reclamation strategy significantly reduce the quantity of materials that must be transported to (or from) the construction site. Less material transportation allows the construction process to be completed in less time, decreasing the period of time that traffic must be diverted, thereby saving pavement wear on detour routes. The three rehabilitation strategies offer alternatives that are less expensive than reconstruction but provide pavement performance that is expected to be significantly greater than mill and overlay. Since rehabilitation strategies have an impact on both cost of individual roads and the quality of the overall road network, use of these strategies is expected to increase in the future.

6. Acknowledgements

The authors would like to thank Dean Debutts from the Pavement Design Quantity Computations section at the NDOR and John Miller from the Estimating Unit of the Contracting Division at the NDOR for their assistance with the quantity and cost calculations that are included in this paper.

C.7 Historical Asphaltic Concrete Weights for Quantities

Asphaltic Concrete Tonnage Table

Asphaltic Concrete Types

	Bit Sand Base Crse	Bit Fnd Crse	OGFCCRMM	GGCRM	GGCRMLV	LC LC-S	SLX	SPS SPH	SPR SRM
Tons per 100 Cubic Feet									
	6.0	6.2	6.3	6.75	6.95	7.25	7.30	7.35	7.40
Pounds per Cubic Foot									
	120	124	126	135	139	145	146	147	148
Tons/SY/Inch									
Inches									
1	0.045	0.050	0.050	0.051	0.052	0.054	0.055	0.055	0.055
1.5									
2	0.090	0.093	0.095	0.101	0.104	0.109	0.110	0.110	0.111
2.5									
3	0.135	0.140	0.141	0.151	0.156	0.163	0.164	0.165	0.166
3.5									
4	0.180	0.186	0.189	0.202	0.208	0.218	0.219	0.221	0.222
4.5									
5	0.225	0.233	0.236	0.253	0.260	0.272	0.274	0.276	0.278
5.5									
6	0.270	0.279	0.284	0.303	0.313	0.326	0.329	0.331	0.333
6.5									
7	0.315	0.326	0.331	0.354	0.365	0.381	0.383	0.386	0.388
8	0.360	0.372	0.378	0.405	0.417	0.435	0.438	0.441	0.444
9	0.405	0.419	0.425	0.456	0.469	0.489	0.493	0.496	0.500
10	0.450	0.465	0.473	0.506	0.521	0.544	0.548	0.551	0.555

Asphaltic Concrete Megagram Table

Asphaltic Concrete Types

mm	Bit Sand Base Crse	Bit Fnd Crse	LC LC-S	SRM SLX	SPH SPS	SPR HRB
Megagrams per Cubic Meter						
	1.922	1.986	2.323	2.339	2.355	2.371
Megagram per Square Meter - Millimeter						
13	0.0250	0.0258	0.0302	0.0304	0.0306	0.0308
25	0.0481	0.0497	0.0581	0.0585	0.0589	0.0593
30	0.0577	0.0596	0.0697	0.0702	0.0707	0.0712
40	0.0770	0.0795	0.0929	0.0936	0.0942	0.0948
45	0.0865	0.0904	0.1045	0.1053	0.1060	0.1067
50	0.0962	0.0994	0.1162	0.1170	0.1178	0.1186
60	0.1154	0.1193	0.1394	0.1404	0.1414	0.1424
80	0.1539	0.1590	0.1858	0.1872	0.1885	0.1898
90	0.1732	0.1789	0.2091	0.2106	0.2120	0.2134
100	0.1924	0.1988	0.2323	0.2340	0.2356	0.2370
105	0.2018	0.2085	0.2439	0.2456	0.2473	0.2490
120	0.2309	0.2386	0.2788	0.2808	0.2827	0.2846
130	0.2501	0.2584	0.3020	0.3042	0.3063	0.3084
135	0.2595	0.2681	0.3136	0.3158	0.3179	0.3218
150	0.2886	0.2982	0.3485	0.3510	0.3534	0.3558
180	0.3463	0.3578	0.4181	0.4212	0.4241	0.4270
205	0.3940	0.4071	0.4762	0.4795	0.4878	0.4961
230	0.4425	0.4572	0.5343	0.5382	0.5419	0.5456
255	0.4901	0.5064	0.5924	0.5964	0.6005	0.6046
280	0.5387	0.5566	0.6504	0.6552	0.6597	0.6642
305	0.5862	0.6057	0.7085	0.7134	0.7183	0.7232
330	0.6343	0.6554	0.7666	0.7719	0.7772	0.7825
355	0.6823	0.7050	0.8247	0.8303	0.8360	0.8417

D.1 Area of Steel Calculation

Date: November 2004
Source: A mechanistic-Empirical tie Bar Design Approach for Concrete Pavements, http://www.acpa.org

Current Tie Bar Design Practice

The most commonly cited basis for the tie bar design in the United States is the subgrade drag theory, or SDT. This theory, which has its origins in the design of steel reinforcement for slab-on-grade concrete flooring, is explained in several textbooks and industry references (Yoder & Witczak, 1975; Huang, 1993; PCA 2008). The SDT method of design is based on the concept of providing sufficient steel to allow the "dragging" of the concrete slab across the base course without yielding the steel or pulling out the tie bars. The basic concepts are as follows:

1. The maximum force a tie bar can sustain without yielding, F_{TB} (lb), is expressed as:

$$F_{TB} = a_s * f_s \tag{1a}$$

where
a_s = cross-sectional area of one tie bar, in^2
f_s = the allowable stress in steel f_s, lb/in^2
(usually taken as two-thirds of the yield strength)

2. The force to drag a concrete slab across the base course, F_{drag}(lb), is computed as:

$$F_{drag} = L_{slab} D_{fc} H_{PCC} W F \tag{1b}$$

where
L_{slab} = slab length, in
D_{fc} = distance to the closest free edge, in
H_{PCC} = concrete slab thickness, in
W = unit weight of concrete, lb/in^3
(approximately 0.0868 lbs/in^3 for a typical paving concrete mixture)
F = coefficient of friction at the slab-base interface
(e.g., a value of 1.5 for unbound bases is recommended by the 1993 AASHTO *Guide for Design of Pavement Structures* or simply the 1993 AASHTO Guide)

3. If n is taken as the number of tie bars per slab length, then the equation of equilibrium of the SDT is:

$$nF_{TB} = F_{drag} \tag{2a}$$

4. From equation (2a), the total area of steel per slab can be determined as:

$$na_s = A_s = \frac{L_{slab} D_{fc} H_{PCC} W F}{f_s} \tag{2b}$$

where
A_s (in^2) = total area of steel for a given slab length.

5. The required tie bar spacing, J_{TB} (in), can be determined as:

$$J_{TB} = \frac{L_{slab} a_s}{A_s} \tag{2c}$$

Example Application of the SDT Method for Tie Bar Design

Problem Statement

Compute the total area of steel (A_s) required for a 12-in concrete slab with a 15-ft transverse joint spacing over an unbound base for a highway consisting of two 12-ft lanes tied at the centerline joint. Assume Grade 60 steel (steel yield strength, f_y = 60,000 psi). Also compute the tie bar spacing required for #4 deformed bars. Assume the unbound coefficient of friction to be 1.5.

Solution

Area of steel calculation (variables previously defined):

$$A_s = L_{slab} * D_{fc} * H_{PCC} * W * F / (f_y * 2/3)$$

$$A_s = (180\ in * 144\ in * 12\ in * 0.0868\ lb/in^3 * 1.5) / (60{,}000\ psi * 2/3)$$

$$A_s = 1.01\ in^2$$

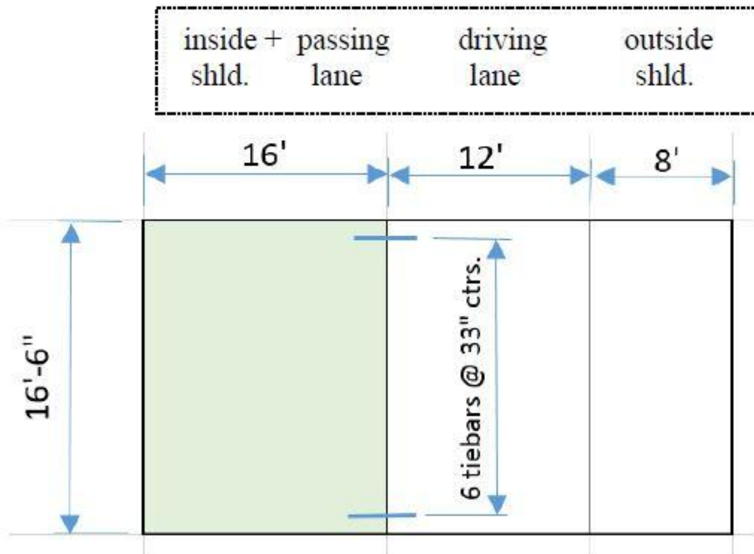
Tie bar spacing calculation:

The area of a #4 bar is 0.20 in^2. Thus, and using equation (2c), a maximum spacing of 35.6 in should be used to drag the concrete slab over the unbound base course and while keeping the steel stress safely below its yield stress. This could practically translate to providing 6 #4 bars per slab at a 30 in spacing.

D.2 Area of Steel Calculations for Interstate and Expressway

Date: 2018
By: Barrett

A_s	=	Area of Steel = $(L_{slab} * D_{fc} * H_{PCC} * W * F) / (f_y * 2/3)$
L_{slab}	=	Length, in
D_{fc}	=	Closest Width to Free Edge, in
H_{PCC}	=	Thickness, in
W	=	Unit Weight, lbs/in^3
F	=	Coefficient of Friction, 1.5 for unbound
f_y	=	Yield Strength of Steel, psi
η	=	Number of Tie Bars per Slab Length



L_{slab}	=	16'-6" = 198"
w_{slab}	=	16'-0" = 192"
t_{slab}	=	12" (> 10", use #6 bars)
a_s	=	Actual Bar Area, #6 bar = $\pi r^2 = \pi((3/4")/2)^2 = 0.4418$ in^2
W	=	0.0868 lbs/in^3
F	=	1.5
f_y	=	40,000 psi

$A_s = (L_{slab} * D_{fc} * H_{PCC} * W * F) / (f_y * 2/3)$
= (198" ∗ 192" ∗ 12" ∗ 0.0868 lbs/in^3 ∗ 1.5) / (40,000 psi ∗ 2/3)
= 2.23 in^2

η = A_s / a_s
= 2.23 in^2 / 0.4418 in^2
= 5.05 bars → NDOT uses 6 bars

$J_{TH} = (L_{slab} * a_s) / A_s$
= (198" ∗ 0.4418 in^2) / 2.23 in^2
= 39.3" spacing → NDOT uses 33" spacing

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- **Image – Frost Damage**
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- **Figure 5.3-1 – Ideal Grain Size Distribution for Surface-Course and Base-Course Aggregate**
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- **Figure 5.4-1 – Resilient Modulus Plot of a Nebraska Material**
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- **Figure 5.4-2 – Summary Resilient Modulus Plot of a Nebraska Material**
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- **Figure 5.4-4 – Hardin's Concept to Evaluate the Degradation of Aggregates (Hardin, 1985)**
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    - Figure 5.6-5 – Output Values for Gravel Road Enhancement Gradation Optimization Tool

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- **Image – Asphalt Pavement Milling**
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- **Image – Seating Concrete Panels**
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Chapter 9: Material Costs and Quantity Estimates

The information contained in Chapter 9 of the Pavement Design Guide has been provided by the Nebraska Department of Transportation for informational purposes.

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