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Gravel Road Performance Enhancement

Bora Cetin, PhD
Professor
Department of Civil and Environmental Engineering
Michigan State University

Kristen Cetin, PhD
Associate Professor

Charan Sai Vangaveeti
Graduate Research Assistant

Mehdi Bulduk
Graduate Research Assistant

Md Mahir Asif
Graduate Research Assistant

Nebraska Department of Transportation Research
Headquarters Address
1400 Nebraska Parkway
Lincoln, NE 68509
ndot.research@nebraska.gov
(402) 479-4697
https://dot.nebraska.gov/business-center/research/

Nebraska Transportation Center
262 Prem S. Paul Research Center at Whittier School
2200 Vine Street
Lincoln, NE 68583-0851
(402) 472-1932
http://ntc.unl.edu

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Gravel Road Performance Enhancements

Bora Cetin, PhD
Professor
Department of Civil and Environmental Engineering
Michigan State University

Kristen Cetin, PhD
Associate Professor
Department of Civil and Environmental Engineering
Michigan State University

Charan Sai Vangaveeti
Graduate Research Assistant
Department of Civil and Environmental Engineering
Michigan State University

Mehdi Bulduk
Graduate Research Assistant
Department of Civil and Environmental Engineering
Michigan State University

Md Mahir Asif
Graduate Research Assistant
Department of Civil and Environmental Engineering
Michigan State University

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16. Abstract

Nearly 30% of roads in the U.S. are unpaved, significantly impacting rural connectivity with Nebraska alone having approximately 75% unpaved roads. This study systematically evaluates local materials to improve gravel road performance, reduce maintenance frequency, and decrease financial burdens on counties. Survey responses revealed that nearly 70% of counties follow NDOT specifications for gravel road design, while more than 90% lack knowledge of local material quality. The most common distresses identified were raveling, loss of crown, dust, and improper drainage. These findings indicate that construction practices rely heavily on experience, rather than systematic design, highlighting the need for a performance-based approach. To address this, commonly used gravel road materials were identified. Seventeen different materials—13 surface materials and 4 subgrade soils—were collected from four counties for laboratory evaluation. Comprehensive testing assessed index properties, while repeated triaxial tests determined the mechanical behavior of granular surface materials. Maximum dry unit weight (MDU) values ranged from 111 to 138 pcf, with corresponding optimum moisture content (OMC) values between 3% and 11.3%. Resilient modulus (M_R) tests showed inconsistent results, ranging from 10 ksi for sand-dominant materials to 30 ksi for open-graded materials. Virgin materials performed poorly in permanent deformation (PD) tests and were deemed unsuitable for road surfaces due to constructability, maintenance, and drainage limitations. A granular stabilization technique was implemented by mixing additional granular materials (e.g., gravel, crusher run, and fine-grained soils) to correct deficiencies in particle size distribution (PSD), shape, and plasticity. After extensive trials, 31 optimized blends were proposed. These blends exhibited increased dry unit weights, averaging 134 pcf, and consistent stiffness, with M_R values ranging from 8 ksi to 27 ksi, predominantly at the higher end. Additionally, they showed nearly a 100% improvement in permanent strain accumulation across most blends. Performance enhancements were attributed to increased mechanical interlock, inter-particle friction, and binding, as well as reduced aggregate breakdown. Exposure to freeze-thaw cycles had no effect on M_R but increased PD, indicating a reduction in material stability under repeated cycles. Stepwise regression models were developed to predict M_R and PD directly from index properties such as particle size distribution, specific gravity, and plasticity. An Excel-based gradation optimization tool was developed to determine the required proportions of existing and fresh aggregates for an optimized gradation range. This tool incorporates road geometry and material characteristics to provide precise material quantities, ensuring improved consistency and performance in gravel road construction and maintenance.

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List of Abbreviations

American Association of State Highway and Transportation Officials (AASHTO)

American Society for Testing and Materials (ASTM)

Federal Highway Administration (FHWA)

Unified Soil Classification System (USCS)

Mechanistic-Empirical Pavement Design Guide (MEPDG)

Particle Size Distribution (PSD)

Specific Gravity (G_S)

Saturated Surface Dry (SSD)

Oven Dry (OD)

Liquid Limit (LL)

Plastic Limit (PL)

Plasticity Index (PI)

Maximum Dry Unit weight (MDU)

Optimum Moisture Content (OMC)

National Cooperative Highway Research Program (NCHRP)

California Bearing Ratio (CBR)

Resilient Modulus (M_R)

Summary Resilient Modulus (SM_R)

Linear Variable Differential Transformer (LVDT)

Permanent Deformation (PD)

The Technique for Order of Preference by Similarity to Ideal Solution (TOPSIS)

Multi-Criteria Decision-Making (MCDM)

Freeze and Thaw (FT)

Zero cycles of Freezing and Thawing (0FT)

Five cycles of Freezing and Thawing (5FT)

Falling Weight Deflectometer (FWD)

Light Weight Deflectometer (LWD)

Dynamic Cone Penetrometer (DCP)

Recycled Asphalt Pavement (RAP)

Recycled Concrete Aggregate (RCA)

pounds per cubic feet (pcf)

kilo pounds per square inch (ksi)

mega pascal (MPa)

kilo pascal (kPa)

centimeter (cm)

millimeter (mm)

inches (in)

feet (ft)

Celsius (°C)

Second (sec)

Kilogram (kg)

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Disclaimer

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Abstract

Nearly 30% of roads in the U.S. are unpaved, significantly impacting rural connectivity with Nebraska alone having approximately 75% unpaved roads. This study systematically evaluates local materials to improve gravel road performance, reduce maintenance frequency, and decrease financial burdens on counties. Survey responses revealed that nearly 70% of counties follow NDOT specifications for gravel road design, while more than 90% lack knowledge of local material quality. The most common distresses identified were raveling, loss of crown, dust, and improper drainage. These findings indicate that construction practices rely heavily on experience, rather than systematic design, highlighting the need for a performance-based approach. To address this, commonly used gravel road materials were identified. Seventeen different materials—13 surface materials and 4 subgrade soils—were collected from four counties for laboratory evaluation. Comprehensive testing assessed index properties, while repeated triaxial tests determined the mechanical behavior of granular surface materials. Maximum dry unit weight (MDU) values ranged from 111 to 138 pcf, with corresponding optimum moisture content (OMC) values between 3% and 11.3%. Resilient modulus (M_R) tests showed inconsistent results, ranging from 10 ksi for sand-dominant materials to 30 ksi for open-graded materials. Virgin materials performed poorly in permanent deformation (PD) tests and were deemed unsuitable for road surfaces due to constructability, maintenance, and drainage limitations. A granular stabilization technique was implemented by mixing additional granular materials (e.g., gravel, crusher run, and fine-grained soils) to correct deficiencies in particle size distribution (PSD), shape, and plasticity. After extensive trials, 31 optimized blends were proposed. These blends exhibited increased dry unit weights, averaging 134 pcf, and consistent stiffness, with M_R values ranging from 8 ksi to 27 ksi, predominantly at the higher end.

Additionally, they showed nearly a 100% improvement in permanent strain accumulation across most blends. Performance enhancements were attributed to increased mechanical interlock, inter-particle friction, and binding, as well as reduced aggregate breakdown. Exposure to freeze-thaw cycles had no effect on M_R but increased PD, indicating a reduction in material stability under repeated cycles. Stepwise regression models were developed to predict M_R and PD directly from index properties such as particle size distribution, specific gravity, and plasticity. An Excel-based gradation optimization tool was developed to determine the required proportions of existing and fresh aggregates for an optimized gradation range. This tool incorporates road geometry and material characteristics to provide precise material quantities, ensuring improved consistency and performance in gravel road construction and maintenance.

Executive Summary

Gravel roads play an important role in rural transportation, but their performance and longevity depend heavily on material quality and proper gradation. Common surface distresses such as washboarding, rutting, erosion, potholes, and material loss are often linked to poor aggregate selection and inadequate gradation control. The performance of unpaved roads is influenced by multiple factors, including material properties, drainage, climate conditions, traffic loads, and maintenance practices. In Nebraska, where approximately 75% of roads are unpaved, maintaining the gravel road network is a challenge due to material variability and the reliance on empirical design practices. The goal of this project was to improve gravel road performance by systematically evaluating local materials and developing an optimization tool for better gradation control. The primary objectives included: (1) assess current practices in gravel road construction and stabilization through surveys and interviews, (2) identify and collect commonly used surface and subgrade materials from various regions of Nebraska, (3) conduct extensive laboratory testing to evaluate geomechanical properties and performance characteristics, (4) implement granular stabilization by blending different materials to optimize gradation and plasticity, and (5) develop a data-driven gradation optimization tool to enhance material selection and improve road durability. A comprehensive literature review summarized the causes of various common gravel road distresses, stabilization techniques, and the mechanical behaviors of geomaterials. The study also examined the effects of F-T cycles on deformation characteristics and identified a gap in optimizing gradation based on performance characteristics.

A survey of Nebraska counties, based on 26 responses, revealed a strong reliance on experience-based methods, with a lack of standardized technical guidelines for material selection and stabilization.

The most common gravel road distresses reported were washboarding, rutting, poor drainage, dust, and frost-related damages. More than half of the counties did not prioritize material quality and performance in road construction. High-quality gravel shortages were widespread. The most common materials were crusher run (18%), pea gravel (16%), crushed stone (14%), and recycled concrete aggregate (10%). Additionally, 52% of respondents reported incorporating 10% to 20% fines in granular road surfaces, while 67% followed Nebraska DOT gradation specifications. Notably, 70% of respondents supported modifying existing material specifications to improve local sourcing and enhance road design. To address these challenges, commonly used granular surface and subgrade materials were collected from four counties in Nebraska. Seventeen materials—including well-graded, open-graded, and poorly graded aggregates with varying sand and silt compositions—were analyzed. Laboratory testing assessed key geomechanical properties, including M_R, PD, and F-T resistance, to evaluate the structural integrity of virgin and blended materials. Laboratory test results showed that well-graded and open-graded materials exhibited higher stiffness and lower PD, while materials with a low gravel-to-sand ratio had significantly lower M_R and higher PD. Although open-graded materials had high stiffness, permeability, and stability under confinement, constructability challenges limited their field use. As expected, subgrade materials had lower MR and higher PD. Blended materials demonstrated improved MR and better PD resistance, indicating engineered gradation with an appropriate amount of fines can significantly enhance gravel road performance. Stepwise regression models were developed to establish statistical correlations between index properties (e.g., particle size distribution, plasticity, and specific gravity) and mechanical behavior, specifically predicting M_R and PD. These statistical models provided a foundation for optimizing material blends. Based on laboratory test results and statistical analysis, an Excel-based gradation optimization tool was developed to calculate the required proportions of existing materials and fresh aggregates needed to achieve an optimal gradation range. This range, defined through statistical analysis and an extensive literature review, was set with upper and lower limits to maximize M_R while minimizing PD. The tool allows users to input road geometry and material characteristics, providing recommended material quantities for achieving target gradation, ultimately improving road performance.

The findings of this study contribute to improving material selection, design, and construction practices for gravel roads, enhancing the durability and resilience of rural transportation infrastructure. This work provides a foundation for data-driven, performance-based specifications, offering practical solutions for long-term, cost-effective gravel road maintenance and construction.

Chapter 1 Introduction

1.1 Background

According to the Federal Highway Administration (FHWA), in 2020, the United States had 1,317,000 miles of unpaved roads. These roads represented nearly 30 percent of the nation's total roadway network, which spans over four million miles (FHWA, 2020). Granular surface roads are a common form of transportation infrastructure worldwide. Nebraska has an extensive network of unpaved roads, making up about 75% of its total road network and spanning 75,000 miles. These roads are crucial in rural and agricultural areas, providing essential access to farm-to-market routes, schools, and emergency services. Consequently, the viability of these routes is significant to the rural economy.

The quality of gravel road materials, including abrasion resistance and freeze-thaw durability, plays a crucial role in road performance. Common surface deteriorations such as material loss, gradation changes, crown loss, surface erosion, rutting, washboarding, and potholes are directly influenced by the material properties used in road construction (Burningham & Stankevich, 2005; Mahedi et al., 2019; Satvati et al., 2020). These deteriorations are particularly pronounced when low-quality aggregates and improper gravel surface gradation are used, often leading to severe rutting and washboarding issues on gravel roadways (Alam-Khan et al., 2021; C. Li et al., 2018; Satvati et al., 2019; Wu et al., 2020; Xue et al., 2021). Studies have shown that the breakage of low-quality aggregates accelerates deterioration, leading to high maintenance costs (Cetin et al., 2019).

The properties of surface materials, particularly their gradation, plasticity, and particle interlock, heavily influence the performance of unpaved roads. Standardized, performance-based design methods for selecting and blending materials for unpaved road surfaces are still lacking.

While NDOT and other state DOTs provide gradation guidelines for base and subbase layers of paved roads, they are not specifically designed for gravel roads. As a result, these guidelines require local calibration and updates to improve gravel road performance and maintenance. Currently, NDOT does not have a dedicated guideline for optimizing the gradation and performance of Nebraska's gravel roads. Additionally, existing practices for mitigating gravel road deterioration have not been systematically compiled or evaluated.

1.2 Problem Statement

The road network of Nebraska spans approximately 194,938 miles, including 72,134 miles of gravel roads (39 miles of which are state highways). These roads play a critical role in the state's rural economy, providing access to agricultural lands and enabling the transportation of goods. However, these roads are highly susceptible to rutting and washboarding, two common forms of deterioration. Rutting occurs when moisture levels are high, leading to surface deformation under repeated traffic loads. Washboarding develops in areas with frequent braking or acceleration and on roads without adequate cross slopes. Seasonal FT cycles and increasing traffic volumes accelerate these issues, requiring frequent maintenance and rehabilitation.

Given the substantial investment counties make in gravel road maintenance, this research project focused on developing cost-effective and sustainable solutions that enhance roadway performance and reduce maintenance frequency. Various stabilization and construction techniques were evaluated, including crushed rock embedment, gradation optimization, and the addition of angular materials and waste fines. These methods were expected to enhance structural integrity, minimize surface deformation, and extend service life.

To assess the effectiveness of these treatments, a comprehensive laboratory study was conducted using gravel road materials and subgrades collected from multiple locations in

Nebraska with significant road distress. The study analyzed index properties, resilient modulus, abrasion resistance, permanent deformation behavior, and FT durability. By quantifying the impact of different stabilization techniques, the research provided useful knowledge for optimizing Nebraska Department of Transportation (NDOT) gradation specifications for gravel surfacing using locally sourced materials.

Additionally, a data-driven predictive model was developed to estimate the type and quantity of materials needed for gravel road improvements. This model incorporated simple index properties and mechanical performance data to guide material selection and stabilization strategies. The findings offer a practical framework for improving gravel road sustainability while lowering long-term maintenance costs, ultimately benefiting Nebraska's transportation infrastructure and rural economy.

1.3 Objectives

The primary objective of this project was to establish comprehensive guidelines for a diverse range of Nebraska-specific gravel road materials through rigorous laboratory testing. This was done by completing the following objectives.

- Review and analyze existing practices among state Departments of Transportation (DOTs) and industry, with a focus on Midwest states. This was achieved with a combination of detailed reports, surveys, and interviews with industry professionals to identify best practices, challenges, and areas for improvement in gravel road maintenance and stabilization.
- Conduct an extensive laboratory study to examine the impact of gradation and plasticity variations on the geomechanical behavior of gravel road materials. The study aimed to

determine the optimal gradation and plasticity ranges that enhance strength, durability, and resistance to FT cycles, ultimately improving road performance and longevity.

- Develop an Excel-based gradation optimization tool that utilizes laboratory-derived index properties to determine the optimal proportions of fresh aggregate, fines, and stabilizer materials. This tool enables engineers and agencies to make data-driven decisions, ensuring an ideal material blend that enhances gravel road performance by improving strength, stability, and reducing maintenance needs.

1.4 Research Plan

The research study was divided into six key tasks to improve gravel roads performance and establish a new performance-based guideline for the state of Nebraska. The proposed study provided background knowledge for the efficient use of gravel materials and improved the ease of choosing optimized material characteristics.

Task 1: Project kickoff meeting,

The research team met with the project's technical advisory committee (TAC) to review the project scope, work plan, schedule, and expected deliverables.

Task 2: Literature Review and Survey

A thorough literature review was conducted to compile data and methodologies from studies on long-term gravel road distress, the influence of material index properties, various stabilization techniques, and geomechanical testing. These insights formed the foundation for improving gravel road performance and durability.

An online survey was conducted with county and city engineers across Nebraska using a web-based platform. The survey aimed to assess current practices, material sources, maintenance challenges, and design considerations for gravel road infrastructure. The collected responses were compiled into a database, critically analyzed, and documented.

Task 3: Sample Collection

Based on survey responses, commonly used surface and subgrade materials were collected from four different counties across Nebraska, representing a diverse range of material origins and physical properties. A total of 13 surface materials and four subgrades were selected, including open-graded materials, crusher run, road gravel, sand-dominated mixtures, and fine-grained materials with clay and silt content. These materials were obtained from gravel pits, county department storage sites, and road sections before being shipped to the testing laboratory.

Task 4: Laboratory testing

Laboratory testing was a key part of this study, aimed at evaluating the geomechanical behavior of collected materials to improve gravel road design and performance. Virgin materials were first tested to determine their physical properties, followed by mechanical behavior assessments. Among available stabilization techniques, gradation stabilization was selected due to its cost-effectiveness, ease of implementation, and potential for improving material performance. The study began with a gradation range recommended in the literature as a reference for material blending. Blended materials were developed while ensuring their gradations remained within these recommended limits. These blends were then tested to assess their mechanical behavior and refine the gradation range further. The goal of the study was to identify the gradation ranges that perform best under local conditions by evaluating resilient modulus, permanent deformation, and FT resistance. The results provided a region-specific gradation guideline to improve stiffness, durability, and resistance to long-term deformation.

Task 5: Data analysis, development of gradation tool

Statistical analysis was conducted to examine the relationships between index properties and material performance. Stepwise regression models were developed to predict M_R and PD based on key material characteristics. These findings were used to refine gradation limits and optimize material blends. Based on the analysis, an Excel-based gradation optimization tool was developed to determine the required proportions of existing and fresh materials to achieve a balanced gradation. This tool provides engineers with a data-driven approach for selecting gradations that enhance the stiffness, durability, and long-term performance of gravel roads.

Task 6: Preparation of final report

A detailed report was prepared to document the findings and provide recommendations to the Nebraska Department of Transportation (NDOT).

1.5 Research Benefits

The results from this study are expected to improve the performance, economics, and service lifespan of gravel surface roads in the state of Nebraska. The results and recommendations of this project provide a foundation to find the optimum gradation and material properties for the county engineers. This research aligns with the Nebraska DOT focus areas of environmental stewardship. The maintenance of gravel roads can consume significant portions of county budgets. The benefits of this research are summarized below:

- Results obtained from the laboratory study will better estimate the field performance of granular road surfaces since it will consider field conditions during testing such as environmental changes and material behavior under traffic loadings.

- The optimized gradation obtained from the user-friendly tool will allow the county engineers to source good quality local materials, which will reduce major distresses and improve the service life of granular road surfaces.
- Future implementation benefits of this research include: (1) reduction of engineering costs through appropriate selection of treatment methods, (2) construction savings through material reduction and natural resource conservation resulting in reduced environmental impacts, (3) longer gravel road service life resulting in a reduced life cycle cost, and (4) reduced operation and maintenance costs.

Overall, the long-term benefits of this research study will be improving the quality, longevity, and state of good repair of Nebraska gravel roads, which constitute a vital component of Nebraska's infrastructure.

1.6 Literature Review

This section provides a literature review focusing on (1) long-term studies conducted on gravel road surface distress, the causes and solutions for reducing these distresses, (2) a review of materials index properties influence on gravel road surface materials, (3) other previous studies on various stabilization techniques employed to improve gravel roads suffering from various distresses, and (4) a review of various laboratory tests conducted to obtain knowledge on geomechanical properties of gravel surface materials.

1.6.1 Long-term studies conducted on gravel road surface distresses.

The primary causes of distress include high traffic loads, seasonal FT cycles, loss of aggregate, and the use of substandard materials. It is markedly important to identify the types, severity, and extent of the frequently ensuing distresses. Additionally, the function and reasons for these distresses are required to be studied thoroughly. As part of this, the research team investigated the frequently occurring distresses on granular surfaced roads in Nebraska and surrounding states and recorded their functions and underlying causes.

Washboarding (corrugation) refers to the undulations that typically extend across the entire width of the granular road and run perpendicular to the traffic stream (Allen & Banash, 2001). These undulations form a pattern of valleys and ridges, as shown in Figure 1.1, with the difference between them referred to as wave height. Ridge heights can range from a few millimeters to 10-20 centimeters (Alzubaidi & Magnusson, 2002). Washboarding reduces the tire–road surface contact area (Matsuyama et al., 2020), compromising driving safety, reducing vehicle speed, and lowering fuel efficiency.

Figure 1.1 Poor riding quality due to washboarding (D. Jones, 2017)

The most common cause of this distress is the presence of excessive sandy materials along with an insufficient amount of plastic fines on the granular surfaced roads. Minimal cohesion among road surface materials also plays a significant role. Additionally, the loss of fines due to high vehicle speeds, continuous change of speed, sudden acceleration, or braking exacerbates this issue (Allen & Banash, 2001).

Washboarding tends to develop in roadways lacking moisture, where the surface crust is dry & brittle. Since the crust that forms on the surface of a good gravel road seems to disintegrate in dry weather. The crust that forms on a well-maintained gravel road tends to break apart in dry weather, allowing the underlying stone and sand-sized particles to loosen or float. Under traffic, these loose materials gradually organize into the characteristic washboard pattern (FHWA, 2015)

Another major distress is the loss of crown, which occurs when a road's surface flattens, as shown in Figure 1.2. A road's crown is the slightly elevated, curved contour of the road surface that allows water to drain off to the sides of the road. According to FHWA and other research studies, a recommended crown slope is 1/2 inch per foot or 4% - 6% on the cross slope. If the crown is insufficient, water accumulates in the center of the road. Conversely, if the crown is too high, drivers begin to lose control as their vehicle begins to drift toward the shoulder (FHWA, 2015; Persson, 1993; Srombom, 1987).

Figure 1.2 Loss of crown (WTTC, 2014)

Crown loss leads to depressions on the road surface. Water quickly collects in these depressions and softens the surface. As vehicles pass over these areas, tires push aggregate outward, forming larger ridges and deepening the depressions (FHWA, 2015). If left unaddressed, a reverse crown may develop, further impeding drainage. This type of distress primarily occurs due to excessive traffic load, surface wear, and low bearing capacity of granular surfaced roads (Alzubaidi & Magnusson, 2002).

Dust in the air is fine particulate (smaller than 0.075 mm) expelled from the road surface and transmitted to the air (Nervis & Nuñez, 2019). The slipstream from passing vehicles stirs up these fine particles, creating dust clouds that disperse to the sides of the roadway (Alzubaidi & Magnusson, 2002). Excessive dust in the air reduces the visibility for drivers as demonstrated in Figure 1.3, increasing the risk of accidents. Dust problems are also detrimental to the environment, human health, and nearby crops (Paige-Green, 1989).

On gravel roads that can accommodate up to 100 vehicles daily, traveling at speeds of 75 km/h, an annual loss of up to 25 tons of gravel-wearing coarse aggregate can occur per kilometer (T. E. Jones, 1984). This results in an approximate 4 mm reduction in road thickness for a road 7 m in width. The loss of aggregate material necessitates frequent maintenance and aggregate replacement, leading to increased costs (Persson, 1993). As a result, dusting is therefore one of the primary deterioration mechanisms of a gravel road and will be discussed in more detail in subsequent sections.

Figure 1.3 Unacceptable levels of dust (Oransi, 2020)

A drainage problem on a granular surfaced road occurs when water is not properly routed or removed from the road surface as displayed in Figure 1.4. Poor drainage can lead to various other distresses. The more irregular the road, the slower the water drains. As a result, the crossfall on a gravel road must be significantly steeper than on a paved road. Crown deteriorates due to vehicle overloading, excessive wearing, and inadequate bearing capacity of roads. Additionally, high shoulders, formed from displaced aggregate and soil, prevent proper drainage. Instead of flowing into ditches, water is obstructed by these elevated shoulders and remains on the roadway.

Figure 1.4 Impassible gravel road due to poor drainage (Saha & Ksaibati, 2017)

Water accumulating within the road structure reduces its bearing capacity (Hubendick P, 1969). Additionally, vegetation growth causes issues by retaining moisture and softening the roadway when it extends from the shoulder into the primary road area (Chong & Wrong, 1989). Improper drainage is also evident in collapsed or debris-filled culverts, which further disrupt water flow.

Rutting is defined as a depression in the wheel path parallel to the midline of the roadway as shown in Figure 1.5. Rutting increases fuel consumption and the risk of skidding. Minor rutting (less than 1 in.) may simply indicate heavy traffic volume and can typically be addressed through routine regrading and surface drainage maintenance. However, deeper rutting (over 3 in.) suggests insufficient gravel thickness or inadequate subgrade support. This defect is very serious and usually indicates that major reconstruction is required. Ruts also allow water to penetrate the pavement instead of draining off, accelerating deterioration. As a result, the bearing capacity of granular surface roads is significantly reduced. Rutting also diminishes the rideability or smoothness. Proper compaction can help mitigate rutting by reducing the extent of surface deformation.

Figure 1.5 Severe rutting damage (ROADEX, 2014)

Frost damage contains frost heave, thaw softening, frost boils, and stone migration (Persson, 1993). The primary cause of frost heave is water migrating upward into the frost zone and freezing into ice. The majority of heave on unpaved roads is vertical since it is the direction of least resistance. The formation of ice layers beneath the surface depends on the drainage quality and the groundwater table height. The entire rise may range from 20 to 50 centimeters (VFW, 1946). During the thawing period, surface softening develops as frost prevents proper drainage, causing excess water to concentrate in the upper layers. This issue is worsened by heavy traffic load, poorly designed ditches, and inadequate culverts. Frost boils typically occur in the later stages of thawing when ice layers within the road melt.

If these layers are thick, significant amounts of water are released, leading to water pumping, where fine particles are forced to the surface under heavy traffic loads. Stone migration is another frost-related phenomenon caused by ice forming around stones. As the ice expands, stones are pushed upward. When the ice melts, the voids left behind fill with fine particles, preventing the stones from settling back into place. The extent of stone migration depends on subgrade and pavement frost susceptibility, groundwater levels, and temperature fluctuations. (Persson, 1993).

Figure 1.6 Seasonal damages due to freezing-thawing cycles (Jennifer DeWitt, 2019)

Potholes are bowl-shaped depressions in the road surface that can form individually or in clusters and frequently coexist with other pavement distresses as shown in Figure 1.7 (Allen & Banash, 2001). They typically range from 0.5 cm in diameter and 3 to 7 cm in depth.

Potholes primarily develop in areas where the subgrade is saturated. As vehicles pass over these weakened sections, the subgrade fails, leading to progressive surface deformations. They expand more rapidly when water accumulates inside the hole. The road continues to deteriorate due to surface material loss or the formation of weak patches in the subsurface soil (Alzubaidi & Magnusson, 2002). Potholes can also occur due to insufficient crown, usage of substandard material, and design flaws.

Figure 1.7 Formation of Potholes (KC Morgan, 2023)

These damages can be mitigated through various techniques, including improving gradation, applying stabilization methods, using high-quality materials, and ensuring adequate pavement layer thickness.

1.6.2 Review on the importance of index properties.

Particle-size distribution (PSD) plays a crucial role in determining the strength of gravel. It achieves this through the interlocking of particles and the application of the maximum density principle. Additionally, PSD influences the material's permeability, particularly the percentage of particles smaller than 0.5 mm. The importance of the index properties of gravel road surface materials such as maximum aggregate size, gradation, plasticity, and quality has long been recognized (Hudson et al., 1986; D. Jones, 2015; Paige-Green, 1998; Skorseth, 2000; Van Zyl et al., 2007). Berthelot & Carpentier (2003) concluded that gravel road materials with larger top sizes of 5/8 in. or 3/4 in. took longer to break down, and test sections with coarser gravel particles provided better traction and surface wearing durability than those with finer gravel under wet conditions. D. Jones et al. (2013) also suggested that gravel road materials having a maximum particle size of 1.5 in. to 1.75 in. are preferable to provide adequate all-weather durability.

Raveling and washboarding issues are usually caused by poorly graded or gap-graded materials with a lack of fines and plasticity because the particles do not bind together, ultimately resulting in significant gravel loss and recurring maintenance (D. Jones et al., 2013; Paige-Green, 1989). The properties of granular materials also depend on the characteristics of fine particles. When they are of limited proportions and combined with moisture, fines (smaller than 500 μm) provide cohesion to GMs, acting as stabilizing agents. Yoder and Witczak (1991) introduced the term “binder-soil” to describe the role of fines in granular roads. Their study, based on the California Bearing Ratio test, highlighted that fines contribute positively to stability when they fill voids between larger particles, enhancing stiffness. They found that optimal stability occurred when fines smaller than 75 μm constituted 6–8% of the solid mass in granular materials.

The inclusion of fine particles in a coarser aggregate skeleton may also occur incidentally through mechanisms like (a) interpenetration between adjacent layers with different grading distributions, (b) the pumping of water mixed with fine particles through open joints or cracks in the pavement, and (c) the diffusion of fine-water due to capillarity and water table movement (Duong et al., 2014; Giroud, 2009; Hajek et al., 1992). In all these mechanisms, a slight modification of the finest part of the grading curve occurs when fines invade the pores between larger grains. The quantity of plastic fines should be limited (lower than 10.8% within the aggregate skeleton) to achieve a high compaction level and to attain a hardening resilient behavior rather than the softening one observed in the case of granular materials containing non-plastic fines (Bassani et al., 2021).

In general, gravel obtained from quarries often falls short of specifications. Consequently, some form of stabilization is typically required to enhance the gravel's properties and align them with the desired standards. As discussed earlier, optimizing the PSD and plasticity properties will improve the load-bearing capacity performance through increased mechanical interlock, increased inter-particle friction, and increased resistance to weakening by water ingress.

Granular materials must meet grading requirements, which can be achieved by milling rocks and sediments or blending aggregates and soils of varying size distributions and origins. From previous studies, there have been varying levels of success in modifying gravels through different methods to prolong the life of these pavements and reduce the impact on the environment. This also reduces costs associated with maintaining the gravel road network and improves rideability for road users due to the slower development of defects caused by gravel loss and shape loss. Stabilization of the gravel surface roads by gradation where a tightly bound surface is formed often by the addition of a sandy clay mix with high plasticity. This technique is often proven as cost effective and practical for many rural roads (Giummarra, 2001). Stabilizations are used to enhance the performance of the granular surface aggregates while being more cost-effective than conventional construction techniques (Praticò et al., 2011).

Vaidyanathan et al., (2018) investigated 28 types of red soil to observe the effect of gradation and plasticity. Among 28 types of soil, 16 were gravel dominated and 12 were sand dominated. To gain the results, grain size distribution, Atterberg limit, modified proctor, and CBR tests were conducted in the laboratory. Finally, it has been decided that the grading, maximum dry unit weight, and optimum moisture content are highly dependent on the specific sizes of particles and the availability of single type or multiple types of particles.

Recent studies emphasized the importance of gradation characteristics (e.g., fines content, gravel-to-sand ratio, effective particle size diameters) on the performance of granular materials that are similar to the ones used in gravel roads (Haider et al., 2014; Hatipoglu et al., 2020; Rosa et al., 2017).

1.6.3 Previous studies on various stabilization techniques

Stabilization on the granular road surface is being used to increase the mechanical characteristics of the aggregates, decrease the maintenance cost & frequency, and withstand abrasion from traffic while being resistant to leaching, etc. (Barbieri et al., 2022). They investigated the effects of stabilization to find an alternative to replacing high-quality unpaved road surface aggregate materials. Eleven unconventional solutions (labeled as brine salts, clay binders, organic nonpetroleum compounds, organic petroleum products, or synthetic polymers) and two conventional solutions (cement and bitumen) were included in this study. A total of three types of laboratory experiments were conducted: the repeated load triaxial test, a modified version of the rolling bottle test, and microscope analysis. Finally, it was decided that the stabilization of unbound aggregates used in road building can give considerable benefits in terms of increased mechanical performance while also alleviating the need for high-quality aggregates.

Wu et al. (2020) compared two mechanically stabilized and two chemically stabilized granular roadway test sections with two nearby control sections. Mainly, a novel optimized gradation with clay slurry method, ground tire rubber incorporated into the surface course, Portland cement incorporated into the surface course, and a proprietary chemical stabilizer mixed with the subgrade and surface course were used as improvement techniques. Light-weight deflectometer, dynamic cone penetrometer, and nuclear density gauge tests were performed to estimate composite elastic modulus, shear strength (CBR), and dry unit weight and moisture content respectively.

Eventually, the novel optimized gradation with clay slurry and cement-stabilized sections displayed the greatest progress in performance.

1.6.4 Review on various laboratory tests on mechanical behavior

In this study, extensive laboratory tests including index properties of soils, resilient modulus (M_R) test using the repeated triaxial cycling loading test equipment, permanent deformation (PD) tests, and FT durability tests were conducted to quantify the effects of gradation and plasticity on the shear strength characteristics of granular surface materials.

The Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004) utilizes a combination of mechanistic and empirical approaches for pavement system design and analysis. It considers various input parameters that impact pavement performance, including the material properties of pavement structures (e.g., stiffness characteristics of foundation layers), as well as traffic and climate conditions. M_R is commonly used to define the stiffness of geomaterials in pavement design and analysis under cyclic traffic loading.

Numerous studies have investigated the influence of gradation on the M_R of granular materials used in road construction. An optimal fines content of 5-10% passing No. 200 sieve enhanced M_R values and reduced moisture susceptibility for base aggregates (Bennert & Maher, 2005). Tutumluer (2013) synthesized past research and concluded that fines contents in the 7-8% range were optimal for improving aggregate strength, M_R, and resistance to PD. (Xiao et al. 2012) also emphasized the significance of the gravel-to-sand ratio, highlighting its strong correlation with aggregate shear strength and demonstrating its potential to be adjusted to optimize gradations for enhanced granular materials performance.

Further research has utilized laboratory tests to evaluate the effects of varying gradations on the resilient modulus of aggregates. Aboutalebi Esfahani & Goli (2018) investigated the impact of gradation variations within the AASHTO limits on limestone and quartzite aggregates, revealing substantial differences in M_R between the upper and lower gradation limits. Ghabchi et al. (2013) observed a positive correlation between gradation density and resilient modulus, with higher densities resulting in improved M_R values and falling weight deflectometer (FWD) modulus. However, this improvement came at the expense of reduced permeability. In conclusion, the gradation of granular road materials significantly influences their M_R. A comprehensive approach to evaluating these factors will lead to more accurate predictions of the mechanical performance of granular materials in road construction.

While the design of pavement systems relies on the use of M_R to assess the elastic response of foundation layers to traffic loading, it is important to acknowledge that these systems also experience plastic deformation due to continuous traffic loads. The gradual accumulation of plastic deformation plays a significant role in the formation of wheel tracks and longitudinal surface depressions on the pavement. Moreover, it can serve as an indicator of pavement structure deterioration, which is a major concern. Although the amount of PD caused by each load application is relatively small, these displacements are non-recoverable. When these loads are repeatedly applied over a high number of cycles, the cumulative effect of the unrecoverable deformations can become significant and lead to noticeable pavement distress. As a result, it becomes necessary to characterize and understand the PDs that occur in pavement systems for thorough characterization. To this end, researchers have performed repeated load triaxial tests to evaluate the plastic deformation properties of materials.

Puppala et al. (1999) observed that the nature and magnitude of the plastic deformation of clayey soils are stress and moisture dependent. The authors also stated that sandy soils experience less plastic strain than clayey soils after performing repeated load triaxial tests.

Shakedown is defined as the point at which PD stabilizes, and the material transitions to predominantly elastic behavior, allowing it to recover its shape under repeated loading without further plastic deformation accumulation (Werkmeister et al., 2001). In their study on granular materials subjected to PD tests at varying stress levels, the researchers classified the behavior into three distinct categories: A, B, and C. Category A represents a behavior characterized by the presence of Shakedown. In other words, the PD reaches a limit where it becomes stabilized, and the material exhibits elastic behavior beyond this point. Category B displays an intermediary behavior compared to the other categories. In this category, it is not possible to clearly identify the occurrence of Shakedown or the failure mechanism. The material's response falls within a transitional range, making it challenging to determine whether the PD is stabilizing, or progressive failure is occurring. Category C pertains to the region where incremental failure takes place. In this category, the material experiences continuous and incremental deformation that eventually leads to failure. The PD does not reach a state of stabilization. These categories, as defined by Werkmeister et al. (2001) serve to classify the behavior of granular materials during PD tests under different stress levels.

In Cerni et al. (2012) the authors focused on laboratory experiments aimed at examining the PD behavior of unbound granular materials used as a subbase. In the study, the authors incorporated the concept of shakedown and demonstrated its usefulness in comprehending the material's susceptibility to rutting. Many researchers have devoted their efforts to studying the PD behavior of pavement foundation layers (Chai & Miura, 2002; Puppala et al., 1999, 2009; Saberian et al., 2018; Uzan, 2004).

M_R and PD play a crucial role as input parameters in pavement design. Consequently, researchers have extensively investigated the impact of environmental conditions, including freezing and thawing, on the stiffness characteristics and plastic deformation of pavement foundation layers. These studies have been conducted over the years by various researchers, particularly in cold regions like the majority of states in the Midwest to ensure long-term pavement performance (Domitrović et al., 2019; L. Li et al., 2011; Liu et al., 2022; Qi et al., 2008; Saberian & Li, 2021; Simonsen et al., 2002; W. Wang et al., 2018)

Literature has demonstrated that adhering to gradation specifications significantly enhances the overall quality of gravel roads. Furthermore, incorporating performance-based laboratory test results provides stronger validation of this principle. Advancing current knowledge requires leveraging laboratory data to establish correlations with index properties and optimizing material blends through multiple linear regression analyses. This approach will lead to more precise and data-driven decisions in gravel road design and maintenance, ultimately improving durability and performance.

However, most state Departments of Transportation (DOTs) specifications for gradation and plasticity of gravel road-surface materials are neither performance-based nor strictly executed. Currently, NDOT does not have a specific gradation and performance improvement guideline for gravel roads in Nebraska. Many of the practices applied to overcome gravel road-related performance and maintenance issues have not been compiled and evaluated. This research would provide opportunities to improve the design and performance of gravel roads in Nebraska and develop new gradation guideline. There is also a need to develop advanced modeling programs to estimate the type and amount of materials to be added that would improve the performance of gravel roads via use of simple index properties. Therefore, there is a need to investigate a variety of stabilization and construction techniques that may significantly reduce the deterioration of gravel roads. Hence, a preliminary survey was required to understand the present practices.

Chapter 2 Survey Results

2.1 Introduction

This chapter presents the findings of a survey conducted with county engineers in Nebraska to gain insights into gravel road management practices. Gravel roads, a crucial component of Nebraska's transportation network, face challenges like material degradation, surface distresses, and maintenance inefficiencies. To address these issues, the research team from Michigan State University (MSU) designed a comprehensive questionnaire to collect information on maintenance procedures, material sourcing, road distress types, and methods for improving performance. The survey aimed to document current practices, evaluate the effectiveness of existing methods, and identify challenges in gravel road construction, maintenance, and rehabilitation. By establishing a nationwide database of practices, the study seeks to provide profound observations for developing standardized guidelines tailored to Nebraska's needs and beyond. Additionally, the survey incorporates lessons from other DOTs to inform strategies for enhancing road performance and sustainability. The findings provide a foundation for optimizing gravel road management and addressing critical challenges in maintaining these vital infrastructure assets.

Out of all the counties, 26 responded, offering valuable information on the state-of-practice and key issues faced in maintaining gravel roads across Nebraska. Figure 2.1 presents a map of Nebraska, highlighting the counties that responded to the survey, providing a visual representation of the geographical spread of the survey data. This chapter delves into a detailed analysis of these survey responses.

To supplement the survey responses, follow-up interviews were conducted with county engineers from the responding counties in Nebraska. The goal of these interviews was to gain a deeper understanding of their practices, experiences, and challenges in managing gravel roads. These discussions provided an opportunity to clarify survey responses and explore specific methods and procedures in greater detail, offering a more comprehensive understanding of their approaches to gravel road construction, maintenance, and rehabilitation.

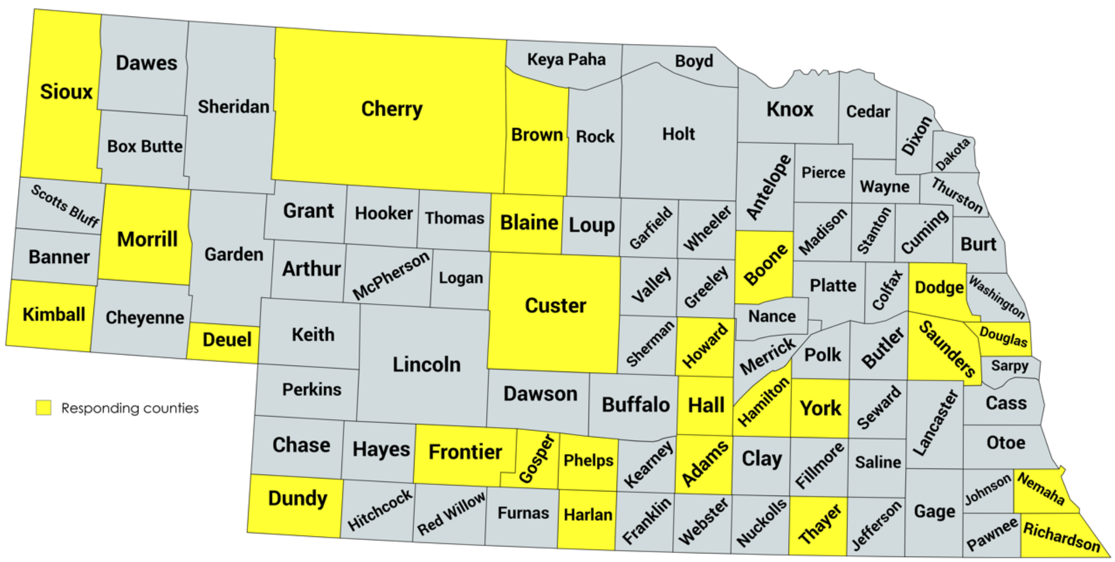


Figure 2.1 Map of participating counties in the state of Nebraska

2.2 Survey Method

The survey, conducted in 2023, targeted the geotechnical or materials divisions of all the counties in the state of Nebraska, specifically those involved in road construction. Developed and administered through Google Forms, the survey offered a user-friendly platform for data collection while ensuring uniformity in the survey format and streamlining the process for respondents.

The survey consisted of 27 structured, multiple-choice questions designed to elicit specific and comparable responses. This structured format facilitated categorization and analysis of responses. The questionnaire focused on five key categories crucial to gravel road construction and maintenance:

1. *Prevalent Distress Types and Severity*: The survey assessed common distress types and their severity, identifying the most prevalent forms of damage experienced on gravel roads and quantifying their impact on road performance.
2. *Maintenance Procedures*: The survey documented routine practices and specialized techniques used to preserve road functionality and extend service life.
3. *Aggregate Material Characteristics and Sources*: The survey gathered information on the physical properties of materials, quality assessments, and local sources for aggregates, including gravel pits and other natural resources.
4. *Stabilization Techniques and Guidelines*: The survey collected details on proprietary and non-proprietary stabilization methods used to improve road performance and the guidelines followed to ensure material and process consistency.

By focusing on these categories, the structured survey provided a comprehensive understanding of practices, challenges, and innovations in gravel road management across Nebraska counties. This robust foundation for data analysis and interpretation will inform decision-making and enhance the effectiveness of gravel road construction and maintenance practices.

2.3 Survey Results and Discussion

2.3.1 Category 1: Distress type and severity related questions

The first question in the survey asked, "*Which distress on the granular (aggregate) surface roads is the most prevalent in your county*?". This question is for understanding the various types of distresses encountered in the different parts of the state. The response options included rutting, raveling (washboarding), potholes, frost damage, dust, surface erosion, improper drainage, loss of crown, sinkholes, and farmer's wastewater. The distribution of the responses is represented using a pie chart in the Figure 2.2.

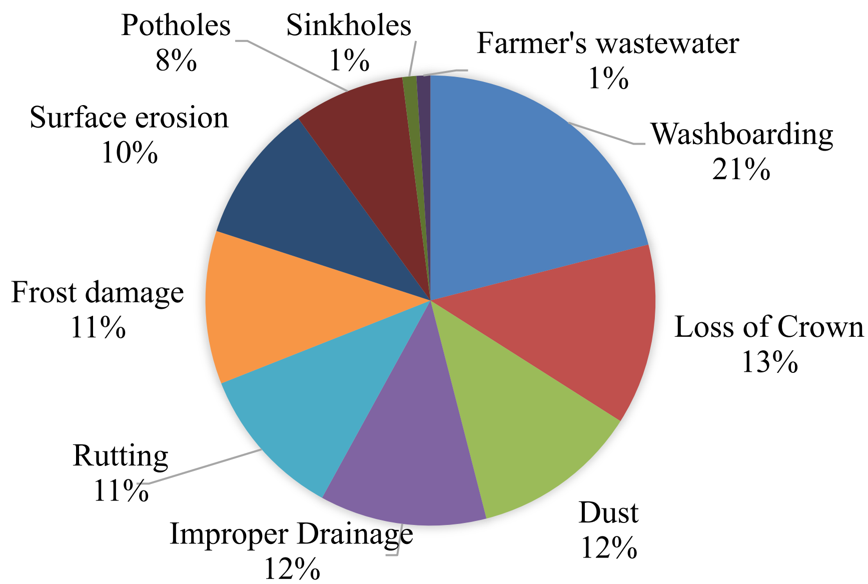


Figure 2.2 Distribution of distress types on the granular surface roads

The pie chart distribution highlights the primary gravel road distresses, with raveling being the most common issue, followed by loss of crown, improper drainage, dust, rutting, frost damage, surface erosion, and potholes. Other less common problems were also identified. These findings emphasize surface instability, drainage issues, and material degradation as key challenges in gravel road maintenance.

The second question of the survey inquired, *"What are the severity and the extent of each individual distress on the granular (aggregate) surface roads in your county?* It aimed to identify the various types of distresses, and their severity occurred in the granular surface roads. The responses categorize gravel road distresses based on severity ratings (1 to 5) reported by county engineers. Key issues include raveling, potholes, frost damage, dust, surface erosion, improper drainage, and crown loss. Sinkhole issues and farmer's wastewater were rarely mentioned. The chart highlights the diverse challenges in gravel road maintenance, particularly addressing critical distresses like raveling, potholes, and drainage-related issues for long-term performance and sustainability.

The third question in the survey was, "*Which granular (aggregate) surface road distress costs the most to maintain? (Cost includes material, labor, operator, machinery, and transportation.)".* The responses illustrate the financial implications of various gravel road distresses, as determined by the survey. Potholes, raveling, and frost damage are the costliest issues, requiring substantial investments. Other distresses, like rutting and surface erosion, also contribute, but less so. Loss of crown impacts road performance but is moderately costly. Sinkhole issues and farmer's wastewater are rare and minimally costly. This analysis emphasizes the need to allocate resources judiciously to address the costliest distresses for effective and economical road management.

2.3.2 Category 2: Maintenance procedures related questions

Two questions were asked regarding maintenance procedures employed across the state. The first question asked, *"What is your typical maintenance procedure?"*. This query aimed to understand the typical maintenance procedures employed by county engineers for gravel roads and allowed respondents to select all applicable methods. Common practices identified include reshaping existing materials using motor graders or similar equipment and adding virgin or recycled materials followed by blading. Additionally, the use of new stabilizers and maintaining proper drainage systems—through effective control of the crown, shoulder, and ditch—emerged as significant maintenance strategies. Respondents also had the option to specify other methods under the "Other" category, providing further context to unique or less common practices implemented in the field. The distribution of the responses is represented using a pie chart in the Figure 2.3.

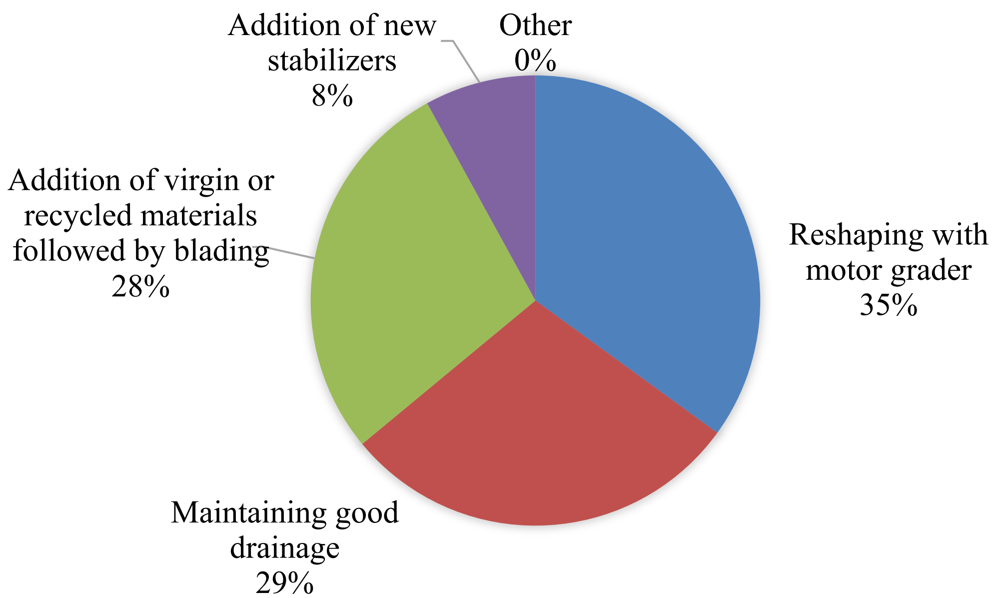


Figure 2.3 Distribution of responses of maintenance procedures

The survey shows that the most common gravel road maintenance method is reshaping materials with a motor grader (35%), followed by drainage management (29%) and adding virgin or recycled materials with blading (28%). The least used method is the addition of new stabilizers (8%). This suggests a focus on cost-effective techniques like reshaping and drainage, with less reliance on advanced stabilization methods.

The second in this category inquired*, "At what condition do you decide to start maintenance of the granular (aggregate) surface roads in your county?"*. This query provides information about the conditions that trigger maintenance for granular surface roads in a county.

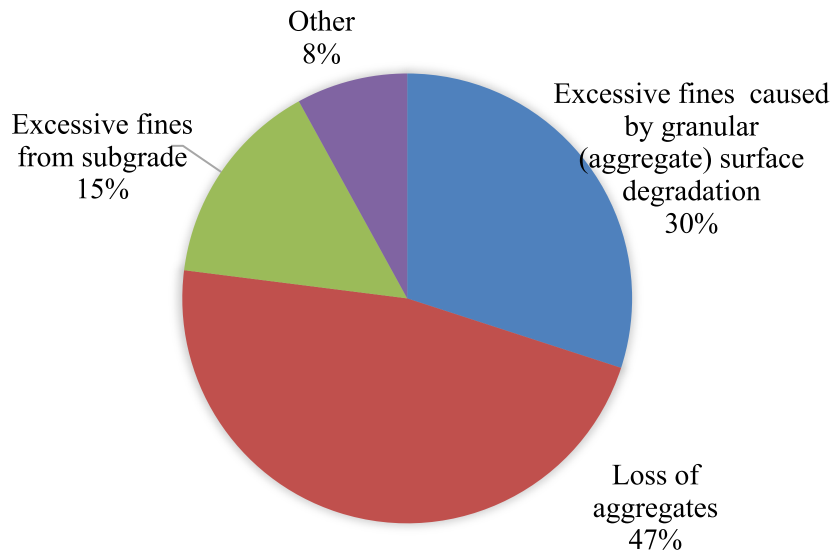


Figure 2.4 Distribution of responses of road condition to initiate the maintenance

Figure 2.4 indicates that nearly 50% of county engineers initiate maintenance procedures when aggregate loss is observed on road surfaces. Additionally, 30% of survey responses highlight excessive fines from granular surface degradation as a critical trigger for maintenance. In contrast, excessive fines from the subgrade and other factors have a minimal impact on the decision to begin maintenance.

2.3.3 Category 3: Material characteristics and sources related questions

This category of questions relates to material characteristics and local sources. The first question asks, *"What type of material is used on granular road surfaces in your county?"*. This question gathers information on aggregate materials for granular road surfaces across counties. Understanding material types helps assess performance, durability, and cost-effectiveness. It also reveals regional material availability, preferences for natural versus recycled aggregates, and sustainable practices. This information guides material selection, maintenance, and road performance improvements. The distribution of the responses was plotted in a pie chart and presented in Figure 2.5.

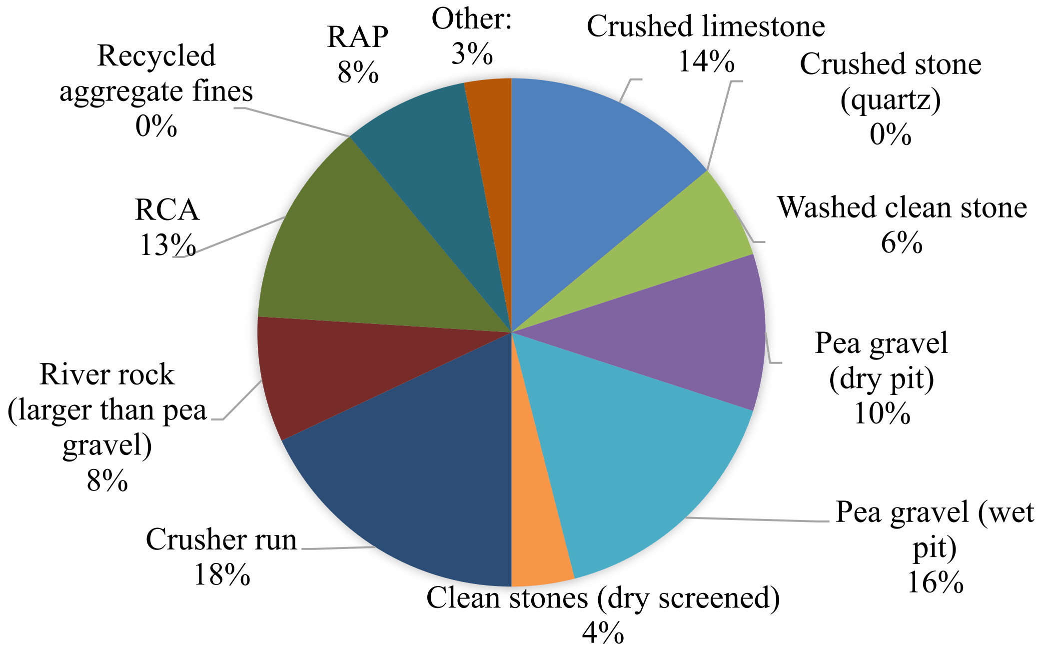


Figure 2.5 Distribution of types of various materials being used

The survey responses indicate that crusher run and pea gravel (wet pit) are the most used materials for granular road surfaces, followed by crushed stone (limestone) and recycled concrete aggregate. Materials like washed clean stone, river rock, and recycled asphalt pavement (RAP) are moderately used, while clean stones (dry screened) and other materials have limited usage. Crushed stone (quartz) and recycled aggregate fines are not used at all. This information helped to understand the material preferences and availability in county road maintenance.

The second survey question asked, *"What is the nominal size of aggregates used on the granular (aggregate) road surfaces in your county?"*. The responses indicate that less than 0.75 inches is the most used aggregate size, accounting for 35% of the answers, followed by one inch at 23%. Aggregate sizes of 0.75 inches (17%) and 1.25 inches and more (15%) are used to a lesser extent. Additionally, 10% of respondents reported that aggregate size is not defined by nominal measurements in their counties. These results reflect the variability in aggregate sizing practices, which can significantly impact road performance and maintenance.

The third question was *"What is the quality of the materials locally available for granular (aggregate) surface roads in your county?"*. The survey responses regarding the quality of materials locally available for granular surface roads indicate that many respondents (61%) consider the materials to be average in quality. A smaller proportion (25%) rated the materials as good**,** while 14% classified them as poor. These results suggest that most counties rely on materials of moderate quality, which may influence the durability and maintenance requirements of their gravel roads.

The fourth survey question relating to materials was *"How do you decide where you get surface materials for granular (aggregate) surface roads in your county?"*. This question aimed to identify the criteria counties use to select surface materials for gravel roads, focusing on practical and technical considerations. The responses reveal that locally available procedures (40%) are the most common factor, followed by compliance with Nebraska DOT or county specifications (22%) and material strength (15%). Factors like gradation (9%) and abrasion value (7%) are less frequently considered, while freeze-thaw soundness, specific gravity, and absorption are rarely prioritized. The findings suggest counties prioritize accessibility and regulatory compliance over extensive material testing when deciding on surface materials.

The fifth question asked was *"What is the typical subgrade material in your county?"*. This question aimed to identify the typical subgrade materials used across counties, providing details about the foundational materials supporting gravel roads. The responses indicate that clay

(38%) is the most common subgrade material, followed by sand (33%) and silt (22%). Fine sand (5%) and rock (2%) are used less frequently. These findings highlight the prevalence of cohesive and granular soils in subgrade construction, which significantly influence road stability, drainage, and maintenance practices.

"Do you have any clay soils that are naturally present and work into the granular (aggregate) road surfaces in your county?" was the sixth question, with only a Yes or No option. The responses indicated the presence of clay in granular road surfaces, reported by 85% of counties, may have both positive and negative effects. A map highlighting the counties which had clay availability is shown in Figure 2.6.

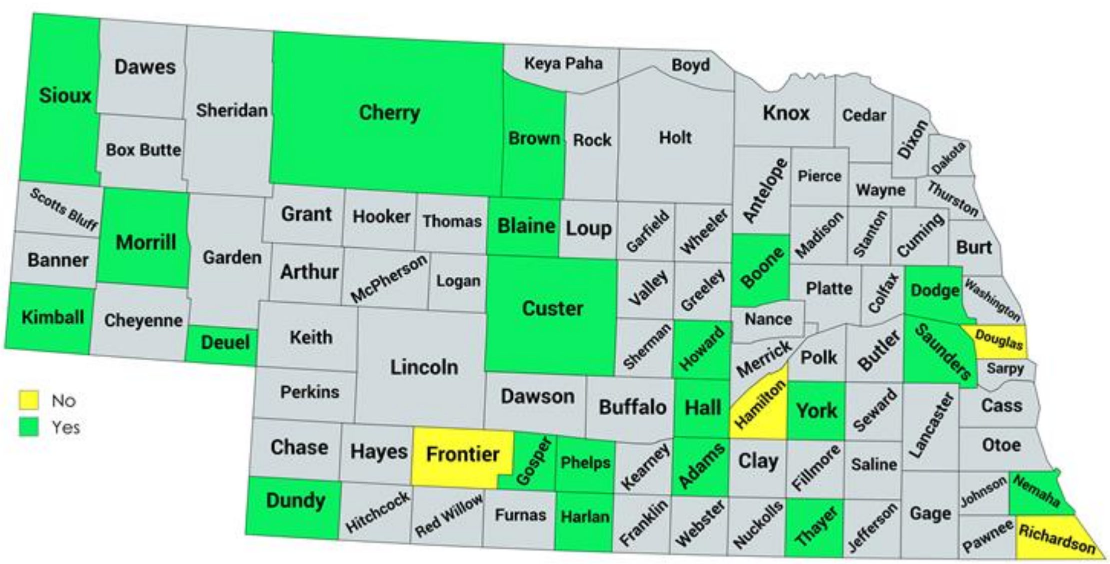


Figure 2.6 Responses of the availability of clay soils in different counties in Nebraska.

On the positive side, clay can act as a binder, helping to hold aggregate particles together and reduce surface dust. This can enhance the compaction and overall stability of the road surface under certain conditions. However, excessive clay content may lead to issues like reduced drainage, increased susceptibility to rutting, and surface instability during wet conditions. Understanding the role of clay in road performance can help counties develop tailored maintenance strategies.

The seventh question inquired was *"What is the typical percentage of fines (particles passing through #200 sieve) used in the granular (aggregate) road surfaces in your county?"*. The question was asked to understand the distribution of finer particles (passing through the #200 sieve) in the aggregate mix, which influences road stability, compaction, and drainage. The survey revealed that the most common percentage of fines used is 5% to less than 10% (30%), followed by 10% to less than 15% (26%) and 15% to less than 20% (26%). A smaller proportion reported fines of 20% or greater (11%) or less than 5% (7%). These results reflect variability in fines usage, likely driven by regional material availability and performance requirements.

The eighth question was asked to assess the accessibility of high-quality aggregate materials for road construction and maintenance: *"Are crushed gravels available in your county?"* with only Yes or No options as a response. The answers indicate that crushed gravels are available in only 35% of counties, while 65% report their unavailability. The available counties are highlighted in the map, presented in Figure 2.7. The limited availability of crushed gravel suggests that many counties rely on alternative materials, which could impact road durability and performance. This highlights the need for optimized sourcing strategies or the use of substitutes to ensure effective road maintenance.

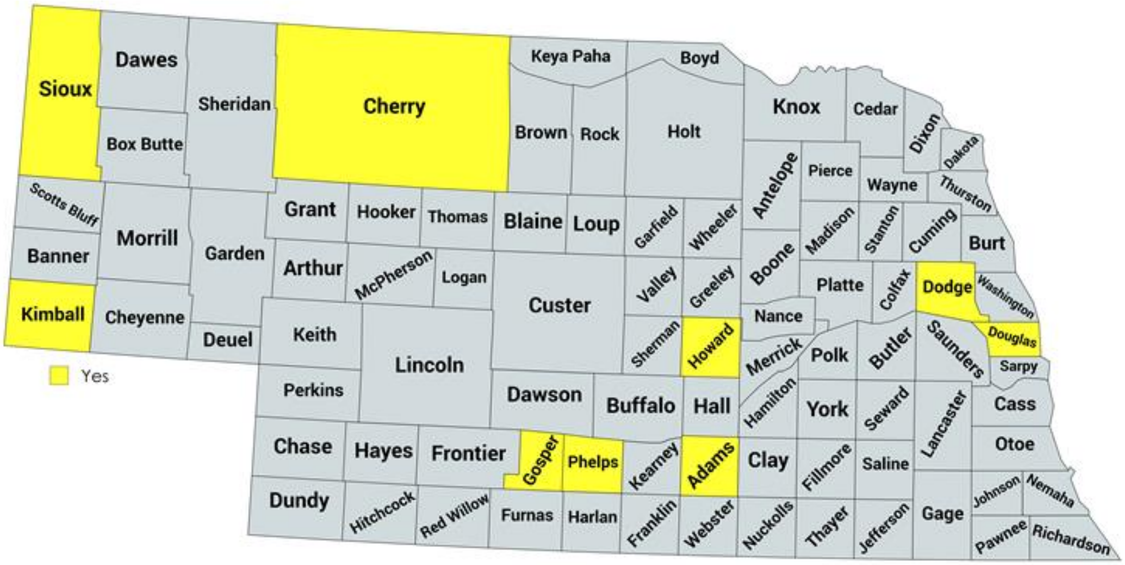


Figure 2.7 Map showing the availability of crushed gravels in state of Nebraska

The ninth question, *"What type of test do you conduct to determine the quality and performance of aggregates?"*, on aggregate testing practices highlights that 50% of respondents do not conduct any quality or performance tests, as indicated by selecting "N/A". Among those who do perform tests, 31% rely on sieve analyses, reflecting the importance of grading and particle size distribution in assessing aggregate quality. Other tests, such as the abrasion test (10%), dynamic cone penetrometer test (3%), and lightweight deflectometer test (3%), are used less frequently, indicating limited emphasis on advanced or performance-based testing. No respondents conduct Atterberg limits or California bearing ratio (CBR) tests, suggesting that these methods are either not relevant to their operations or are considered unnecessary. The 3% selecting "Other" likely represent unique or region-specific testing practices. These findings reveal a significant reliance on basic testing methods and a potential gap in comprehensive performance evaluation for aggregate materials.

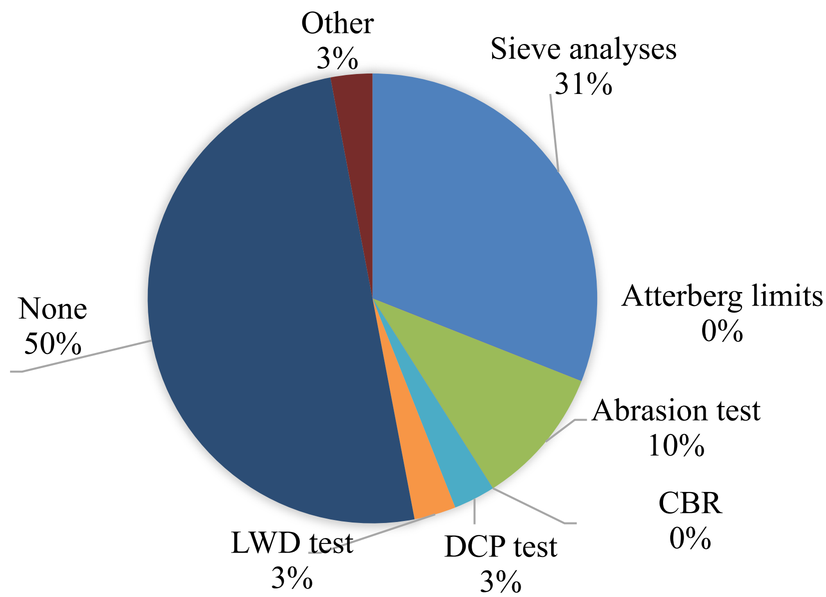


Figure 2.8 Chart showing distribution of tests conducted for determining quality of aggregates

The ninth question inquired was *"Do you have any excess quarry fines in any nearby aggregate quarries?"* This question evaluates the prevalence of excess quarry fines in nearby aggregate quarries. According to survey responses, 81% of participants reported having access to excess quarry fines, while 19% indicated their unavailability. This high availability suggests that quarry fines could play a significant role in road surface maintenance if other factors, such as quality and cost, align with project requirements.

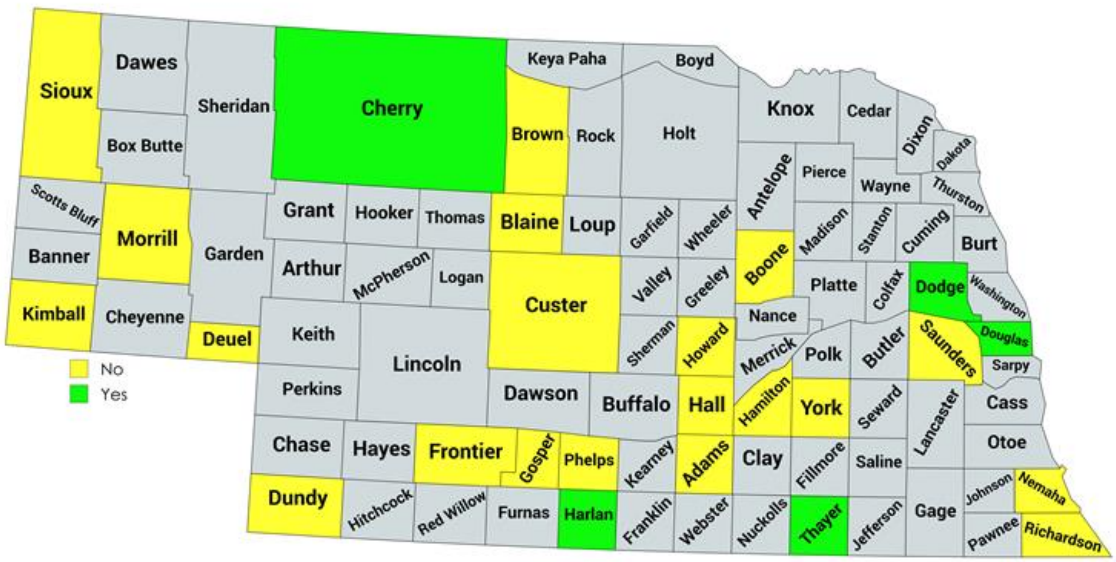


Figure 2.9 Responses of the availability of excess quarry fines in different counties in Nebraska.

Other questions in this category were dependent on whether the respondent answered Yes to question 10. If they answered No, they could respond "NA" to the following questions. The next question asked was "*If the answer to the tenth question is 'Yes', then please check the percentage of the fine particles (passing #200 sieve) of this quarry fines material. Check all that apply. (If the answer is 'No', then please check 'N/A')*". The proportion of fine particles (passing the #200 sieve) in quarry fines significantly affects their suitability for road surface applications. Excessive fines can compromise drainage and stability, while too few may reduce bonding strength. Survey responses revealed that 43% of respondents reported fine particle content of 10% to less than 30%, while 29% indicated 30% to less than 50%. A smaller percentage reported contents of 50% or greater (14%) and less than 10% (14%). These results suggest that quarry fines with moderate fine content are the most commonly available, which may be suitable for a range of road maintenance needs.

The plasticity index measures the ability of quarry fines to retain water and resist deformation, making it an important parameter for assessing their stability and drainage properties. Most respondents (43%) reported a plasticity index of 10% to less than 20%, followed by 29% reporting less than 10%. A smaller proportion indicated plasticity indexes of 20% to less than 30% (14%) and 30% or greater (14%). These findings indicate that the majority of quarry fines have moderate to low plasticity, which is generally favorable for road surface applications as it helps maintain stability and prevents excessive water retention.

The twelfth question related to the moisture content of the quarry fines and asked, "*If the answer to the tenth question is 'Yes', please check the moisture content of this quarry fines material"*. Moisture content directly affects the handling, transport, and compaction of quarry fines. High moisture levels can make materials difficult to work with and impact road performance. Survey results showed that 50% of respondents reported moisture content of 5% to less than 10%, while 33% reported 10% or greater. A smaller portion (17%) indicated moisture content of less than 5%. These findings suggest that most quarry fines have manageable moisture levels, making them easier to incorporate into road maintenance and construction processes.

The next question was an open-ended question *"If the answer to the tenth question is 'Yes', please add any other characteristics of this quarry fines material below"* and aimed to gather all the practical challenges and benefits of using quarry fines. Respondents highlighted two key aspects. First, quarry fines are generally of good quality but are costly to purchase and transport. Second, they consist primarily of fine gravel particles, which may offer specific benefits for road surfaces. This information underscored the need to address economic and logistical challenges to promote the wider adoption of quarry fines as a sustainable material for granular road surfaces.

Quarry fines are widely available and suitable for various road surface maintenance applications due to their moderate fines content, plasticity, and moisture levels. However, their high cost of acquisition and transportation hinders their widespread use. Optimized logistics and cost-sharing measures can enhance their adoption as a sustainable resource for granular road construction and maintenance.

2.3.4 Category 4: Design guidelines and stabilization related questions

The thickness of the various layers of a granular surface road is vital for assuring the roadway's durability, performance, structural integrity, and safety. Each layer of a granular surface road has a specific purpose, and the thickness of each layer must be carefully determined to satisfy the demands of the road's intended use. Some information has been asked in the survey question regarding the thickness of the different layers of high-volume (Average Annual Daily Traffic (AADT) > 500), medium-volume ($50 \leq AADT \leq 500$), and low-volume (AADT<50) granular (aggregate) surface roads in different counties of Nebraska.

For high-volume granular surface roads (AADT > 500), the survey responses indicate that most roads have a surface layer thickness of four inches or less. Similarly, the embedment layer is typically eight inches or less, though a significant number of roads lack this layer entirely. The base layer is generally between four and eight inches thick. The subbase layer is absent in most cases, but where present, it commonly falls within the four to eight inch range. Additionally, most pavements do not incorporate subgrade preparation, such as chemical or mechanical stabilization.

For medium-volume granular surface roads ($50 \leq AADT \leq 500$), the responses show a similar trend. The surface layer is predominantly four inches or less, while the embedment layer, where present, does not exceed eight inches. The base layer is typically between four and eight inches thick. The subbase layer is absent in most cases, though some pavements include a subbase within the four to eight inch range. As with high-volume roads, subgrade preparation is largely absent.

For low-volume granular surface roads (AADT < 50), the surface layer remains primarily four inches or less. The embedment layer, where present, does not exceed eight inches. The base layer is generally within the four to eight inch range, though a notable number of roads have a base layer of four inches or less. The subbase layer is largely absent, but when included, its thickness does not exceed eight inches. Similar to higher-volume roads, subgrade preparation through chemical or mechanical stabilization is rarely implemented.

The fourth question, "*Which specification/guideline do you use for the design of granular (aggregate) surface roads in your county?"*, explored regional practices and preferences *for* designing granular surface roads. Nebraska DOT (NDOT) guidelines, used by 67% of respondents, were most widely used due to their local relevance and practicality. The AASHTO Guide for Design of Pavement Structures (1993) ranked second with 15% of the responses, while the Federal Highway Administration's Gravel Road Construction and Maintenance Guide (2015) was used by 12%. Notably, SDDOT guidelines were not used, and 6% reported using other unspecified sources. These findings suggest a preference for localized guidance (NDOT) as the primary source, with national guidelines as supplementary resources.

The fifth question, *"What type of stabilizer is used on the granular (aggregate) surface roads in your county?"*, sought to identify the prevalent stabilization practices in the county. Most respondents (85%) indicated N/A, suggesting that stabilizers are rarely applied to granular road surfaces. A smaller portion (11%), particularly Douglas and Nemaha counties, reported using proprietary stabilizers, while only 4% (Saunders County) used non-proprietary stabilizers due to cost, availability, or limited necessity. This data highlights the rarity of stabilization treatments in the region, reflecting reliance on traditional maintenance methods.

"What is the drawback to meeting the gradation specifications you encountered during the design and maintenance of granular (aggregate) surface roads?" was the sixth question to determine what challenges county departments face in maintaining quality standards for granular surface roads. The most common issue is the lack of high-quality gravel road surface materials, leading to a reliance on locally available materials that may not meet ideal standards. Unavailability of specific material gradations and moisture-related issues, such as excess moisture content or lack of moisture, also contribute to these challenges. Additionally, contractors meeting gradation specifications can be a challenge, possibly due to variability in capabilities or quality control. These findings highlight the logistical, material, and environmental constraints in meeting gradation standards.

County engineers were also asked, "*If Nebraska DOT (NDOT) changes the granular (aggregate) surface course specification, can you make adjustments to the specification for your locally sourced materials?"*. The majority (69%) of respondents can adjust. This flexibility suggests that most counties have the technical expertise and resources to modify specifications to align with the properties of available materials. However, 31% of respondents reported an inability to adjust specifications, which may indicate limitations in technical capacity, material variability, or regulatory constraints in those counties. These results highlight the need for adaptable specifications to accommodate local conditions while maintaining road performance standards.

2.4 Summary

The survey findings highlight the critical need for improving surface stability, optimizing drainage, and enhancing material durability to minimize long-term maintenance costs on granular surface roads in Nebraska. Counties primarily focus on reshaping and drainage management as cost-effective maintenance strategies, with maintenance needs frequently triggered by aggregate loss and surface degradation. This underscores the necessity of material stabilization and improved surface durability to reduce frequent interventions. Material selection is largely driven by availability and regulatory compliance, often with limited emphasis on extensive material testing. Most counties rely on moderate-quality aggregates, which increases maintenance demands, and while clay content (85%) provides binding benefits, it also presents drainage challenges. Crushed gravel availability is limited (35%), which makes alternative material sourcing strategies essential. Quarry fines, though widely available (81%), face cost and logistical constraints that limit their use. Notably, half of the counties do not conduct aggregate quality testing, indicating a gap in performance-based evaluation. In terms of design practices, Nebraska DOT (NDOT) guidelines dominate, with AASHTO and FHWA guides serving as supplementary references.

Layer thickness trends across different traffic volumes remain consistent, with surface and base layers typically ranging between four to eight inches and minimal subbase or subgrade preparation. Stabilizers are rarely used due to cost considerations, and counties often struggle with challenges related to material availability, moisture content, and contractor variability in meeting gradation specifications. While a majority of counties can adjust NDOT specifications to accommodate locally sourced materials, some face regulatory or technical limitations, highlighting the need for more flexible and adaptable design standards. These insights provide valuable guidance for county engineers and policymakers in refining material selection.

Chapter 3 Material Characterization and Research Methodology

This chapter outlines the research methodology employed to evaluate the physical and mechanical properties of the various soil and aggregate materials. The study began with the selection of material types and locations, followed by a sample collection process. A series of laboratory tests were then conducted to analyze the index and mechanical properties of the materials. The comprehensive methodology provided a critical insight into the material characterization, which is fundamental for the development of new gradation guidelines and enhances the long-term performance of gravel surface roads in the state of Nebraska.

3.1 Materials collection

The research team, in coordination with several county or city engineers, superintendents, administrators, and commissioners of Nebraska, aimed to encompass a diverse range of materials commonly used in Nebraska as subgrade and granular surface materials. The primary objective is to collect and analyze these materials from various parts of the state, ensuring a comprehensive and representative evaluation of their properties and performance.

To ensure a comprehensive representation of the material variability across Nebraska, the research team selected counties from four distinct regions of the state based on critical analysis of the survey results. The survey identified the most used materials in gravel roads, guiding the selection of Douglas County (east), Harlan County (south), Cherry County (north), and Scotts Bluff County (west) for material sampling and analysis. The goal was to collect representative natural soils (e.g., subgrade clay and silts), virgin aggregates such as dense graded base materials including crusher run, gravels, clean, and sandy materials (1.5-inch, 1-inch, and ¾-inch sizes). Figure 3.1 presents a Nebraska county map highlighting the selected sampling locations, while Figure 3.2 provides images from the material collection process.

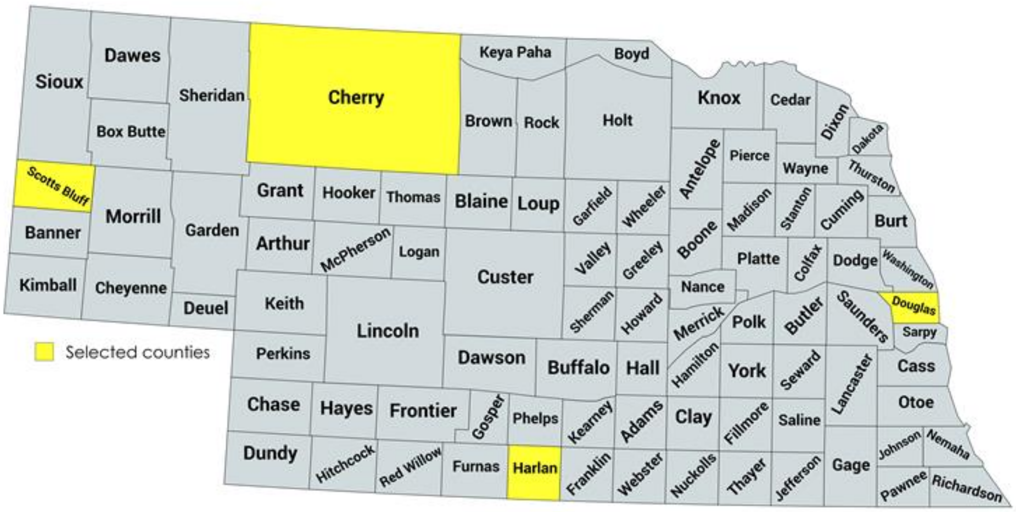


Figure 3.1 Map Indicating Material Collection Locations

Figure 3.2 Materials collections from gravel pits

The research team collected a total of 17 materials, representing a diverse range of material types. These materials were classified into three distinct categories based on their characteristics. The first category, fines with a content greater than 5%, included eight materials. The second category consisted of materials with a gravel-to-sand ratio less than one, comprising four samples. The third category included materials with a gravel-to-sand ratio greater than one, consisting of five samples. This classification ensured a comprehensive analysis of the collected materials, capturing their unique properties and variations. The list of all collected materials, presented in Table 3.1, provides an overview of materials included in the study. Also, all the collected material samples are provided in Figure 3.3, Figure 3.4, and Figure 3.5.

Table 3.1 List of collected materials

S. No	Material	Types of Materials	County	Regio n	Collection Date
1	Clean (1.5")	Surface Materials	Douglas	East	NA
2	Clean (1")		Douglas	East	NA
3	Road Gravel (3/4")		Douglas	East	4/24/23
4	Crusher run (1.5")		Douglas	East	NA
5	Crusher run (3/4")		Douglas	East	NA
6	Crusher Run (1" minus)		Harlan	South	4/24/23
7	Green Sand Surface		Harlan	South	4/24/23
8	Sand Surface		Harlan	South	4/24/23
9	Gravel		Harlan	South	4/24/23
10	Rock (3/8"-1")		Scottsbluff	West	4/28/23
11	Road Gravel (3/4" - 1")		Scottsbluff	West	4/28/23
12	Section 27		Cherry	North	4/24/23
13	Section 7		Cherry	North	4/24/23
14	Subgrade	Subgrade	Douglas	East	4/28/23
15	Subgrade (Clay)		Harlan	South	4/24/23
16	Subgrade (Silty Loam)		Harlan	South	4/24/23
17	Subgrade		Scotts Bluff	West	4/28/23

NA – Not available

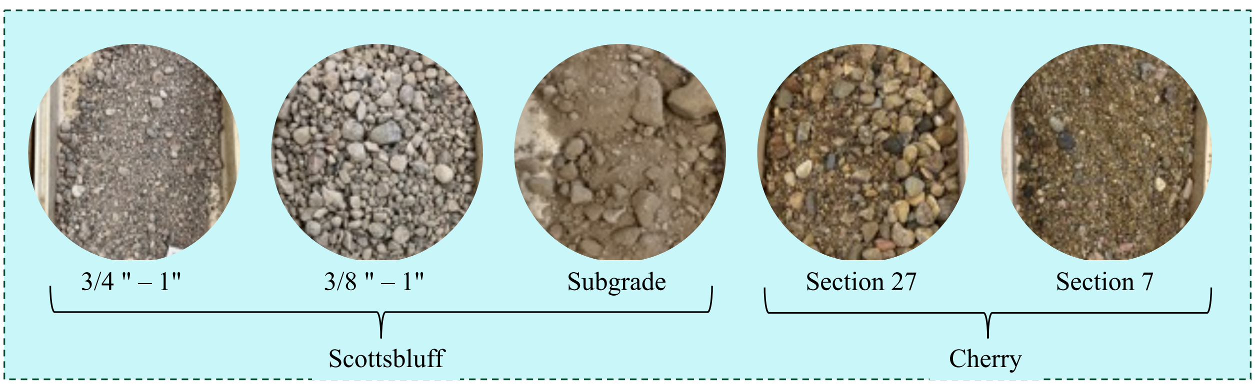


Figure 3.3 Scottsbluff and Cherry County materials used in the study

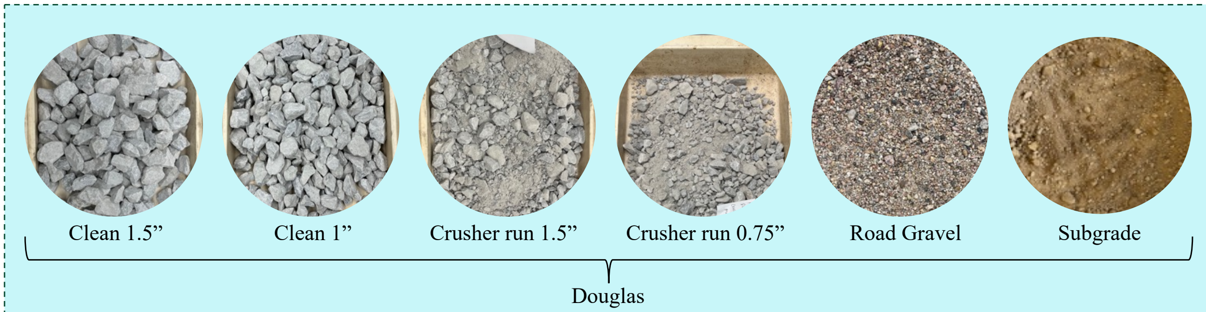


Figure 3.4 Douglas County materials used in the study

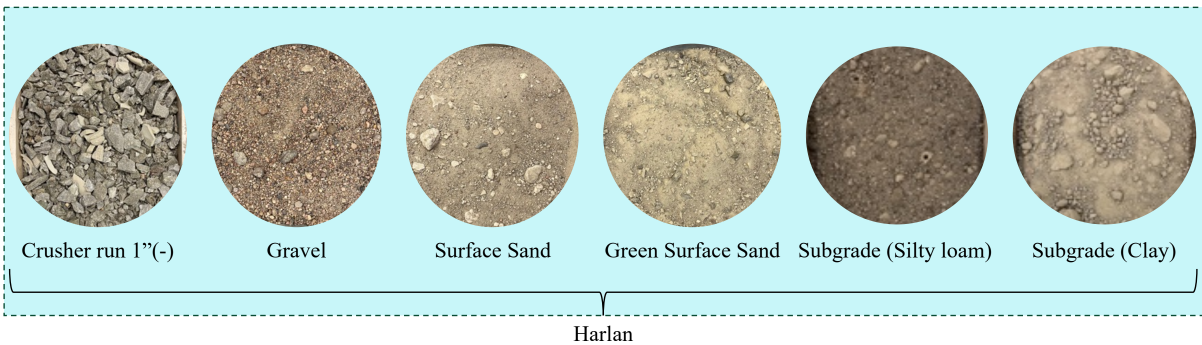


Figure 3.5 Harlan County materials used in the study

3.2 Physical characterization of virgin materials

3.2.1 Sieve/Hydrometer Analysis

Sieve analysis was performed on coarse and fine-grained aggregates, as well as natural subgrade soils, following AASHTO standards. The procedure began with *“Standard Method of Test for Materials Finer Than 75-Mm (No. 200) Sieve in Mineral Aggregates by Washing,”* (AASHTO T 11-23, 2023) to separate fine particles using wet sieving. This step was critical for accurately determining material finer than 75 µm. Subsequently, *“Standard Method of Test for Sieve Analysis of Fines and Coarse Aggregates”* (AASHTO T 27-23, 2023) was applied to evaluate the particle size distribution of the remaining material. For fine-grained soils, *“Test*

Method for Particle- size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis" (ASTM D7928, 2017) was employed to assess their gradation and particle-size distribution comprehensively.

Figure 3.6 Sieve analysis: (a), (b) wet Sieving, (c) sieve shaker (dry sieving), (d) hydrometer test

The gradation curves from the four counties highlight significant variability in particle size distributions. The soil classification from AASHTO and USCS, coefficient of uniformity, and coefficient of curvature values presented in Table 3.2. According to AASHTO classification, almost all the surface materials were classified as A-1-a, except two materials from Harlan County; Green surface sand and Surface sand were classified as A-1-b. There were seven different USCS groups for the surface materials: (1) GP, poorly graded gravel with or without sand, (2) SP, poorly graded sand with or without gravel, (3) GC, clayey gravel, (4) GM, silty gravel, (5) SM, silty sand, (6) SW-SM, well graded sand with silty sand, and (7) GW, well graded gravel. Finally, the samples of subgrade soils were classified into A-7-6 (clayey soils) and A-4 (silty soils) groups according to AASHTO, and ML (clayey silts with low plasticity) following the USCS groups. Overall, these classifications serve as a basis for assessing material properties and optimizing gradation.

Table 3.2 Virgin materials classification

Material Name	County	Gravel (%)	Sand (%)	Fines (%)	C_u	C_c	AASHTO	USCS
Clean (1.5")	Douglas	98	1	1	1.4	0.9	A-1-a	GP
Clean (1")		98	1	1	1.5	1.0	A-1-a	GP
Road gravel (0.75")		28	71	0	4.5	1.7	A-1-a	SP
Crusher run (1.5")		69	23	7	44.9	5.7	A-1-a	GC
Crusher run (3/4")		72	21	7	28.9	5.2	A-1-a	GC
Subgrade		0.0	12	88	47.6	0.4	A-7-6	ML
Crusher run (1" minus)	Harlan	76	20	3	17.7	4.1	A-1-a	GM
Green surface sand		10	74	16	173.1	22.3	A-1-b	SM
Surface Sand		10	82	9	8.3	1.9	A-1-b	SW-SM
Gravel		41	57	2	5.6	1.4	A-1-a	SP
Subgrade (Silty loam)		7	37	55	122.7	6.4	A-4	ML
Subgrade (clay)		0.0	19	81	51.4	1.0	A-4	ML
Rock (3/8 - 1")	Scottsbluff	84	13	3	7.0	2.7	A-1-a	GM
Road Gravel (3/4”-1”)		34	64	1	5.0	1.4	A-1-a	GW
Subgrade		0.0	43	57	42.0	4.4	A-4	ML
Section 27	Cherry	94	6	0	2.6	1.4	A-1-a	GP
Section7		30	69	1	2.9	1.2	A-1-a	SP

Fines = Silt and Clay, C_u is Coefficient of uniformity, C_c= Coefficient of Curvature
AASHTO = American Association of State Highway and Transportation Officials
USCS = Unified Soil Classification System

The particle size distribution (PSD) curves for all the collected materials covering both coarse and fine fractions were combined. Figure 3.7 gives the Douglas County PSD curves. Figure 3.8 shows the PSD of Harlan County and Figure 3.9 shows the virgin materials PSD from Scottsbluff County. The PSD curves of the materials collected from Cherry County are showed in Figure 3.10.

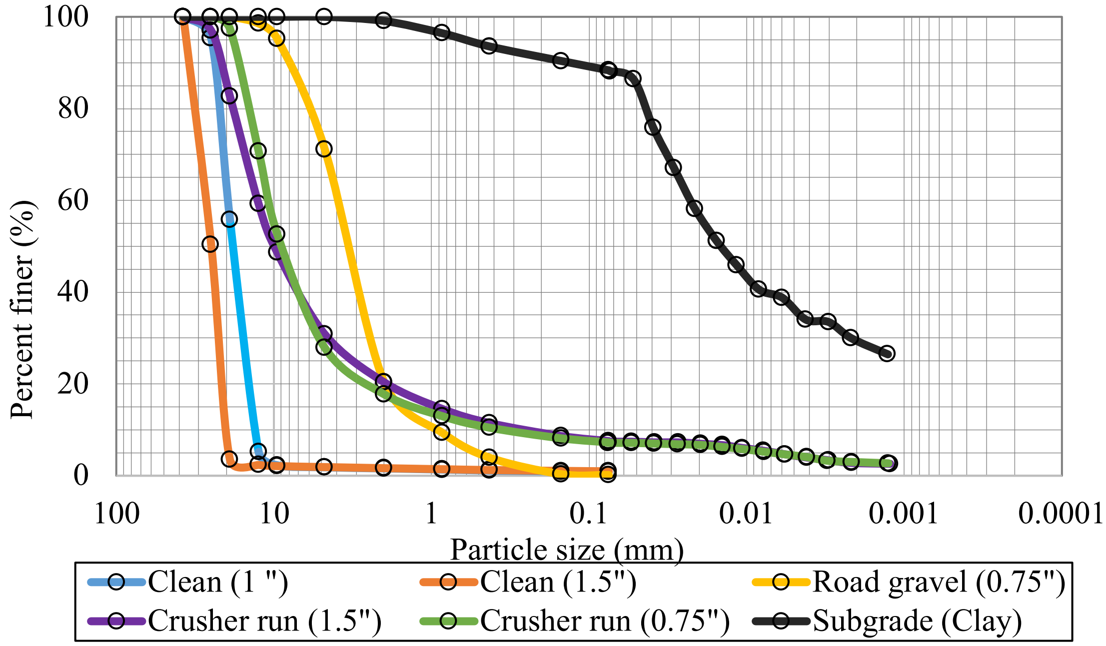


Figure 3.7 Gradation size distribution of Douglas County virgin materials

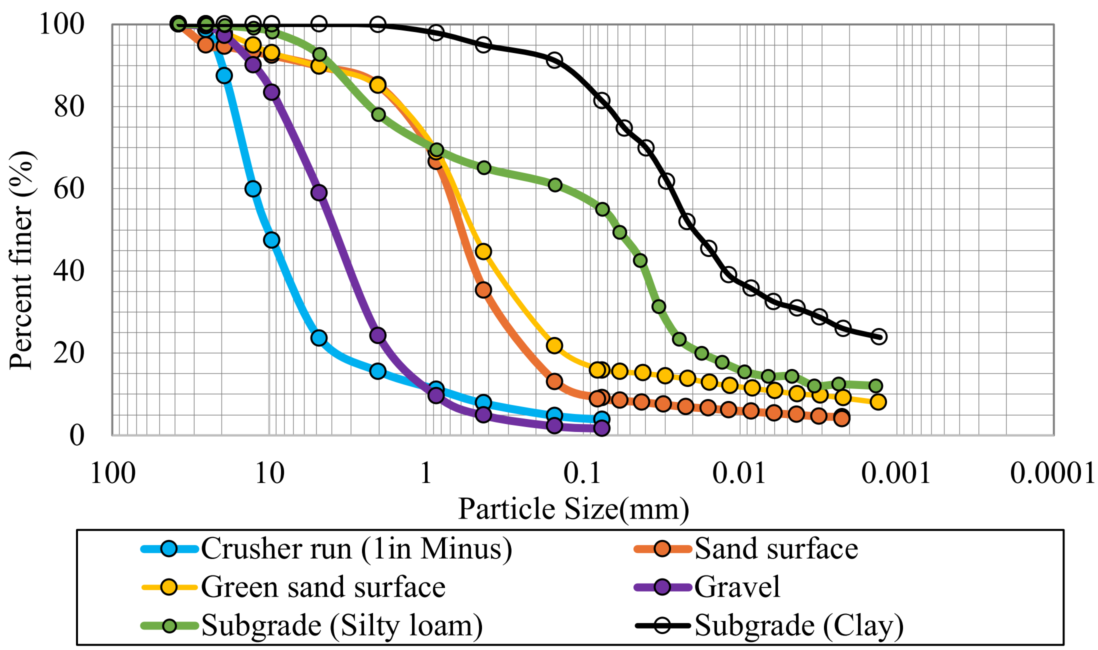


Figure 3.8 Gradation size distribution of Harlan County virgin materials

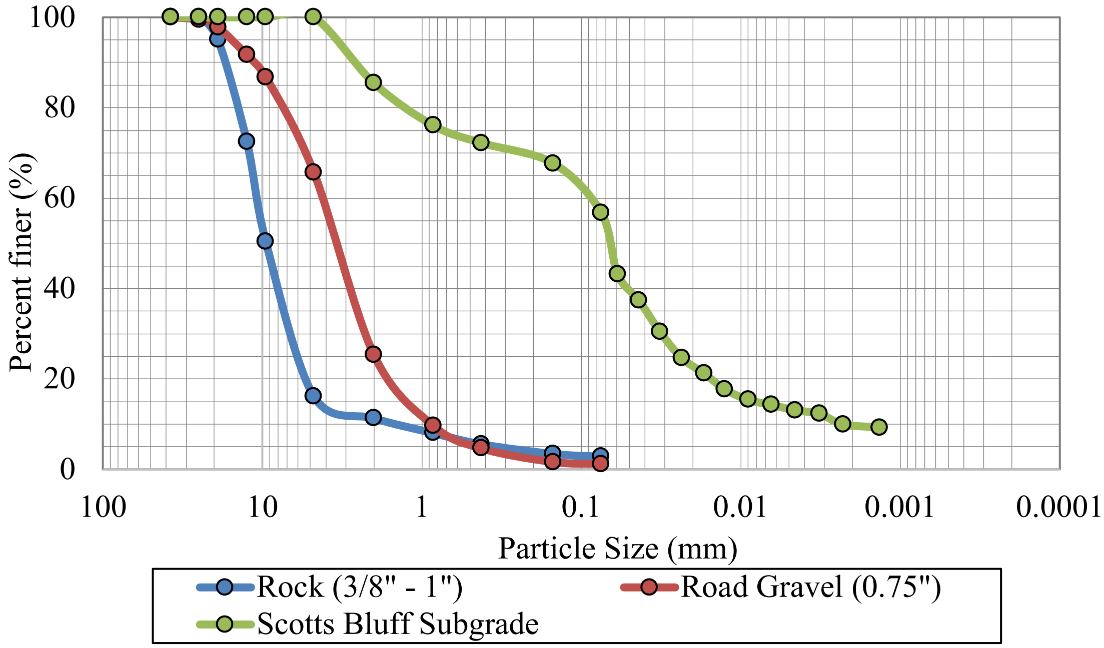


Figure 3.9 Gradation size distribution of Scottsbluff County virgin materials

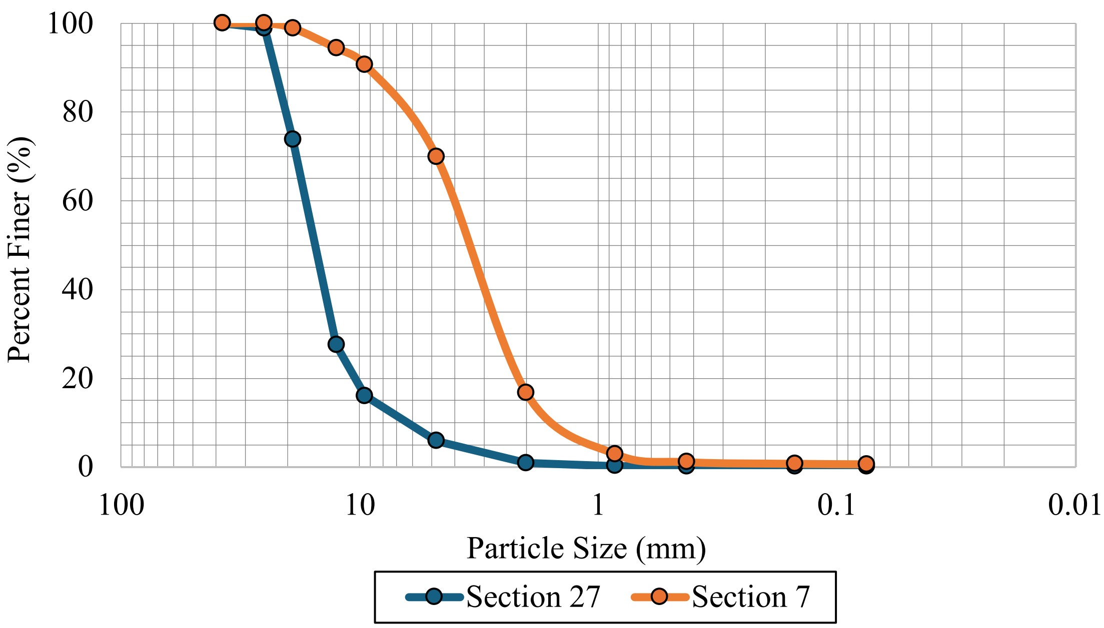


Figure 3.10 Gradation size distribution of Cherry County virgin materials

3.2.2 Specific Gravity (G_s) and Absorption

Specific gravity tests were conducted for coarse surface and subgrade soils on collected samples. These tests were conducted following *"Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate"* (AASHTO T 84-22, 2022), *"Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate"* (AASHTO T 85-22, 2022), and *"Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer"* (ASTM D854, 2010). In addition to determining specific gravity, the absorption capacity of the aggregates was measured, following the procedures outlined in AASHTO T 84-13 and AASHTO T 85-14. Each virgin material was assigned a unique code: the first letter represents the county's initials, 'V' denotes virgin material, and the number indicates its sequence within that county.

In Table 3.3 the results of the specific gravity tests, absorption tests, Atterberg limits, and moisture content tests from all four counties' materials were reported. The specific gravity values ranged from 2.30 to 2.66, indicating differences in the materials density and composition. The absorption values varied, with crusher run and subgrade materials generally exhibiting higher absorption rates, reflecting their ability to retain higher moisture.

3.2.3 Atterberg Limits

The Atterberg limit tests, conducted according to AASHTO T 89-22 (2022), and AASHTO T 90-22 (2022), were used to determine the liquid limit (LL), plastic limit (PL), and plasticity index (PI) of fine-grained soils. The test equipment representation is shown in Figure 3.11.

From the Atterberg limits test results (see Table 3.3), most virgin granular materials such as clean and road gravels were non-plastic, while the subgrade materials exhibited varying plasticity. The highest PI was observed in Douglas and Harlan clay subgrade materials, indicating potential shrink-swell behavior.

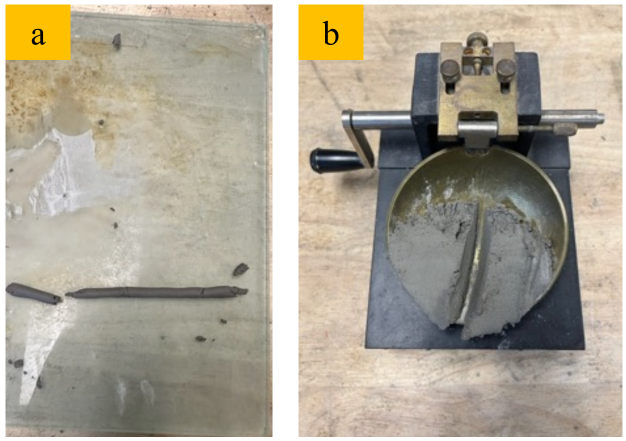


Figure 3.11 Atterberg Limit tests: (a) Plastic limit of the soil, (b) Casagrande's Liquid limit equipment

3.2.4 Moisture content

The moisture content of the soil samples was determined following *"Standard Method of Test for Total Evaporable Moisture Content of Aggregate by Drying"* (AASHTO T 255-22, 2022) and *"Standard Method of Test for Laboratory Determination of Moisture Content of Soils"* (AASHTO T 265-22, 2022). This standard test method requires measuring the water content of the samples by comparing their mass before and after oven-drying. Soil or aggregate samples collected from the field were carefully sealed and transported to the laboratory to prevent any moisture loss. Ensuring the samples remained intact and well-preserved was essential for accurate moisture determination. Table 3.3 provides the results of the specific gravity, Atterberg limits, and moisture content tests of the collected virgin materials.

In Table 3.3, moisture content results are presented. The subgrade materials retained the highest moisture levels, exceeding 11%, while sand dominant materials showed a moderate retention of moisture; however, clean aggregates have minimal moisture content, reinforcing their stability and drainage efficiency. These results highlight the diverse material properties across the sampled locations, emphasizing the importance of selecting and blending materials for optimal gravel road performance.

Table 3.3 Specific gravity (G_s), Atterberg limits and moisture content (%) of the virgin materials

Material Name	**Code**	**County**	**Specific Gravity**			**Atterberg Limits**			**Moisture content (%)**
			OD G_s	SSD G_s	Abs. (%)	LL	PL	PI	
Clean (1.5")	DV1	Douglas	2.63	2.66	1.17	NP	NP	NP	0.1
Clean (1")	DV2		2.63	2.66	1.14	NP	NP	NP	0.1
Road gravel (0.75")	DV3		2.53	2.55	0.74	NP	NP	NP	2
Crusher run (1.5")	DV4		2.61	2.65	1.36	16	13	3	0.2
Crusher run (3/4")	DV5		2.57	2.62	1.74	20	14	6	0.6
Subgrade	DV6		2.66	2.66	NA	41	26	15	12.2
Crusher run (1" minus)	HV1	Harlan	2.3	2.36	2.56	NP	NP	NP	1.8
Green surface sand	HV2		2.44	2.50	2.56	NP	NP	NP	6.6
Surface sand	HV3		2.48	2.52	1.57	NP	NP	NP	3.7
Gravel	HV4		2.53	2.55	0.78	NP	NP	NP	1.6
Subgrade (Silty loam)	HV5		2.54	2.54	NA	27	23	4	7.5
Subgrade (clay)	HV6		2.66	2.66	NA	33	24	9	11.3
Rock (3/8 - 1")	SV1	Scottsbluff	2.49	2.55	2.35	NP	NP	NP	3.9
Road Gravel (3/4"-1")	SV2		2.51	2.55	1.89	NP	NP	NP	2.2
Subgrade	SV3		2.57	2.54	NA	31	27	4	12.2
Section 27	CV1	Cherry	2.58	2.60	0.98	NP	NP	NP	0.5
Section 7	CV2		2.56	2.59	0.98	NP	NP	NP	0.6

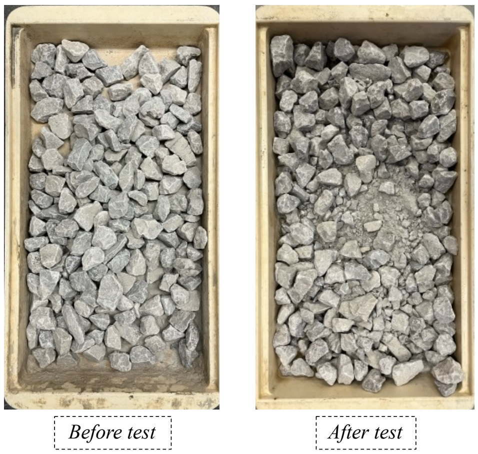
OD – Oven dry, G_s – Specific gravity, SSD – Saturated surface dry, Abs. = Absorption
NA – Not available, LL- Liquid limit, PL – Plastic limit, PI – Plasticity Index, NP – Non-plastic

3.2.5 Gyratory Compaction and Abrasion tests

The strength and stress-strain behavior of an element of soil is affected greatly by the degree to which crushing or particle breakage takes place during loading and deformation. Mechanical degradation or abrasion of granular materials used for unpaved road surface and pavement base layers can significantly influence their mechanical properties, drainage conditions, and FT durability. The gyratory abrasion test method was employed in this study to evaluate material quality.

The density-shear resistance-compaction energy relationship derived from the gyratory compaction test results facilitates the development of performance-based field specifications for granular material compaction, ensuring the final product meets the desired performance standards (C. Li et al., 2017). This test was conducted in accordance with AASHTO T 312-22 (2022). A total of 500 gyrations were applied to each specimen using a Pine 125x Gyratory Compactor. The sample before and after the test is shown in Figure 3.12

After compaction, the coarse fractions were washed, dried, and scanned to analyze changes in gradation and morphology. Volume changes in the specimens were calculated from their height measurements taken using the system's integral displacement transducer. The dry unit weight (γ_d) and void ratio (e) were determined based on the dry mass and volume of the specimen. To evaluate abrasion resistance during compaction, breakage potential (B_p), total breakage (B_t), and relative breakage (B_r) parameters were calculated. B_p is the area between the initial gradation curve and the line defining the upper limit of the silt size (0.075 mm), B_t is the area between the initial and final gradation curves, and B_r is the ratio between B_t and B_p. Figure 3.13 shows the area under curve which is used to calculate the test parameters. The condition of the sample before and after the gyratory abrasion test and change in the gradation curve are presented in Figure 3.14 . All remaining virgin material gradation curves are provided in Appendix A.



(a)

(b)

Figure 3.12 (a) Sample before and after test (b) Gyratory compactor

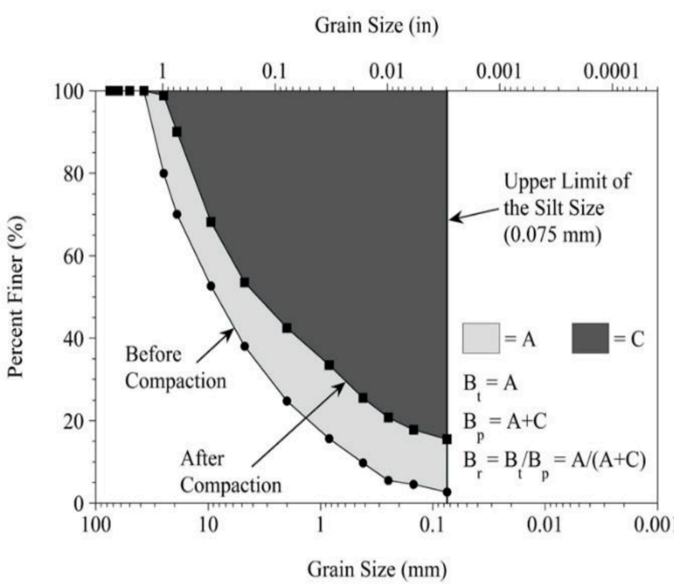


Figure 3.13 Hardin's concept to evaluate the degradation of aggregates (Hardin, 1985),

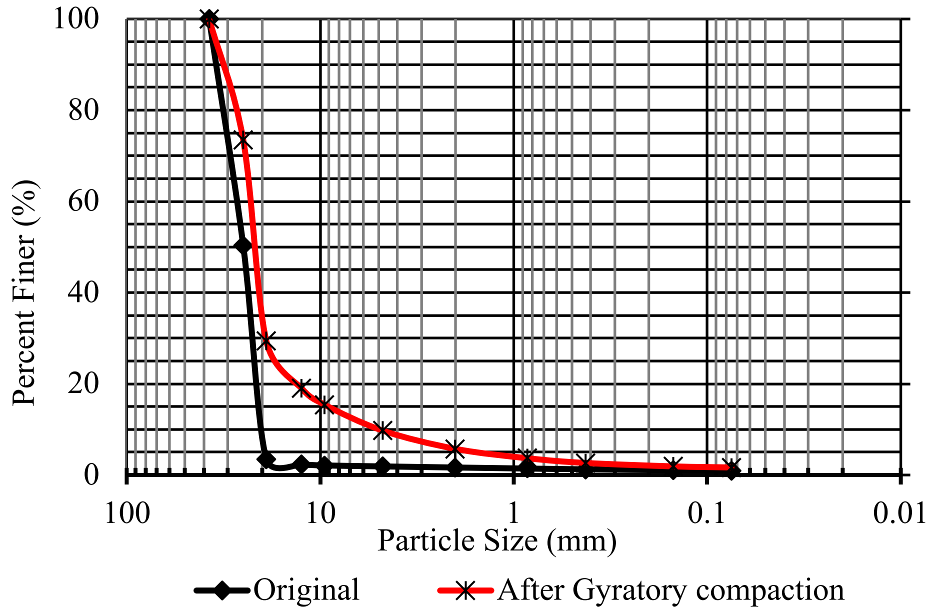


Figure 3.14 Sample gradation curves-before and after gyratory compaction test (DV1- Douglas Clean 1.5”)

Table 3.4 Gyratory Abrasion results

Material Name	Code	Total Breakage (%)	Relative breakage (%)
Clean (1.5")	DV1	17.3	6.7
Clean (1")	DV2	14.0	6.0
Road gravel (0.75")	DV3	3.9	2.3
Crusher run (1.5")	DV4	4.3	2.3
Crusher run (3/4")	DV5	6.7	3.7
Subgrade	DV6	NA	NA
Crusher run (1" minus)	HV1	11.3	5.7
Green surface sand	HV2	2.9	2.2
Surface sand	HV3	2.8	2.0
Gravel	HV4	3.0	1.7
Subgrade (Silty loam)	HV5	NA	NA
Subgrade (clay)	HV6	NA	NA
Rock (3/8"-1")	SV1	7.9	4.0
Road Gravel (3/4”-1”)	SV2	3.3	1.9
Subgrade	SV3	NA	NA
Section 27	CV1	16.3	7.2
Section 7	CV2	4.2	2.5

NA – Not applicable

The gyratory compaction tests on collected surface materials results revealed significant variation in particle breakage across different counties and material types. Among all materials, Douglas Clean (1.5") and Cherry Section 27 exhibited the highest total breakage values of 17.3% and 16.3%, respectively, indicating that these materials are more susceptible to fragmentation under stress. Additionally, Section 27 has the highest relative breakage (7.22%), suggesting that it has nearly reached its maximum breakage potential. Conversely, materials such as Road Gravel (3/4"-1") from Scottsbluff and Surface Sand from Harlan displayed lower total and relative breakage values, demonstrating greater durability and resistance to particle degradation.

In Douglas County, Clean materials showed higher breakage percentages compared to crusher-run materials, which were more durable due to their well-graded nature. In Harlan County, materials like Crusher Run (1" minus) showed moderate breakage values, while sands and gravel, such as Surface Sand and Green Surface Sand, exhibited lower breakage, highlighting their stability. Scottsbluff's Rock (3/8"-1") and Road Gravel (3/4"-1") showed moderate and low breakage values, respectively, suggesting better performance in stress-prone environments. In Cherry County, Section 27 stands out with the highest total and relative breakage, indicating a higher tendency for fragmentation, while Section 7 demonstrates better resistance.

Subgrade materials from all counties, primarily fine-grained soils like clay and silty loam, were excluded from the breakage analysis (marked as NA), likely due to their lower susceptibility to particle breakage. Overall, materials with high breakage values, such as clean materials and Section 27, may require stabilization or modification to enhance durability, while well-graded materials like Crusher Run, Surface Sand, and Road Gravel are more suitable for applications demanding higher resistance to breakage.

3.2.6 Moisture-Density test

3.2.6.1 Standard Proctor Compaction

In compliance with the NDOT requirements, the Standard Proctor compaction test, Method C of AASHTO T 99-18, was performed for surface materials and Method A was chosen for subgrade samples. These tests were used for obtaining the optimum moisture content (OMC) and maximum dry unit weight (MDU) for materials with less than 30% retained on a 19 mm (¾-inch) sieve. A mechanical soil compactor, as shown in Figure 3.15 was used to carry out the tests. This machine is equipped with a programmable digital controller, which includes pre-programmed settings for various test procedures, including AASHTO T 99-18 Method C. This feature ensures the automatic adjustment of parameters like drop height and blow count, in line with the specified standards. The test sample was prepared based on controlled gradation, as demonstrated in Figure 3.15, ensuring that the particle size distribution matched specific gradations for each material which is identified through prior particle size distribution analysis.

(a) (b)

Figure 3.15 (a) Mechanical compactor (b) Sample material prepared following gradation

3.2.6.2 Vibratory Hammer

The application of the AASHTO T 99-18 methodology to the clean materials (Clean 1.5" and Clean 1") was not possible due to the size limitations of the compaction testing standard. These materials contained more than 30% oversized particles, which exceeded the limit percentage to apply the correction factor for oversized particles. Therefore, the alternate method using a vibratory hammer, following ASTM D7382 - 20 (2020), was performed on these materials. Tests were employed in oven-dry and saturated states to determine the MDU values.

The results from both the compaction tests conducted on collected virgin materials both surface and subgrade with varying gradations indicate notable differences in MDU and OMC are presented in Table 3.5 and Table 3.6 respectively.

Table 3.5 Compaction test results for surface virgin materials

Code	**County**	**Material Name**	**MDU [pcf]**	**OMC [%]**	**C. MDU [pcf]**	**C. OMC [%]**
DV1	Douglas	Clean (1.5")	107	NA	NA	NA
DV2		Clean (1")	109	NA	NA	NA
DV3		Road gravel (0.75")	113	5.1	NA	NA
DV4		Crusher run (1.5")	134	7.3	138	6.2
DV5		Crusher run (3/4")	130	7.2	NA	NA
HV1	Harlan	Crusher run (1" minus)	108	8.4	111	7.7
HV2		Green Surface Sand	121	11.3	NA	NA
HV3		Surface sand	122	11.0	123	10.7
HV4		Gravel	119	4.4	NA	NA
SV1	Scottsbluff	Rock (3/8"-1")	120	7.3	NA	NA
SV2		Road Gravel (3/4"-1")	117	6.6	NA	NA
CV1	Cherry	Section 27	119	5.6	128	4.4
CV2		Section7	116	6.4	NA	NA

MDU – Maximum Dry Unit weight, OMC – Optimum moisture content, C.MDU – Corrected Maximum Dry Uni weight, C.OMC – Corrected Optimum moisture content, pcf – pounds per cubic feet, NA – Not available

Figure 3.16 shows the compaction curves of the surface materials, and Figure 3.17 reports the compaction curves of the subgrade materials. Among these results, the MDU values for surface materials (except clean materials) ranged from 111 pcf to 138 pcf; OMC values were between 4.4% and 11.3%. Douglas crusher run materials exhibited the highest MDU values. This is explained by their well-graded nature, as evidenced by their gradation curves. The wide range of particle sizes promotes effective packing, thereby enhancing the material's compaction characteristics. Conversely, the Harlan crusher run material had the lowest MDU value. This material also had the lowest specific gravity, which may explain its lower MDU compared to other crusher run materials.

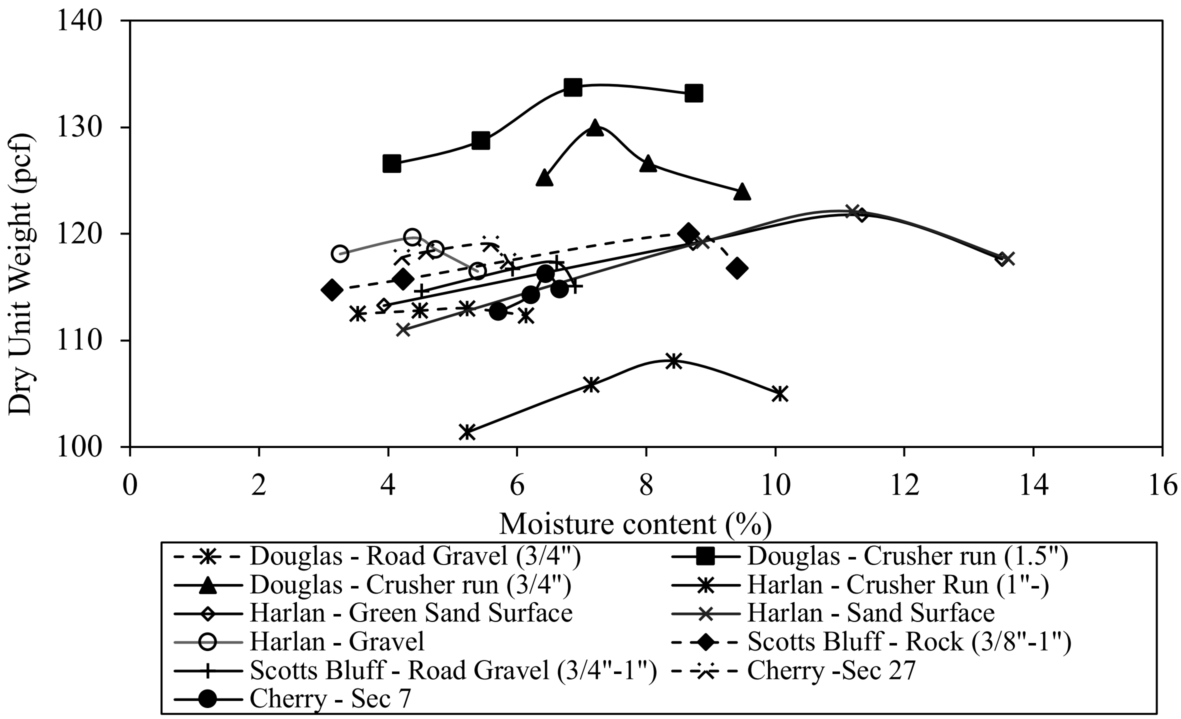


Figure 3.16 Proctor compaction curves for virgin surface materials

For clean materials, the MDU values were 107 pcf and 109 pcf; there was no OMC value since the materials were very coarse (gravel content > 90%). The average results from both the oven-dry and saturated state tests were used to determine their compaction characteristics, providing a comprehensive evaluation of their performance under varying moisture conditions.

Table 3.6 Compaction results of subgrade virgin materials

Code	County	Material Name	MDU [pcf]	OMC [%]	C. MDU [pcf]	C. OMC [%]
DV6	Douglas	Subgrade (Clay)	103	19.0	NA	NA
HV5	Harlan	Subgrade (Silty loam)	107	13.5	NA	NA
HV6	Harlan	Subgrade (Clay)	102	18.5	NA	NA
SV3	Scottsbluff	Subgrade	93	22.0	NA	NA

MDU – Maximum Dry Unit weight, OMC – Optimum moisture content, C.MDU – Corrected Maximum Dry Uni weight, C.OMC – Corrected Optimum moisture content, pcf – pounds per cubic feet, NA – Not available

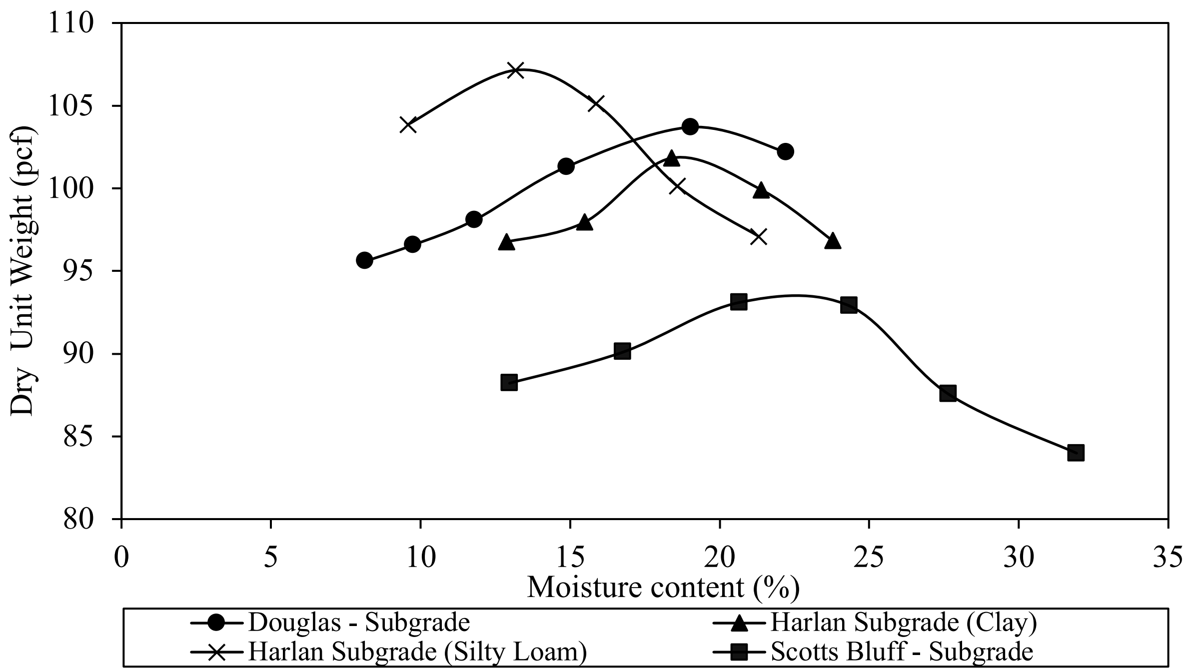


Figure 3.17 Proctor compaction curves for virgin subgrade materials

All four collected subgrade materials showed the highest OMC and lowest MDU values. The MDU values were between 93 pcf and 107 pcf and the OMC values of the same materials were between 13.5% and 22%. This is attributed to their high fines content, which possesses a greater specific surface area. These fine particles retain more water, resulting in increased OMC and reduced compaction efficiency.

3.3 Mechanical characterization of materials

Pavement systems, regardless of type, depend on all their components. Insufficient design or construction of any component reduces functionality or system deterioration, causing distress and reduced serviceability. Characterizing their mechanical properties is essential for reliable service analysis.

3.3.1 Resilient Modulus of granular materials

The resilient modulus (M_R), analogous to the elastic modulus in elastic theories, is defined as the ratio of deviatoric stress to resilient or elastic strain experienced by a material under repeated loading conditions that mimic traffic conditions. The primary reason for employing the M_R, modulus, or stiffness as a parameter for subgrades and bases is that it encapsulates a fundamental material property and can be utilized in mechanistic analyses to predict various distresses, such as rutting and roughness.

The M_R test using repeated cyclic loading triaxial test equipment is designed to simulate traffic wheel loading on in situ subsoils by applying a sequence of repeated or cyclic loads on compacted soil specimens. The stress levels for testing the specimens are based on the location of the specimen within the pavement structure. The confining pressure typically represents the overburden pressure on the soil specimen with respect to its location in the subgrade. The axial deviatoric stress is composed of two components, the cyclic stress (which is the actual applied cyclic stress) and a constant stress (which typically represents a seating load on the soil specimen).

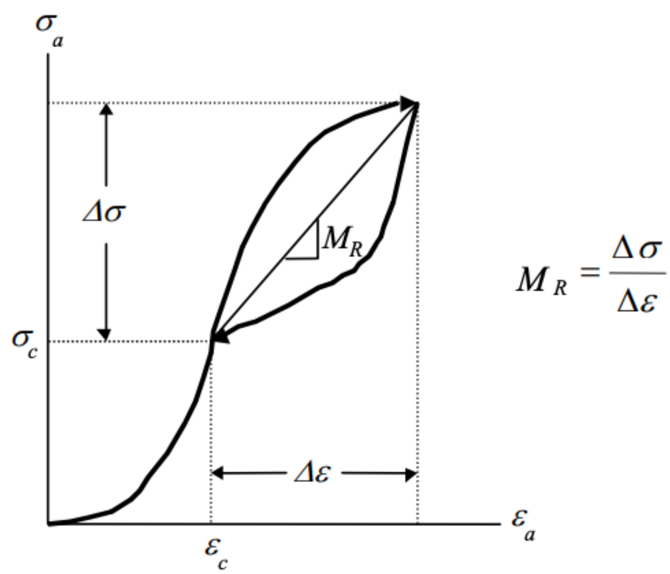


Figure 3.18 Definition of Resilient Modulus (M_R) for cyclic triaxial loading

Understanding the stress-strain characteristics of pavement foundation layers is crucial for optimizing pavement performance, as they significantly contribute to surface distresses. Previous studies have shown that the stiffness and strength of these layers play a dominant role in causing pavement distresses, especially in moisture/drainage and freeze/thaw conditions.

Conventional strength parameters like cohesion and friction angle are insufficient for assessing the structural capacity of unbound layers since pavement systems are designed to withstand traffic loads without causing foundation layer failure. In this context, the M_R as a stiffness property, holds immense value for pavement analysis. M_R values can be utilized in structural response analysis models to calculate the pavement's structural response to wheel loads and in pavement design procedures to design pavement structures. It is the most crucial unbound material property input in most current pavement design procedures.

Laboratory characterization of M_R is one of the most widely used methods to determine the stiffness characteristics of the unbound layers. M_R is a parameter that accurately represents the ratio between the cyclic stress ($\Delta\sigma$) applied to a material and the corresponding recoverable (elastic) strain after undergoing multiple cycles of repeated loading, which closely mirrors the traffic loading. As such, M_R serves as a direct measure of unbound material stiffness within pavement systems.

3.3.1.1 Test method/Procedure

The materials were prepared meticulously in accordance with AASHTO T 307-99 (1999) standards for preparing specimen for the M_R test. Since variations in gradation can lead to distinct behaviors, it was paramount to manufacture and control the particle size distribution of the mixtures employed in this research. Thus, materials were washed over a sieve (75 μm) and the washed portion was dried, sieved, and stored. The washing water that passed through the 75 μm sieve was collected and left for at least 24 to 48 hours for settlement. The resulting fines (settled fines) material then oven dried at 105 ºC for at least 24 hours.

The quantity of the material was calculated to achieve 95% compaction density of the cylindrical test specimen (6 in. by 12 in.). This procedure consists of preparing six separate oven-dry batches with the original gradation of the corresponding material.

Figure 3.19 Six batches of materials prepared following the gradation

Figure 3.19 shows the six batches prepared for Harlan Crusher run (1”) material as an example. The gradation of each lift was kept identical to reduce inhomogeneity occurring during specimen preparation.

Initially, a latex membrane was folded and inserted into the mold. To ensure a tight fit against the mold, a vacuum was applied, as depicted in Figure 3.20a. A filter paper was positioned on the bottom platen to prevent the clogging of porous stones by fine particles.

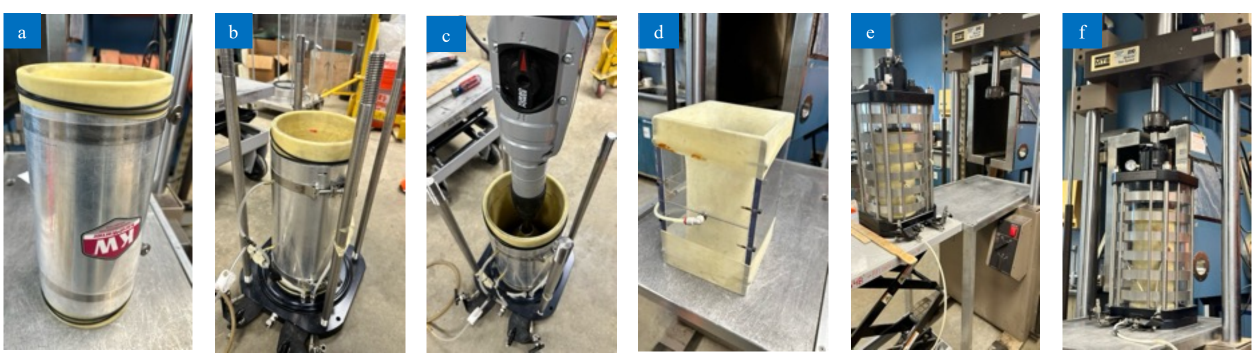


Figure 3.20 Specimen preparation steps: (a) split mold and membrane, (b) mold is placed on the platen and vacuum is supplied (c) first compaction layer (d) second membrane with membrane extender (e) setup placing under the loading actuator (f) specimen ready for testing

The six batches of the sample material were mixed to corresponding OMC. The first layer of the specimen was introduced into the mold and platen assembly, where it was compacted using vibratory compaction until reaching the desired thickness of approximately two inches per layer. After compacting the first layer, the surface of the specimen was trimmed to ensure its integrity, and the subsequent layer was then placed and compacted accordingly. This process was repeated for each layer until a total height of 12 inches was attained. The compacted specimen was then transferred to the chamber for the subsequent assembly step. One significant concern during compaction, particularly for coarse-grained materials, was the risk of the latex membrane being punctured by sharp edges. To address this issue and mitigate its potential consequences, a second membrane (placed on the second membrane mold as shown in Figure 3.20d) was utilized to create a confining space for the specimen within the chamber. This second membrane was introduced immediately after detaching the compaction mold. Before removing the mold, the vacuum port was redirected to the bottom platen to apply vacuum pressure during the placement of the second membrane. The folded membrane was then affixed to the bottom platen and sealed with O-rings. Subsequently, filter paper and the top platen were positioned and sealed with O-rings again to minimize the leakages (Figure 3.20f). An acrylic chamber was placed which allowed application of confining pressure. Throughout the assembly process, the specimen remained under vacuum via a vacuum line until the desired confining pressure was applied. The constant stress applied was typically equivalent to about 10% of the total axial deviatoric stress.

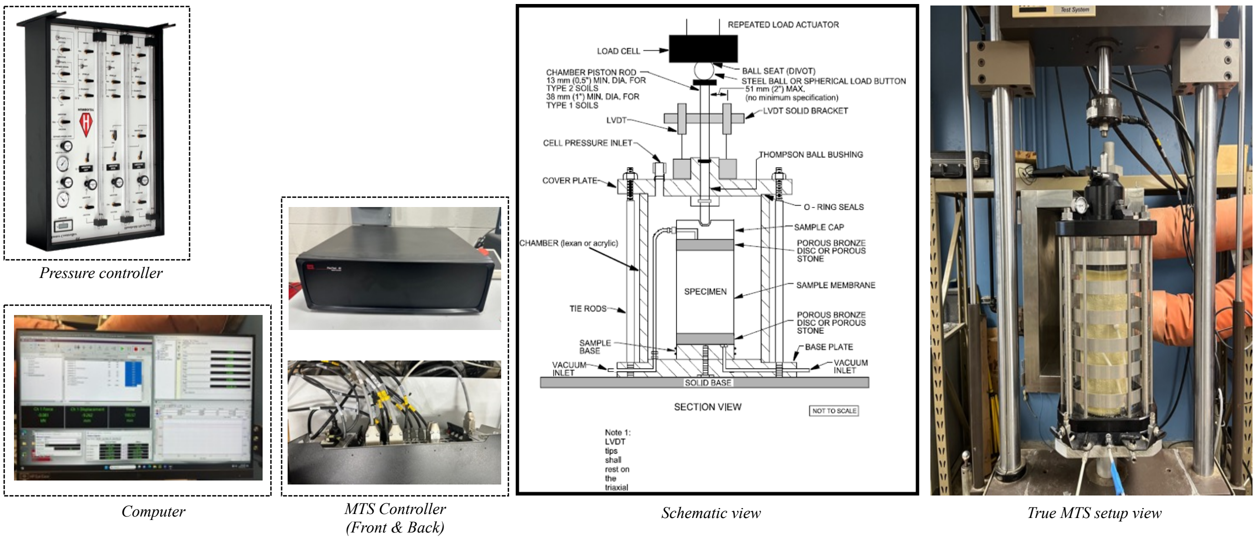


Figure 3.21 MTS 810 resilient modulus testing equipment setup

The MTS 810 testing equipment showed in Figure 3.21 was employed to perform the M_R tests. This specialized equipment can simultaneously apply both deviatoric stress and confining pressure through a series of cyclic loadings, effectively simulating the traffic loading conditions on pavement layers. The axial strains were measured using Linear Variable Differential Transformers (LVDTs). The test data can be obtained simultaneously using a data collection system.

As shown in Figure 3.22 a haversine-shaped wave load pulse with a loading period of 0.1 s and a relaxation period of 0.9 s is used in the testing.

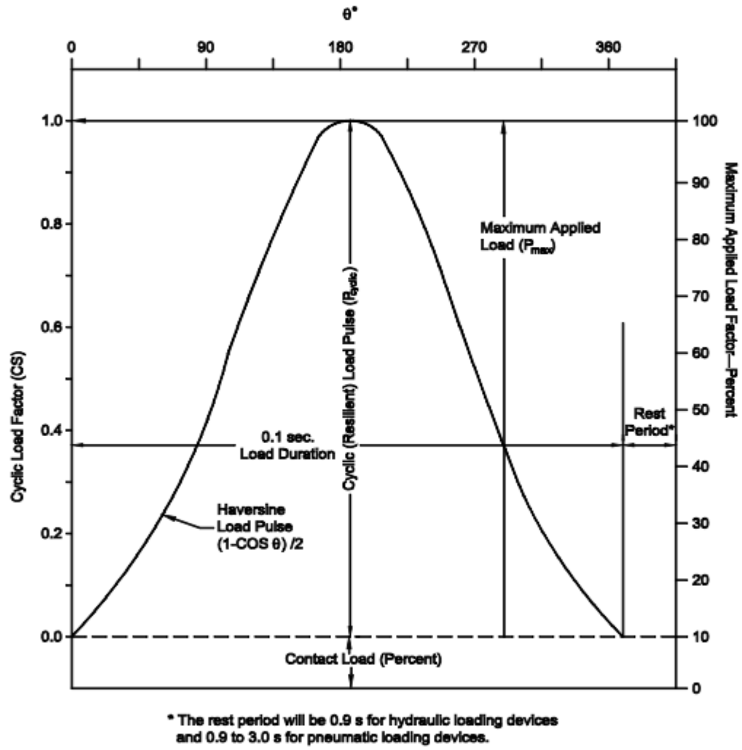


Figure 3.22 Cyclic axial load for Resilient modulus

After completing the assembling process, the specimen was adjusted to the center of the loading actuator. A computer application, "MTS test suite", was utilized to generate the required M_R testing sequences. There were two separate loading sequences for both base/subbase materials and subgrade materials. For the surface materials, the loading sequences for base/subbase was adapted. The testing program for the surface materials is presented in Table 3.7 and the subgrade loading sequences are provided in Appendix A. These loading sequences were created as outlined in AASHTO T 307-99 (1999).

Table 3.7 Testing sequences for base/subbase materials provided in AASHTO T 307-99

Seq. No.	Confining Pressure (kPa)	Cyclic Stress (kPa)	No of Load Applications
0	103.4	93.1	500–1000
1	20.7	18.6	100
2	20.7	37.3	100
3	20.7	55.9	100
4	34.5	31.0	100
5	34.5	62.0	100
6	34.5	93.1	100
7	68.9	62.0	100
8	68.9	124.1	100
9	68.9	186.1	100
10	103.4	62.0	100
11	103.4	93.1	100
12	103.4	186.1	100
13	137.9	93.1	100
14	137.9	124.1	100
15	137.9	248.2	100

During the M_R tests for the subbase materials, a total of 2,000 load cycles were applied, including 500 conditioning cycles and 100 cycles per sequence (as detailed in Table 3.7), under five different confining pressures. These tests adhered to the specifications outlined in the AASHTO T 307-99 standard, ensuring the accurate simulation of field conditions in the laboratory environment.

3.3.2 Permanent Deformation Test

While the resilient behavior test is employed to determine the necessary strength parameter for designing the appropriate height of granular layers in a pavement structure, the permanent PD study aids in predicting the anticipated accumulation of non-recoverable deformations in granular layers over a specified number of load repetitions. These accumulated permanent deformations subsequently contribute to the development of rutting within the pavement structure.

The permanent deformation (PD) test provides a fundamental relationship between stress and PD of pavement materials for structural analysis and performance prediction of layered pavement systems. This test offers a means of characterizing pavement construction materials, including unbound granular materials, under various conditions (e.g., moisture, density, etc.) and stress states that simulate the conditions in a pavement subjected to moving wheel loads. The studies conducted in this field reveal that the accumulation of plastic deformation in granular layers is influenced by various factors, including stress level, the number of load applications, moisture content, density, aggregate type and gradation, fines content, and stress history.

3.3.2.1 Test method/Procedure

A repeated cyclic loading triaxial test was performed to calculate PD for all subbase virgin materials. Since unpaved road materials are surface materials subjected to relatively low confining pressures, Stress Level 1 is more appropriate for testing these materials to simulate the field conditions that unpaved road surfaces experience, where significant overlying layers do not confine them and are thus more vulnerable to environmental and traffic-induced stresses. This standardized testing protocol ensures the reliability of results for assessing the PD characteristics of granular materials.

Table 3.8 Permanent deformation testing protocol for geosynthetic-reinforced and unreinforced granular material (National Academies of Sciences, 2017)

Stress Level	**Confining Pressure**		**Max. Axial Stress**		**Cyclic Stress**		**Contact Stress**		**No. of Load Applications**
	(kPa)	**(psi)**	**(kPa)**	**(psi)**	**(kPa)**	**(psi)**	**(kPa)**	**(psi)**	**(kPa)**
0*	103.4	15	103.4	15	93	13.5	10.3	1.5	500
1	27.6	4.0	192.9	28.0	173.6	25.2	19.3	2.8	10,000
2	48.2	7.0	130.9	19.0	117.8	17.1	13.1	1.9	10,000
3	68.9	10.0	68.9	10.0	62.0	9.0	6.9	1.0	10,000
4	48.2	7.0	192.9	28.0	173.6	25.2	19.3	2.8	10,000
5	68.9	10.0	192.9	28.0	173.6	25.2	19.3	2.8	10,000
6	89.6	13.0	192.9	28.0	173.6	25.2	19.3	2.8	10,000
* Note: Stress Level 0 is for preconditioning of granular material.									

The PD test procedure was conducted after completing the M_R test. Initiated with a preconditioning phase, 500 load cycles were applied to the specimen at a maximum axial stress of 15 psi (103.4 kPa) and confining pressure of 15 psi (103.4 kPa; Stress Level 0; see Table 3.8). This preconditioning step aimed to mitigate the effects of the time interval between compaction and loading and to distinguish between initial loading and subsequent reloading behaviors. Following preconditioning, the test began by applying a confining pressure of 4 psi (41.4 kPa) while maintaining contact stress of 10% ± 0.1 psi of the maximum applied axial stress during each stress level. A specified axial load was then applied to the specimen for 10,000 load cycles according to the stress levels outlined in Table 4.

3.3.3 Freeze-thaw performance of granular materials

This testing method is particularly relevant for understanding the behavior of unpaved roadway materials. It simulates the environmental conditions these materials encounter, especially in regions with frequent FT cycles and high moisture levels. The freezing and thawing was employed in accordance with ASTM D560M - 16 (2016), which outlines the procedures for evaluating material stiffness changes under cyclic freezing and thawing. The open system approach employed in this study replicates field conditions of unpaved roadways, where materials are exposed to fluctuations in moisture and temperature. This exposure is crucial because, during freezing, water in the soil expands, leading to increased internal pressures, while thawing causes contraction, potentially weakening structures and causing damage.

This open system FT method is valuable in demonstrating how materials in unpaved roadways respond to the expansion and contraction forces induced by FT cycles. By replicating real-world environmental stresses, this method evaluates material degradation and stiffness loss over time, aiding in the design of more durable unpaved roadway materials.

The specimens were compacted using the same procedures as the M_R tests. Each specimen had a diameter of 6 in. (15.24 cm) and a height of 12 in. (30.48 cm). Each specimen was compacted in six layers, with each layer consisting of the same material at the MDU and OMC and then exposed to five FT cycles under the open system method. After compaction, the specimens were placed on a saturated felt pad with a filter paper base, which allowed water to rise through the sample, mimicking the natural capillary action observed in field conditions where soils remain saturated in cold climates.

Before commencing the FT cycles, the membranes covering the specimens were removed to ensure the total exposure of the material to freezing and thawing conditions, accurately simulating field environmental effects. The specimens were then subjected to freezing (Figure 3.23a) in a controlled environment set at -23 ± 2°C for 24 hours per cycle, followed by a thawing period of 24 hours in a humidity-controlled box (Figure 3.23b), making sure that the felt pad was always saturated; otherwise water was supplied. This setup replicated conditions commonly found in the field where moisture is present during thawing, enhancing the accuracy of the simulation in assessing the material's behavior. After this process of freezing and thawing, the specimen is ready for M_R and PD testing (Figure 3.23c).

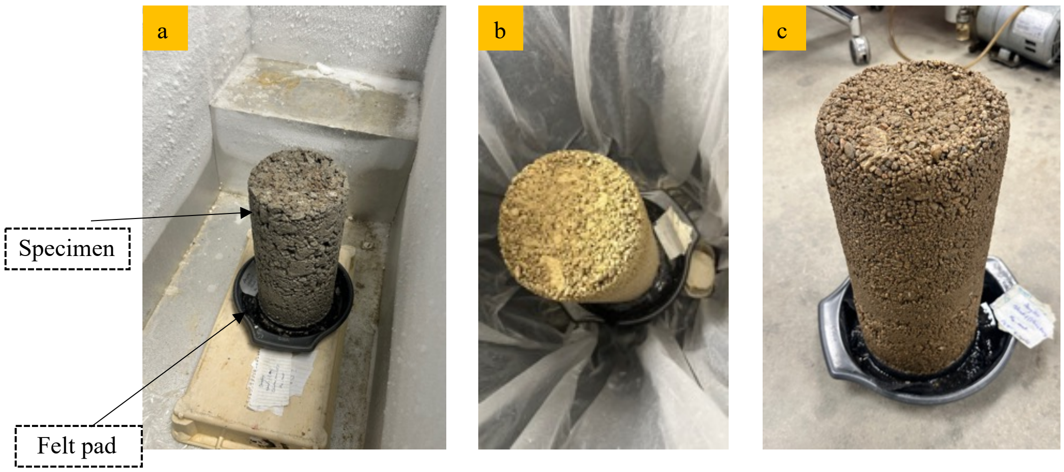


Figure 3.23 Open-system FT cycle process: (a) freezing the specimen in a freezer at -23ºC, (b) thawing specimen in a humidity-controlled chamber and (c) specimen ready for testing

This testing process offers critical data for understanding long-term material performance in challenging climates, contributing to the development of improved construction and maintenance strategies for unpaved roads.

3.4 Gradation Optimization

Particle-size distribution plays a pivotal role in determining the strength of gravel, primarily through the interlocking of particles and the principle of maximum density. Furthermore, particle size distribution influences the material's permeability, particularly the percentage of particles smaller than 0.5 mm. Plasticity also contributes to the aggregate's strength. When wet, the fine material reduces interlocking, while in dry conditions, it provides cohesive strength that holds the aggregate in place. Furthermore, hardness is a crucial characteristic, as aggregates are anticipated to possess sufficient hardness to withstand substantial deformation under compaction and traffic, and durability to endure without breaking down due to exposure.

Granular stabilization is the process of adding other granular material to correct a deficiency in the materials properties of the virgin materials. These stabilizing materials include natural gravel, crusher run, and fine-grained soils.

Materials collected from the gravel pits are open graded and poorly graded, and stabilization is usually required to improve the gravel properties and behavior under traffic loads. Numerous stabilization techniques, including the incorporation of lime, cement, and chemicals, have demonstrated efficacy. However, the additional cost associated with purchasing and incorporating these additives may render them economically unfeasible. Consequently, an alternative technique known as granular stabilization presents a potential solution. Granular stabilized materials are those to which another granular material is added to rectify a deficiency in the inherent material properties of the parent material (Jameson, 2019).

Moreover, materials from the same county were blended to reduce hauling and transportation costs associated with road construction. The gradation of these blends was assessed by following a recommended range, as shown in Figure 3.24 which defines the upper and lower limits of the ideal particle size distributions for surface and base course applications. This process was carried out to develop a database for performance-based material characterization.

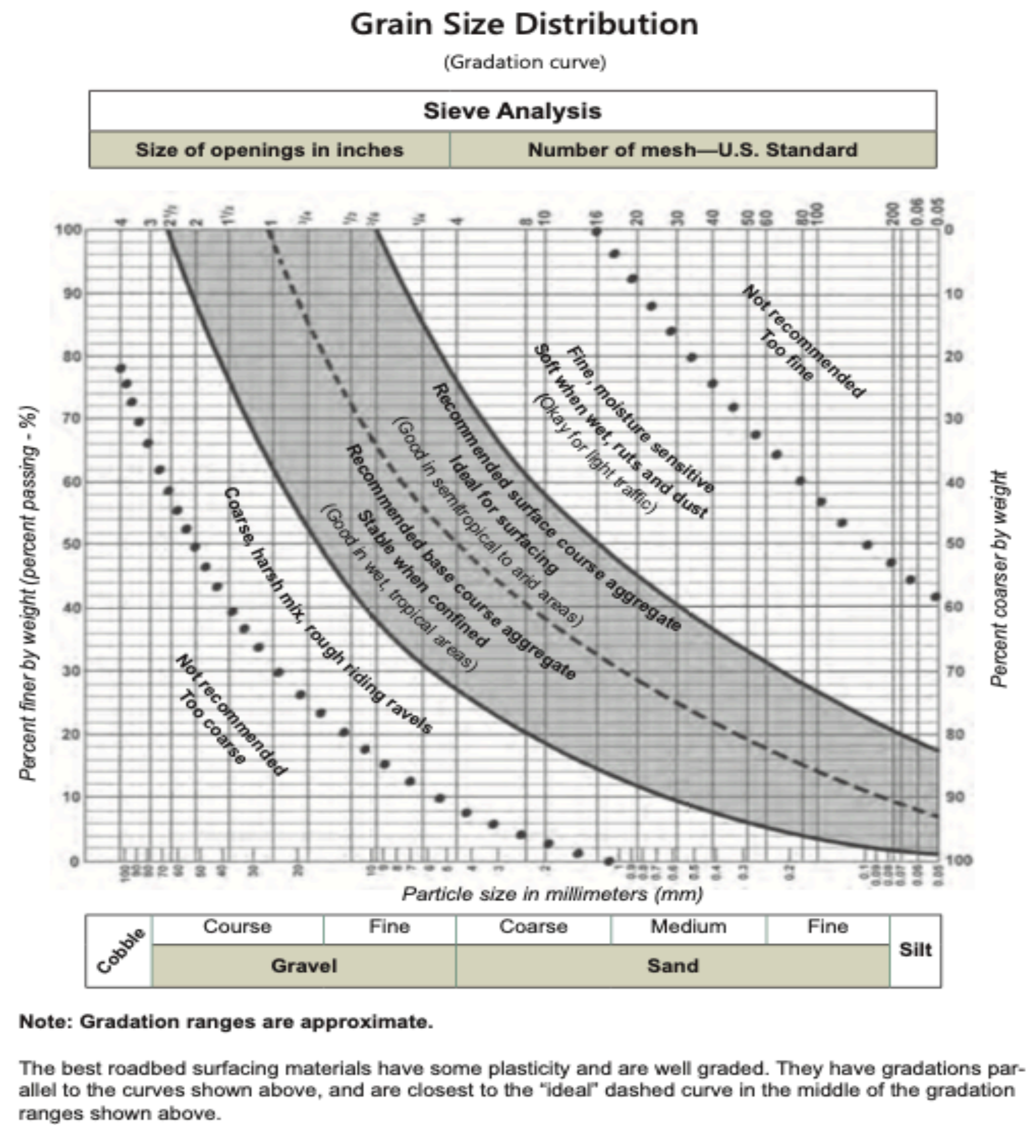


Figure 3.24 Ideal grain size distributions for surface-course and base-course aggregate (Keller & Sherar, 2003)

3.4.1 Blending methodology

The blending process involved systematically combining the two to three materials in various proportions to create a wide range of potential gradation curves. Each blend started with a baseline mix ratio of coarse aggregate, fine aggregate, and binder material. This initial ratio was determined based on previous studies and practical field experiences.

Proportions of the three components were adjusted incrementally to explore different gradation curves. The percentage of coarse aggregate was reduced in some trials to examine its impact on the finer portion of the gradation curve, while in others, the fines content was varied to assess its effect on overall gradation. Blends showing potential were further refined by making smaller adjustments to the component ratios. This step ensured the gradation curve moved closer to the desired range without exceeding the limits. Promising blends were replicated to confirm consistency in achieving the desired gradation. Any inconsistencies were addressed by re-evaluating the blending ratios and material properties.

To illustrate with an example, one of the successful blends consisted of 50% gravel (passing 1.5-inch retained on #4 sieve) content, 40% sand (passing No. 4 sieve retained on No. 200) portion and 10% of fines (passing No. 200) fractions. This combination was selected after multiple iterations. Initially, a 60-30-10 mix resulted in a gradation curve that was overly coarse, exceeding the lower limit of the gradation chart. By reducing the coarse aggregate and increasing the fine aggregate proportion, the adjusted 50-40-10 mix fell within the recommended range.

Chapter 4 Results and Discussion

This chapter presents the laboratory tests conducted to assess the properties and performance of the materials under investigation. Resilient modulus (M_R) tests are discussed for both virgin materials and the blends of materials within each county. The gradation curves of various blends with varying percentages of gravel, sand, and fines are examined to comprehend the materials size distribution. Blends are analyzed to evaluate their mechanical behavior under cyclic loading, while permanent deformation (PD) tests assess their resistance to rutting under repeated loading. Furthermore, tests conducted after FT evaluate the materials' durability under cyclic freezing and thawing. These comprehensive results provide a thorough evaluation of the materials' suitability and performance for gravel road applications.

4.1 Virgin materials

4.1.1 Resilient Modulus results of virgin materials

The M_R test was conducted following AASHTO standards as outlined in Section 3.3.1.1. Figure 4.1 illustrates the variations in gravel, sand, and fines content for the collected materials from Douglas County, while details for other counties are provided in Appendix A. This will assist in comprehending the current state of the materials. In Douglas County the virgin materials include open graded materials such as Clean 1.5 in., Clean 1 in., road gravel, crusher run 1.5 in. and ¾ in. The road gravel has a sand dominated composition while the crusher run contains a well graded mix of gravel, sand, and fines. The subgrade, composed of clay, has a fines content of 90%. Each material was prepared according to its corresponding gradation, with six layers compacted at optimum moisture content (OMC) to a 95% compaction degree following AASHTO procedures.

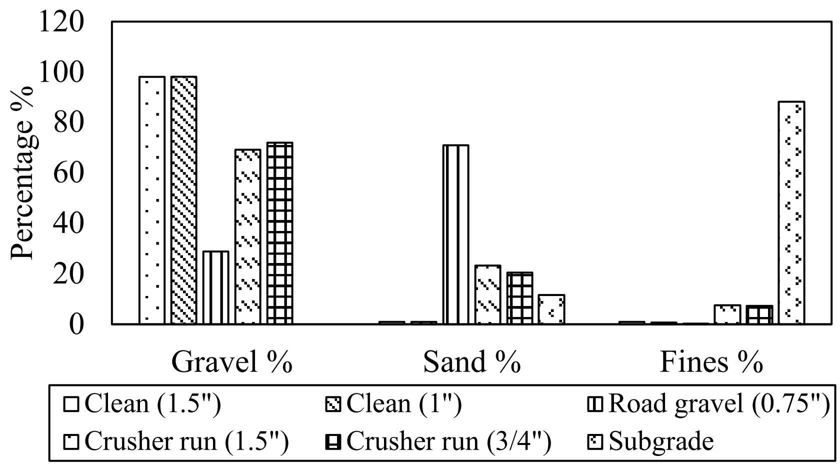


Figure 4.1 Variations in gravel, sand, fines content of virgin materials in Douglas County

The M_R test was conducted on both surface and subgrade materials using a controlled loading sequence. The test system recorded displacement, force, and cycle count data, which were stored in a text file. A custom computer algorithm was developed to extract key parameters, including deviatoric stress and strain values from the last five cycles of each sequence. These values were then used to compute the M_R for all 15 loading sequences. As shown in Figure 4.2, the M_R testing generated fifteen data points for each test, capturing the stiffness behavior of the materials comprehensively. These data points are essential for identifying trends in stiffness response under low, medium, and high traffic loads. Figure 4.2 provides an example of the virgin materials from Douglas County, illustrating how stiffness varies with bulk stress θ.

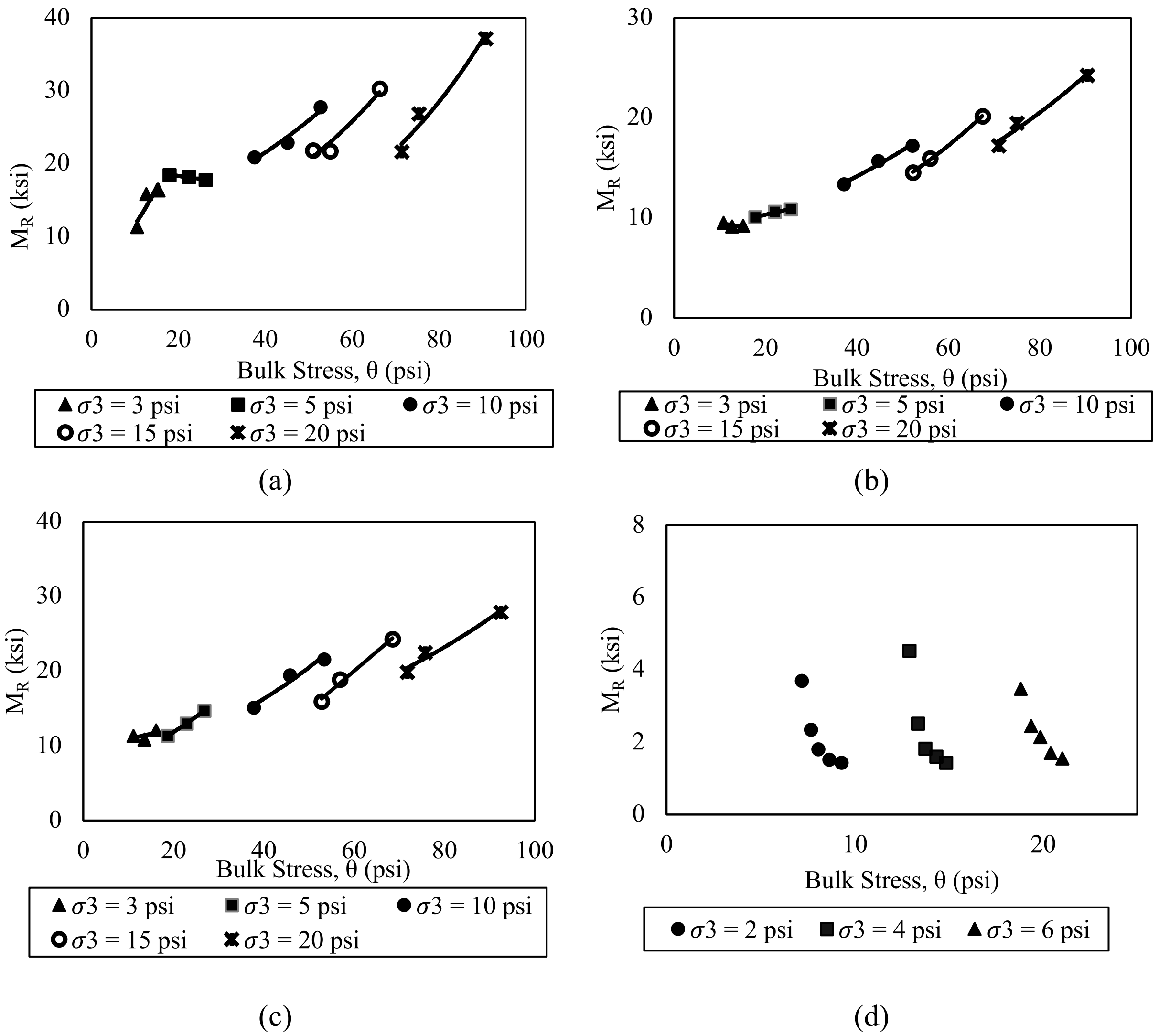


Figure 4.2 Resilient modulus varying with bulk stress, θ (a) DV1 – Clean (1.5”), (b) DV3 – Road Gravel, (c) DV4 – Crusher run (1.5”), (d) DV6 – Subgrade (Clay)

To generalize these trends, the Mechanistic-Empirical Pavement Design Guide (MEPDG) model (ARA, 2004) was used to determine the M_R characteristics of the materials using the elastic deformations recorded during the last five cycles of each testing sequence. This model is integrated into the AASHTOWare® Pavement ME software and has been widely validated in literature, demonstrating a high goodness-of-fit to M_R data. The adopted model can capture both the stiffening effect of bulk stress and the softening effect of shear stress. The goodness of fit must exceed 0.90 to properly find the values of the model constants according to Tutumluer (2013).

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \quad 4.1$$

where, θ is the bulk stress = ($\sigma_1 + \sigma_2 + \sigma_3$) or $\sigma_1 + 2\sigma_3$ (ksi), σ_1, σ_2, and σ_3 are the principal stresses (ksi) , τ_{oct} is the octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$ $= \frac{\sqrt{2}}{3}\,\sigma_d$ (ksi), P_a is atmospheric pressure, and k_l, k_2, k_3 are the regression constants.

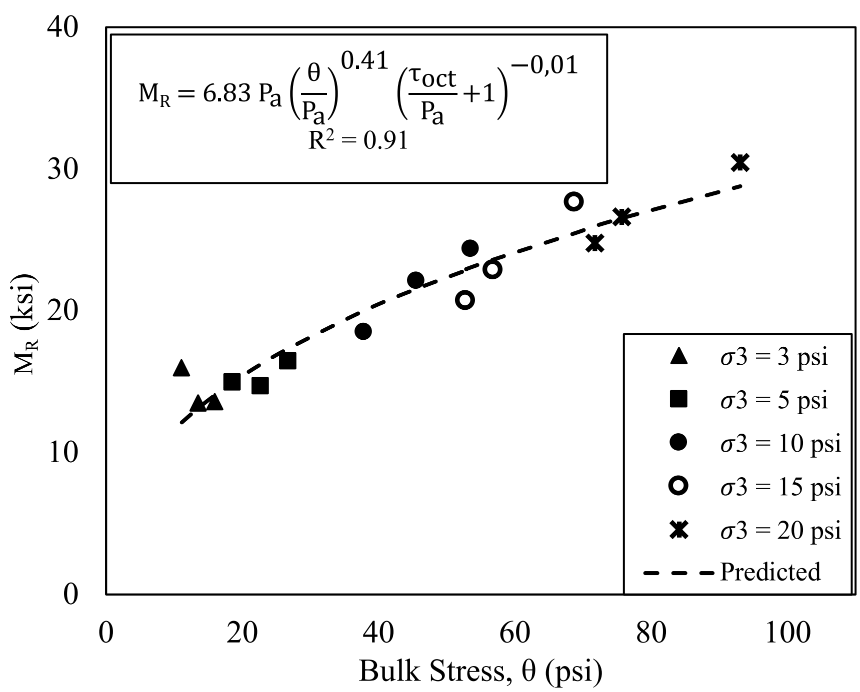


Figure 4.3 Bulk stress (θ) model fit for the (M_R) of Cherry - Section 27 (CV1).

Figure 4.3 presents the results of the Section 27 (CV1) material as a representative example, demonstrating the application of the bulk stress model. The figure showcases the model's fit to the data, displaying the model parameters and the coefficient of determination, which indicates the goodness-of-fit between the model and the observed data. This representation demonstrates the relationship between the bulk stress model and the M_R behavior of the material, serving as a reference for the remaining results.

The summary M_R (SM_R) value represents the M_R value to be used for pavement design. Among those 15 M_R values, a representative value, which is the summary resilient modulus (SM_R) value, is chosen for pavement design. As per NCHRP 1-28A study recommendations, for the base materials and some of the subbase materials, the bulk stress (θ) and the octahedral shear stress (τ_{oct}) values corresponding to the sixth sequence of the base/subbase testing sequences were used to calculate SM_R (θ = 30 psi and τ_{oct} = 7 psi, per NCHRP 1-28A recommendation). For the subgrade materials and some of the subbase materials, the stresses corresponding to the 13th sequence of the subgrade testing sequences were used to calculate SM_R (θ = 12 psi and τ_{oct} = 3 psi, per NCHRP 1-28A recommendation). It is noted that the NCHRP 1-28A study is the most comprehensive study to test unbound materials under different stress conditions for different pavement foundation layers.

Summary M_R results of surface and subgrade materials are provided in Table 4.1. Key parameters, regression coefficients k_1, k_2, and k_3, which characterize the non-linear behavior of the geomaterials under cyclic loading are also included. Figure 4.4 show the SM_R values of the Douglas County surface and subgrade materials. Detailed MR results of all other counties are provided in Appendix A.

Table 4.1 Summary M_R and model parameters for all virgin materials

County	Materials Name (Virgin)	Code	SM_R (ksi)	k_1	k_2	k_3	R^2
Douglas	Clean (1.5”)	DV1	19.5	1016.5	0.39	-0.05	0.75
	Clean (1”)	DV2	29.2	1620.0	0.31	-0.05	0.74
	Road gravel (0.75”)	DV3	12.6	610.5	0.48	-0.02	0.92
	Crusher run (1.5”)	DV4	15.0	740.7	0.46	-0.02	0.88
	Crusher run (3/4”)	DV5	17.3	843.6	0.47	-0.01	0.87
	Subgrade	DV6	5.0	486.3	-0.08	-2.02	0.85
Harlan	Crusher run (1” minus)	HV1	17.5	763.6	0.63	-0.01	0.95
	Green Surface Sand	HV2	10.6	483.4	0.58	-0.03	0.95
	Surface sand	HV3	NA	NA	NA	NA	NA
	Gravel	HV4	16.0	824.4	0.41	-0.05	0.77
	Subgrade (Silty loam)	HV5	5.8	937.8	0.28	-4.62	0.44
	Subgrade (Clay)	HV6	12.5	852.2	0	0	0.55
Scottsbluff	Rock (3/8”-1”)	SV1	13.4	612.3	0.56	-0.01	0.89
	Road Gravel (3/4”-1”)	SV2	16.3	844.9	0.41	-0.06	0.91
	Subgrade	SV3	10.9	834	-0.26	-0.93	0.29
Cherry	Section 27	CV1	18.2	926.3	0.41	-0.01	0.91
	Section 7	CV2	14.0	672.7	0.49	-0.02	0.81

k_1, k_2, k_3 = fitting parameters in Equation 4.1, SM_R = summary M_R, R^2= coefficient of determination

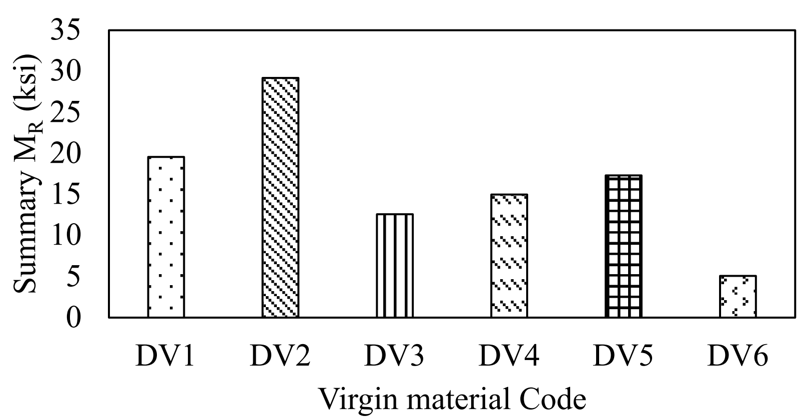


Figure 4.4 Summary M_R values for the Douglas County virgin materials

In surface materials, the open graded materials like Clean 1.5 in., Clean 1 in. from Douglas County, and Section 27 from Cherry County exhibited the highest SM_R values ranging from 18 ksi to 29 ksi. Among the surface materials, DV4 (Crusher run 1.5 in.), DV5 (Crusher run ¾ in.) and HV1 (Crusher run 1 in.) with similar gravel, sand, and fines percentage showed similar SM_R values varying from 15 ksi to 17.5 ksi.

The SM_R results of virgin surface materials indicate that materials with approximately 70% sand content, such as Green Sand Surface and Douglas Road Gravel, exhibited the lowest SM_R value of around 12 ksi. This outcome is expected, as the lack of plasticity reduces binding capacity, while the absence of larger particles limits structural interlocking, thereby diminishing load-bearing capacity. Additionally, in Harlan County, the Surface Sand (HV3) material, which is over 80% sand, failed the MR test despite multiple attempts, highlighting the importance of coarse aggregates in enhancing mechanical performance.

Among surface materials, road gravel samples such as HV4, SV2, and CV2, which share similar gradation characteristics, exhibited comparable SM_R values ranging from 14 ksi to 16 ksi.

In Figure 4.5, Figure 4.6, Figure 4.7**,** and Figure 4.8**,** the relationship between M_R and bulk stress for all surface materials were presented. A consistent increase in M_R with rising bulk stress was observed across all specimens, aligning with typical granular material behavior.

In the regression model parameters, k_1 represents the initial stiffness of the material, k_2 indicates changes in M_R with confining pressure (bulk stress), and k_3 reflects the influence of shear stress—where negative values suggest reduced stiffness under higher shear loads.

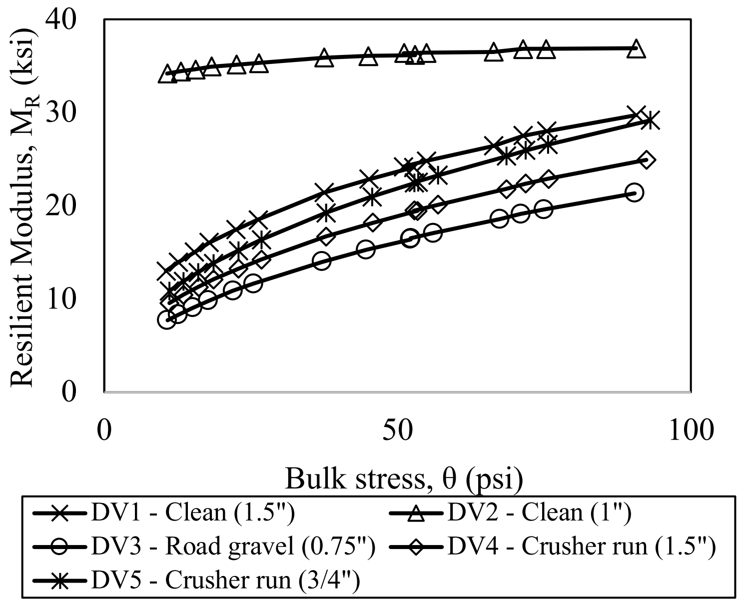


Figure 4.5 M_R vs θ, model fit results of Douglas County virgin materials

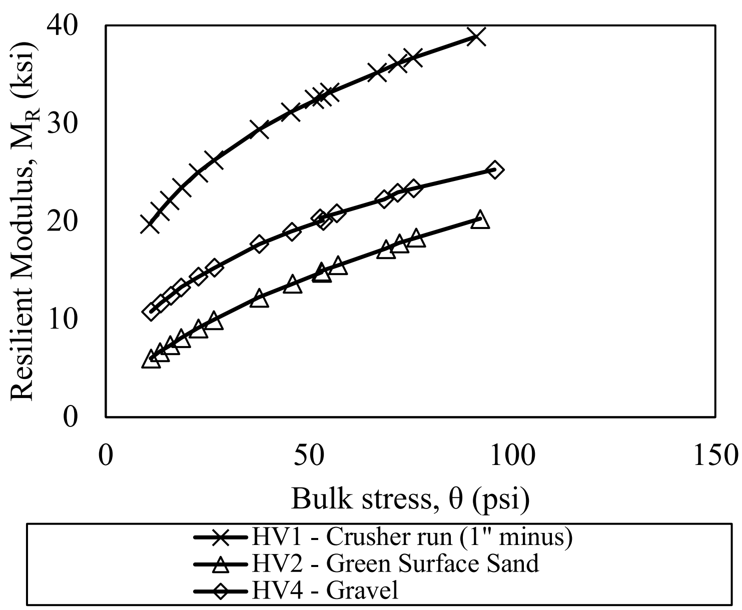


Figure 4.6 M_R vs θ, model fit results of Harlan County virgin materials

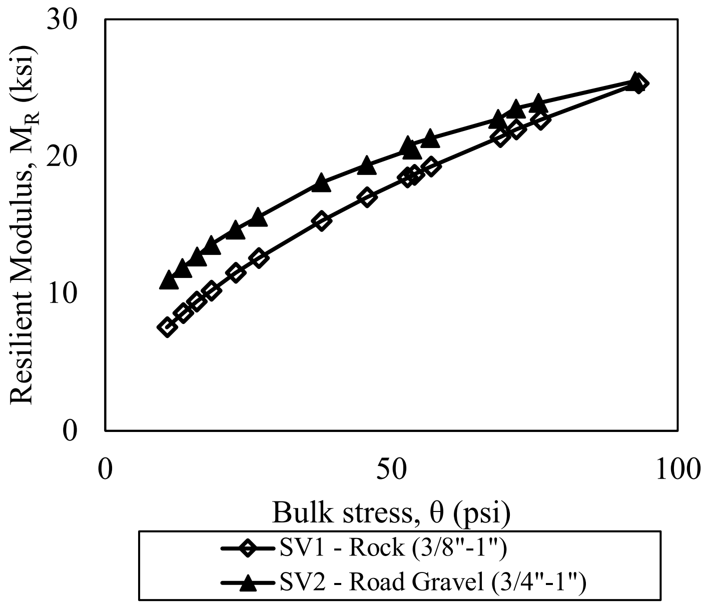


Figure 4.7 MR vs θ, model fit results of Scottsbluff County virgin materials

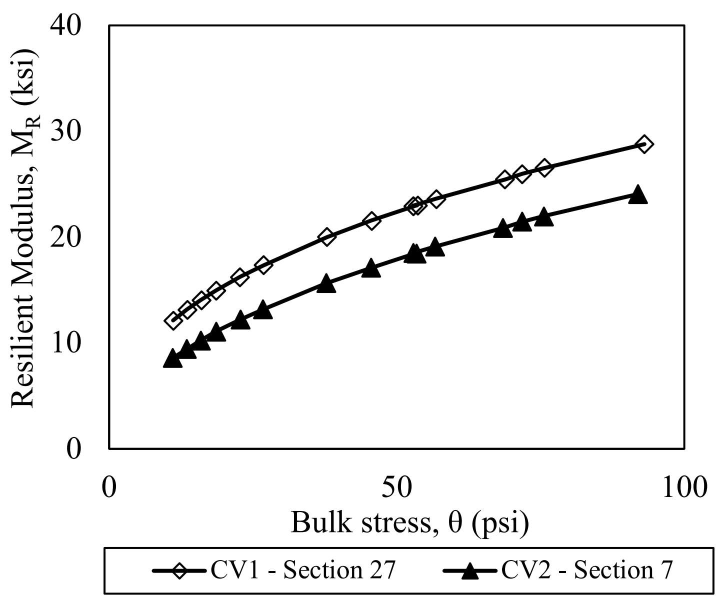


Figure 4.8 MR vs θ, model fit results of Cherry County virgin materials

The k_1 coefficient, which defines the initial M_R, varies significantly among materials. Douglas Clean 1 in. (DV2) exhibited the highest k_1 value, indicating superior stiffness, whereas Douglas Subgrade had the lowest k_1, reflecting weaker load-bearing capacity. Most granular materials demonstrated moderate k_1 values, highlighting their effective load distribution, while subgrades exhibited greater variability.

The k_2 parameter, representing stress sensitivity, was positive for most aggregates, indicating sustained or improved stiffness under increasing stress. However, certain subgrades, such as Douglas Subgrade (DV6) and Scottsbluff Subgrade (SV3), had negative k_2 values, signifying stress-softening behavior where strength diminishes under repeated loading. Similarly, the k_3 parameter (bulk stress exponent) was slightly negative for most materials, indicating a gradual modulus reduction with increasing bulk stress. Extreme negative values, such as Harlan Subgrade (HV5) (-4.62) and Douglas Subgrade (DV6) (-2.02), suggest rapid stiffness deterioration under high-stress conditions.

The goodness-of-fit (R^2) values for the M_R model were generally above 0.75, indicating strong correlations for most materials. However, exceptions such as the Scottsbluff Subgrade (SV3) ($R^2 = 0.29$) and Harlan Subgrade (HV6) ($R^2 = 0.55$) suggest inconsistent behavior, likely due to material heterogeneity or testing variability.

The surface materials, while demonstrating relatively better performance, exhibited a key limitation due to the lack of sufficient fine sand and fine-grained materials. Despite their high stiffness, the open-graded virgin surface materials pose challenges in constructability, drainage, and durability, making them susceptible to raveling and surface deterioration, which are undesirable for road surfacing.

4.1.2 Permanent deformation results of virgin materials

Testing was planned with the aim of characterizing the development of cumulative permanent strain with number of load applications and its variation with stresses. The repeated load triaxial test for the virgin surface materials was carried out in accordance with AASHTO T 307-99 (1999) procedures. Specimens were compacted and tested following the procedure mentioned in Section 3.3.2.1. The tests were executed with a target of 10,000 load cycles. The granular surface materials samples present quasi-sine-shaped deformation curves due to the sine-shaped cyclic stress. Both elastic and PD existed in each loading cycle. The elastic deformation was recovered in tens of thousands of load times, while the PD is accumulated gradually.

The permanent strain can be plotted against loading cycles to illustrate the long-term deformation properties of these materials. Table 4.2 shows PD results of all virgin surface materials at a constant confining pressure of 4 psi for a cyclic stress of 25.2 psi.

Table 4.2 Permanent deformation results of virgin surface materials

Material Name	**Code**	**Plastic strain (%)**	**PD Range**
Clean (1.5")	DV1	1.43	C
Clean (1")	DV2	1.07	C
Road gravel (0.75")	DV3	NA	NA
Crusher run (1.5")	DV4	1.35	C
Crusher run (3/4")	DV5	1.64	C
Crusher run (1"-)	HV1	0.42	B
Green Surface Sand	HV2	NA	NA
Surface sand	HV3	NA	NA
Gravel	HV4	1.38	C
Rock (3/8"-1")	SV1	0.9	C
Road Gravel (3/4"-1")	SV2	NA	NA
Section 27	CV1	1.32	C
Section7	CV2	NA	NA

NA – Not available (test failed)

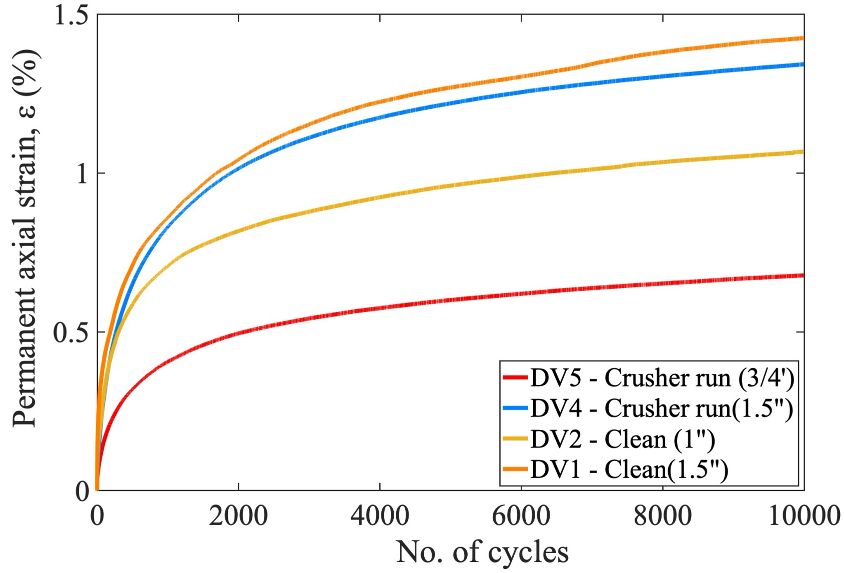


Figure 4.9 Cumulative permanent axial strain versus number of load repetitions for Douglas County virgin surface materials

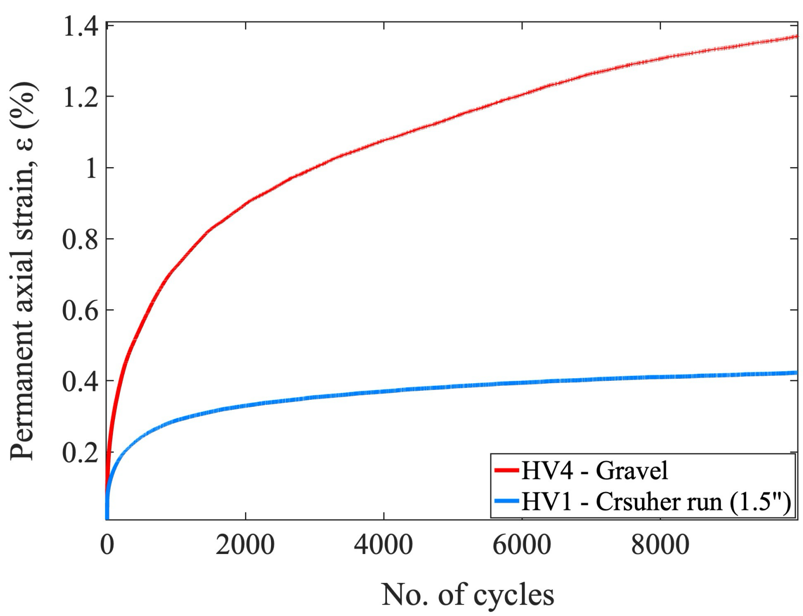


Figure 4.10 Cumulative permanent axial strain versus number of load repetitions for Harlan County virgin surface materials

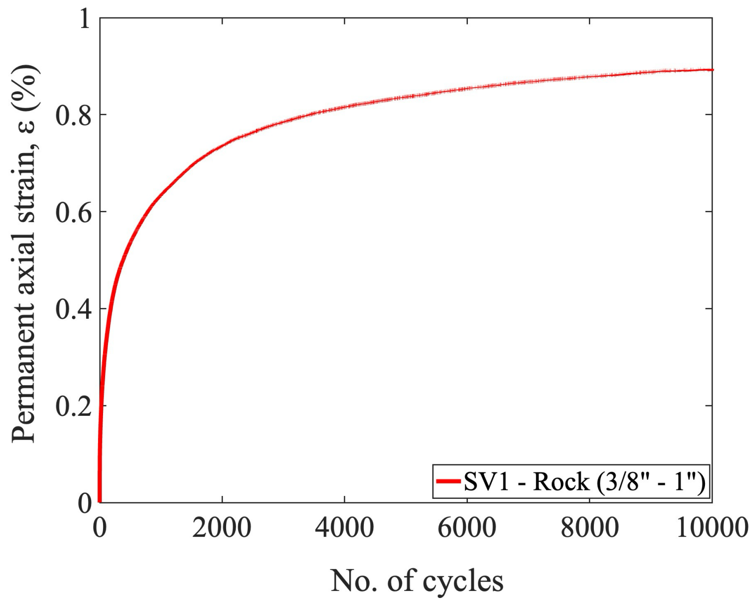


Figure 4.11 Cumulative permanent axial strain versus number of load repetitions for Scottsbluff County virgin surface materials

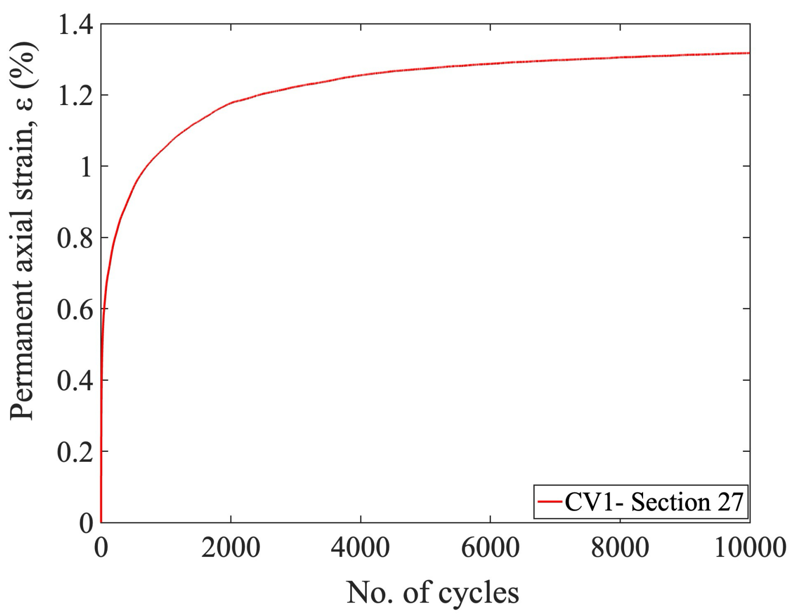


Figure 4.12 Cumulative permanent axial strain versus number of load repetitions for Cherry County virgin surface materials

Sharp (1983) first introduced this theory for analyzing the deformation behavior of pavements. The shakedown theory has been extensively adopted in characterizing the response of granular materials under cyclic loading. Based on shakedown theory, this approach categorizes materials into three shakedown ranges based on their permanent strain response. Initially, Range A is in the plastic shake down range. Here, the response is plastic for a finite number of load applications, but after completion of the post compaction period the response becomes entirely resilient and no further permanent strain occurs. In Range B, permanent strain accumulation continues at a slower rate, indicating non-critical deformation and incomplete stabilization. This is called a plastic creep range, where gradual creeping occurs under cyclic loading. This can be managed but requires attention. Range C, incremental collapse phase, is characterized by continuous and significant permanent strain accumulation without stabilization, increasing the likelihood of structural failure.

Table 4.3 presents the Werkmeister criterion to understand the performance of surface materials used in this study.

Table 4.3 Shakedown theory ranges

Ranges	Werkmeister et al., 2001
Range A – plastic shakedown	$\varepsilon_{p,5000} - \varepsilon_{p,3000} < 4.5 * 10^{-5}$
Range B – plastic creep	$4.5 * 10^{-5} < \varepsilon_{p,5000} - \varepsilon_{p,3000} < 4.5 * 10^{-4}$
Range C – incremental collapse	$\varepsilon_{p,5000} - \varepsilon_{p,3000} > 4.5 * 10^{-4}$

As per the literature, Werkmeister criterion was used to evaluate the performance of all the materials, if they are in Range A and Range B, these materials are accepted as high quality granular surface materials, while Range C behavior should not occur. However, it is crucial to recognize the limitations of the criteria, particularly in long-term cyclic loading scenarios, to ensure reliable performance predictions.

All samples have experienced high initial permanent strain in the first stage followed by a low increasing rate in permanent strain. As seen in Table 4.2, permanent strain (%) for all the virgin materials ranged from 0.42% to 1.64%.

Figure 4.9, Figure 4.10, Figure 4.11, and Figure 4.12 show the permanent axial strain versus the number of virgin material loading cycles from four counties respectively. It can be seen that with an increasing number of cycles, the magnitude of accumulated permanent strains increases. For Douglas County surface materials with a high percentage of gravel (Clean 1.5", Clean 1") exhibit relatively lower plastic strain values (1.43% and 1.07%, respectively). This may be attributed to the coarser particles bearing most of the applied load. However, these materials fall under Range C, which is associated with incremental collapse and is therefore not recommended. On the other hand, Road Gravels with a higher sand content from Douglas and Scottsbluff counties (70.9% sand) tend to fail the test. Since the particles are dominated by fines, the load carrying capacity is less. This also indicates the materials need to have a well-defined gradation to get the optimal strength and resistance to PD.

The Harlan crusher run has a well graded gradation curve showing the highest resistance to PD. This behavior could be due to an adequate amount of gravel, sand, and fines portions, or having a better particle packing and interlocking.

Examining the PD ranges, most materials with recorded accumulated strain values fell into category "C", which is likely in an undesirable performance range. Additionally, materials that failed the PD test are consistently associated with high fines or sand percentages, implying weaker structural integrity.

After conducting the mechanical tests and analyzing the results, almost all virgin materials collected exhibited poor performance, which can impact the granular roads by incurring severe distresses. Some kind of stabilizing is needed to enhance the performance of the granular surface materials. Therefore, the gradation curves are modified by incorporating different materials in various proportions to have a well graded mix of all the particle sizes which can affect the gravel road performance. From a thorough analysis of previous studies, gradation curves were strategically modified by blending different materials in varying proportions to achieve a well-graded mix of full-range of particle sizes. This optimized gradation enhances gravel road performance by improving load distribution, increasing stability, and minimizing issues such as rutting, washboarding, and material loss.

4.2 Blending of materials

As outlined in Section 3.4.1, materials from four different counties were blended within each county to create mixtures comprising gravel, sand, and fines in specific proportions. These materials were carefully selected and combined to achieve an optimized gradation, ensuring appropriate material properties for the intended application. The percentage composition was determined based on literature recommendations to achieve a well-balanced gradation.

Following multiple trials, a total of 10 blends were finalized in Douglas County. Harlan County contributed 12 blends, Scottsbluff County produced five blends, and Cherry County provided four blends. To ensure clear identification, each blend was assigned a unique code. The first letter corresponds to the county's initial, while the number represents the specific blend sequence within that county. For example, D1 indicates the first blend from Douglas County. Details of Douglas County blends (D1–D10) are provided in Table 4.4**,** Table 4.5**,** and Table 4.6, while blends from the remaining locations are detailed in Appendix B.

To examine variations in test results, blends were designed with a decreasing gravel content while increasing the proportions of sand and fines. Based on an extensive literature review, each blend was formulated with a minimum gravel content of 25%, a sand content not exceeding 60%, and a fines content maintained between 5% and 20%.

Blends within Douglas County were categorized into three groups based on their primary gravel sources. In the first group, blends D1, D2, and D3 incorporated Clean 1.5-inch material (~>90% gravel) as the primary gravel component, while road gravel and subgrade materials contributed to the required sand and fines fractions. Similarly, blends D4, D5, and D6 used Clean 1-inch material as the gravel source. The final group, consisting of blends D7 to D10, utilized crusher-run 1.5-inch and 3/4-inch materials combined with road gravel and subgrade to ensure a balanced distribution of gravel, sand, and fines.

Figure 4.13, Figure 4.15, and Figure 4.17 shows the variations in gravel, sand, and fines of all Douglas blends. The obtained gradation curves with the specified upper and lower gradation limits are shown in Figure 4.14, Figure 4.16, and Figure 4.18.

Table 4.4 Details of Douglas County blends D1, D2, D3

Code	**Materials**	**Gravel (%)**	**Sand (%)**	**Fines (%)**
D1	Clean (1.5") [30%] + RG [62%] + Sub [8%]	47.36	45.18	7.46
D2	Clean (1.5") [20%] + RG [66%] + Sub [14%]	38.71	48.62	12.67
D3	Clean (1.5") [5%] + RG [75%] + Sub [20%]	26.60	55.57	17.84

RG = Road Gravel, Sub= Subgrade (Clay)

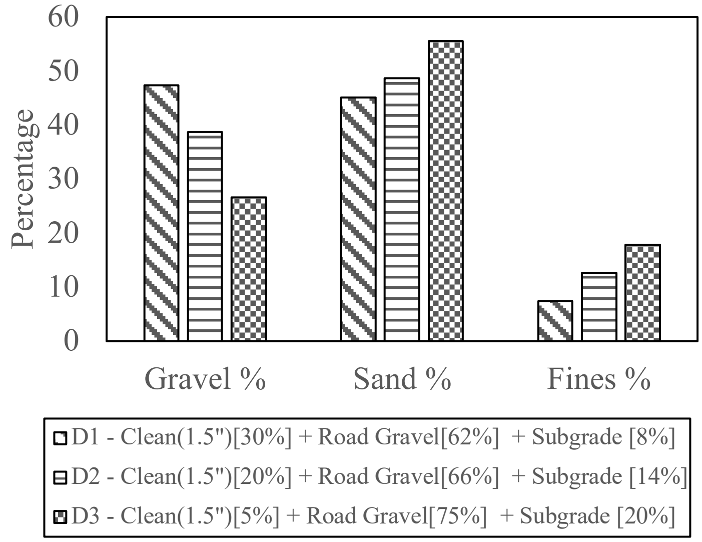


Figure 4.13 Gravel, Sand, Fines proportions of Douglas County blends (D1, D2, D3)

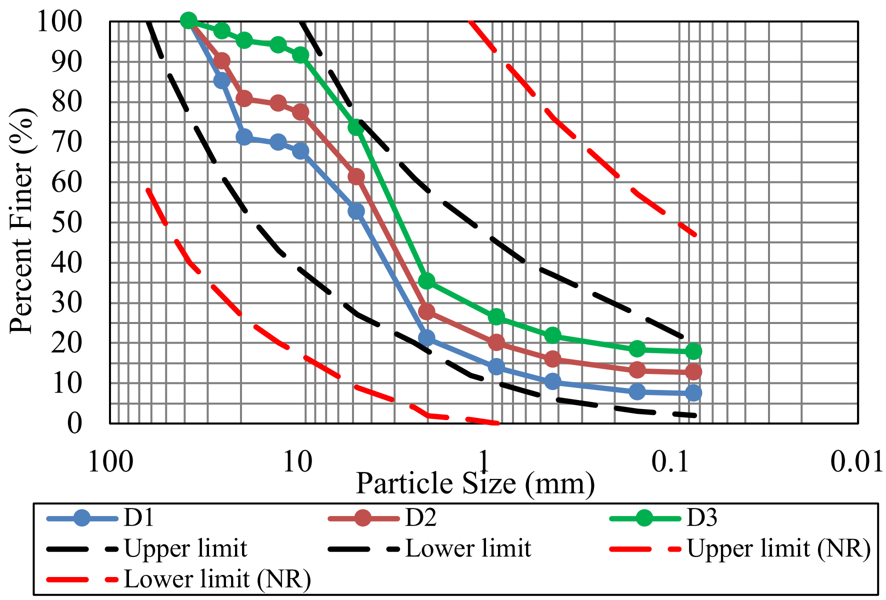


Figure 4.14 Gradation curves of the blends D1, D2, D3 with gradation recommended limits

Table 4.5 Details of Douglas County blends D4, D5, D6

Code	Materials	Gravel (%)	Sand (%)	Fines (%)
D4	Clean (1") [30%] + RG [68%] + Sub [8%]	47.38	45.20	7.42
D5	Clean (1") [20%] +RG [66%] + Sub 14%]	38.72	48.64	12.64
D6	Clean (1") [5%] + RG [75%] + Sub [20%]	26.60	55.57	17.83

RG = Road Gravel, Sub= Subgrade (Clay)

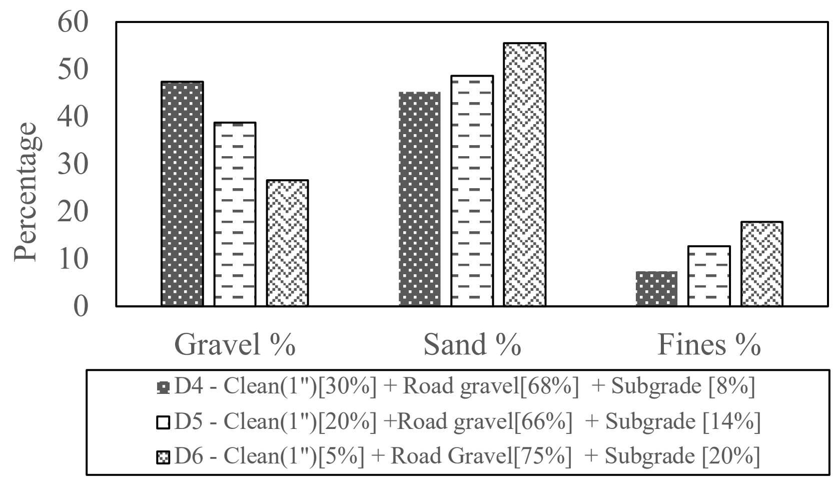


Figure 4.15 Gravel, Sand, Fines proportions of Douglas County blends D4, D5, D6

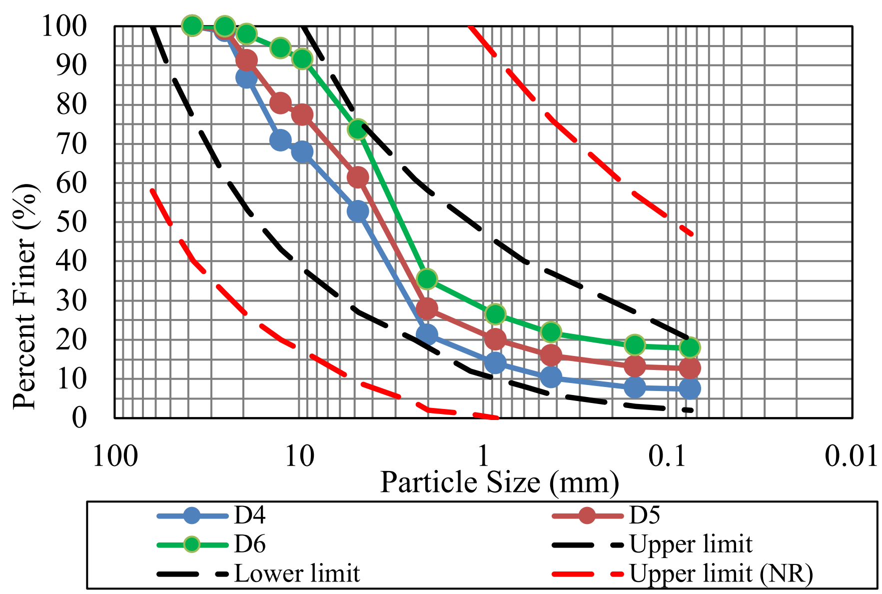


Figure 4.16 Gradation curves of the blends D4, D5, D6 with recommended limits

Table 4.6 Details of Douglas County blends D7, D8, D9, D10

Code	Materials	Gravel (%)	Sand (%)	Fines (%)
D7	RG (0.75") [10%] + CR (1.5") [90%]	65.18	28.02	6.80
D8	RG (0.75") [24%] + CR (1.5") [70%] + Sub [6%]	55.39	34.00	10.62
D9	RG (0.75") [36%] + CR (3/4") [50%] + Sub [14%]	46.48	37.46	16.06
D10	RG (0.75") [55%] + CR (3/4") [25%] + Sub [20%]	33.94	46.49	19.57

RG = Road Gravel, CR = Crusher run, Sub = Subgrade (Clay)

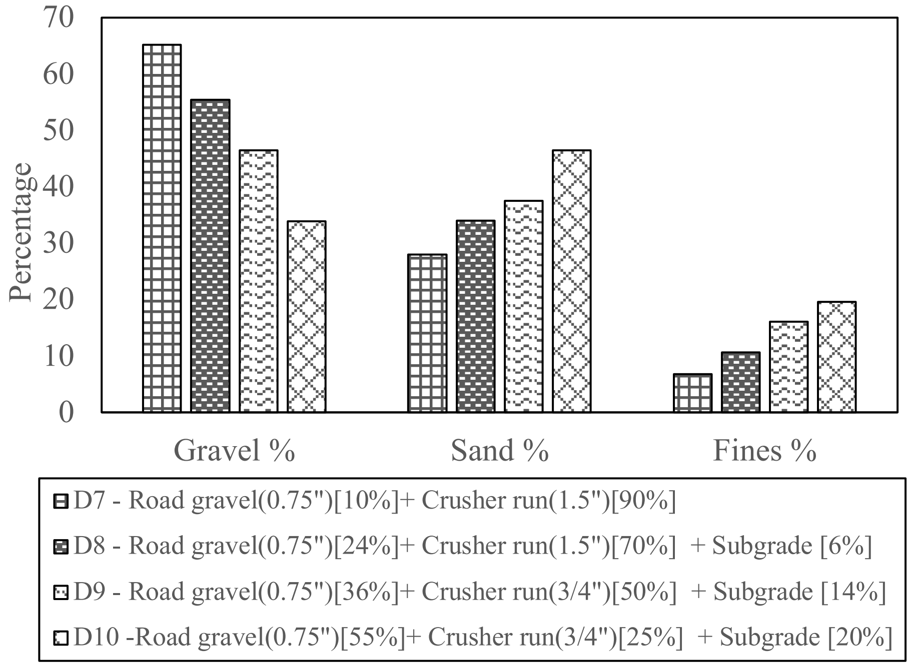


Figure 4.17 Gravel, Sand, Fines proportions of Douglas County blends (D7, D8, D9, D10)

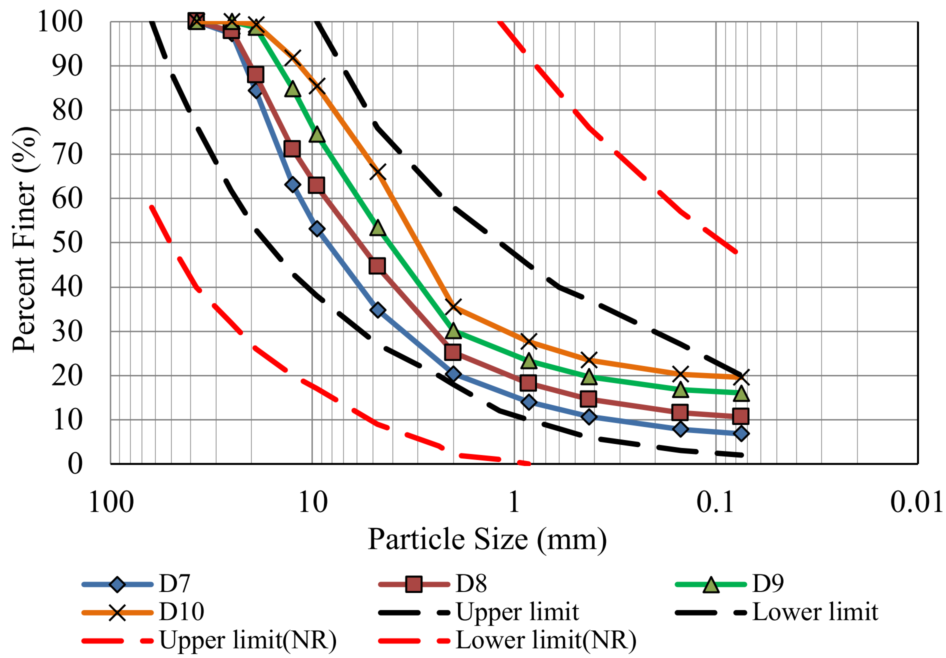


Figure 4.18 Gradation curves of the blends D7, D8, D9, D10 with recommended limits

4.2.1 Compaction test results of blends

The proctor compaction test was conducted to determine the maximum proctor dry unit weight (MDU) and corresponding optimum moisture content (OMC) of all the selected blends. The compaction test is conducted following Method C of AASHTO T 99-22 (2022).

To ensure strict compliance with the chosen grain size distribution, the material was prepared using 12 different fraction sizes. All materials in a blend were individually prepared and thoroughly mixed before testing to ensure uniformity and consistency. As an example, in Figure 4.19, material prepared to obtain blend D10 is provided.

Figure 4.19 Example material preparation of blend - D10 for proctor compaction test (a). DV5 (crusher run 3/4in), (b). DV3 (road gravel), (c). DV6 (subgrade) and (d). blended material

Table 4.7 presents the compaction results for the selected Douglas County blends (D1 to D10). These results include key parameters such as maximum dry unit weight (MDU) and optimum moisture content (OMC), which were determined through Proctor compaction tests. The values provide insight into the compaction characteristics of each blend, helping to assess their suitability for engineering applications.

To visualize the compaction behavior, Proctor compaction curves for these blends are illustrated in Figure 4.20, Figure 4.21, and Figure 4.22. These curves depict the relationship between moisture content and dry density, showing how each blend responds to compaction at varying moisture levels. The graphical representation aids in identifying trends across different blends and understanding how changes in gravel, sand, and fines content influence compaction performance. While this section focuses on Douglas County blends, the compaction results for blends from Harlan, Scottsbluff, and Cherry Counties are provided in Appendix B for reference. These results follow the same methodology, ensuring consistency in data analysis across all locations.

Table 4.7 Compaction results of all Douglas County blends

Blend code	MDU (pcf)	OMC (%)	Corrected MDU (pcf)	Corrected OMC (%)
D1	133.9	6.8	140.8	5.4
D2	133.7	7	138.3	6.0
D3	134.1	6.7	NA	NA
D4	135.6	6.2	138.5	5.6
D5	138.2	6.6	140.0	6.1
D6	133.0	7.1	NA	NA
D7	138.6	6.1	141.8	5.5
D8	141.2	6.4	143.4	5.9
D9	136.7	7.3	NA	NA
D10	133.9	6.8	NA	NA

NA – Not applicable, MDU – Maximum dry unit weight, OMC – Optimum moisture content

The Maximum Dry Unit (MDU) values for the Douglas County blends ranged from 134 pcf to 144 pcf, with the corresponding Optimum Moisture Content (OMC) values varying between 6.1% and 7.3%. In comparison, for the Harlan County blends, the MDU values ranged from 125 pcf to 135 pcf, with OMC values spanning from 6.6% to 8.5%. The MDU values for the Scottsbluff County blends ranged between 127 pcf and 136 pcf, while the OMC values varied from 6.85% to 7.8%. For Cherry County, the MDU values ranged from 134 pcf to 139 pcf, and the OMC values fluctuated between 6.5% and 7.8%. Notably, the MDU values for the blends were generally higher and more consistent compared to the virgin materials, indicating better particle packing within the blends. This suggests the blends achieved a more efficient arrangement of particles, which is beneficial for the compaction and overall stability of material.

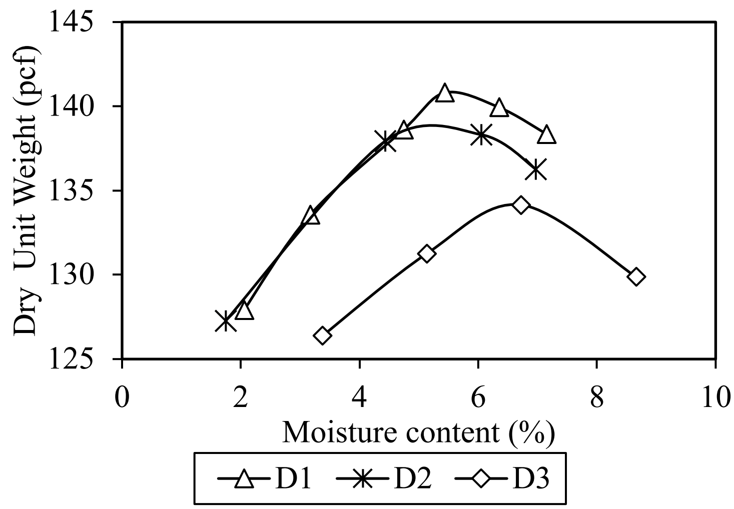


Figure 4.20 Compaction curves for the blends D1, D2, D3

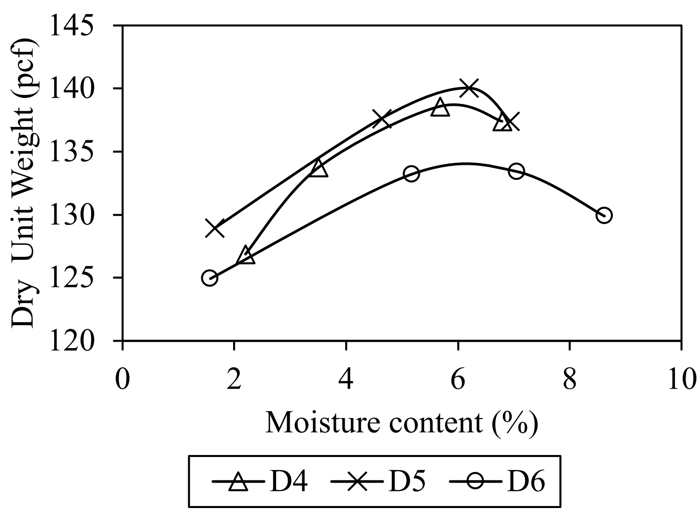


Figure 4.21 Compaction curves of the blends D4, D5, D6

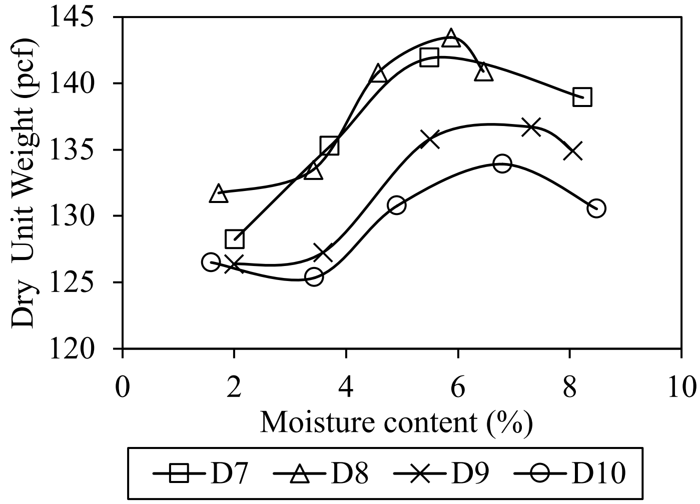


Figure 4.22 Compaction curves of the blends D7, D8, D9, D10

Overall, the compaction results showed a significant increase in MDU values, indicating enhanced packing and particle interlocking within the material. This improvement positively impacts load distribution, leading to better overall performance.

4.2.2 Resilient Modulus results of blends

This section examines the M_R values of different material blends, highlighting the influence of gradation properties, index properties, and material composition on their performance. The analysis aims to identify trends in stiffness characteristics, assess the impact of gravel, sand, and fines content, and establish correlations between M_R and other geotechnical properties. Understanding these relationships is essential for optimizing blend compositions to enhance pavement durability and resistance to deformation.

The tests were conducted as per the AASHTO standards stated in Section 3.3.1.1. The results provided in Table 4.8 represent the M_R values for various gravel road blends, calculated by using the MEPDG (Mechanistic-Empirical Pavement Design Guide) model. The table includes the SM_R, model coefficients (k_1, k_2, k_3), and the coefficient of determination (R^2) for each blend. These results illustrate the mechanical behavior and performance of the blends under traffic loads.

Table 4.8 Resilient Modulus (M_R) results of all blends

Blend Code	SM_R (ksi)	k_1	k_2	k_3	R^2
D1	17.7	881.4	0.44	-0.01	0.77
D2	23.9	1097.8	0.56	-0.02	0.89
D3	20.9	1237.5	0.2	-0.01	0.25
D4	19.1	1008.3	0.36	-0.02	0.82
D5	10.9	520.9	0.51	-0.03	0.8
D6	13.9	734.8	0.36	-0.01	0.75
D7	24.8	1199.4	0.48	-0.02	0.9
D8	20.1	1102.9	0.31	-0.01	0.75
D9	21.6	1211.7	0.28	-0.02	0.59
D10	20.4	1075.2	0.37	-0.02	0.79
H1	17.5	763.7	0.63	-0.01	0.95
H2	19.5	1088.5	0.28	-0.01	0.49
H3	20.6	1210.7	0.21	-0.01	0.28
H4	10.5	469.8	0.59	-0.02	0.99
H5	17.8	877.1	0.46	-0.02	0.9
H6	13.6	601.3	0.62	-0.04	0.93
H7	9.5	393.0	0.70	-0.01	0.85
H8	7.6	336.8	0.61	-0.02	0.78
H9	12.9	527.3	0.71	-0.01	0.87
H10	20.8	1174.7	0.28	-0.03	0.6
H11	11.1	488.6	0.63	-0.04	0.9
H12	11.9	540.8	0.58	-0.04	0.93
S1	20.7	999.9	0.48	-0.01	0.96
S2	23.2	1134.8	0.47	-0.02	0.98
S3	17.8	769.4	0.64	-0.01	0.92
S4	24.3	1139.3	0.53	-0.02	0.95
S5	21.5	1021.4	0.54	-0.07	0.94
C1	27.3	1483.2	0.32	-0.01	0.84
C2	23.0	1262.1	0.32	-0.03	0.73
C3	22.0	1038.9	0.53	-0.04	0.88
C4	18.2	963.8	0.36	-0.02	0.83

From the M_R results, Cherry County blends exhibited the highest SM_R values, particularly in C1 (27.3 ksi), suggesting high strength and stiff material composition. This can be attributed to a well-balanced combination of gravel and sand with controlled fines content. Clay, the material used to incorporate fines in this blend, has the highest PI among all materials, which is likely the reason for this increase.

The SM_R values for Douglas County were between 11 ksi and 25 ksi. The consistency and similarity of the results among all the blends from D1 to D10 are observed. The lower SM_R values were obtained from blends D5 and D6. Figure 4.23 shows the variation of SM_R in the Douglas blends. For Harlan County, the SM_R values had a wide range from 7.6 ksi to 20.9 ksi. Upon visual inspection, the gravel material was noted as flaky and relatively light weighted, which may be the reason for the reduced stiffness behavior of the materials. Also, the fines used are not as plastic in nature. Blends S1 to S5 have better stiffness resistance with SM_R values varying from 17.8 ksi to 24.3 ksi. Figure 4.24 through 4.31 shows the M_R model fit results for all blends, which are presented based on their respective counties.

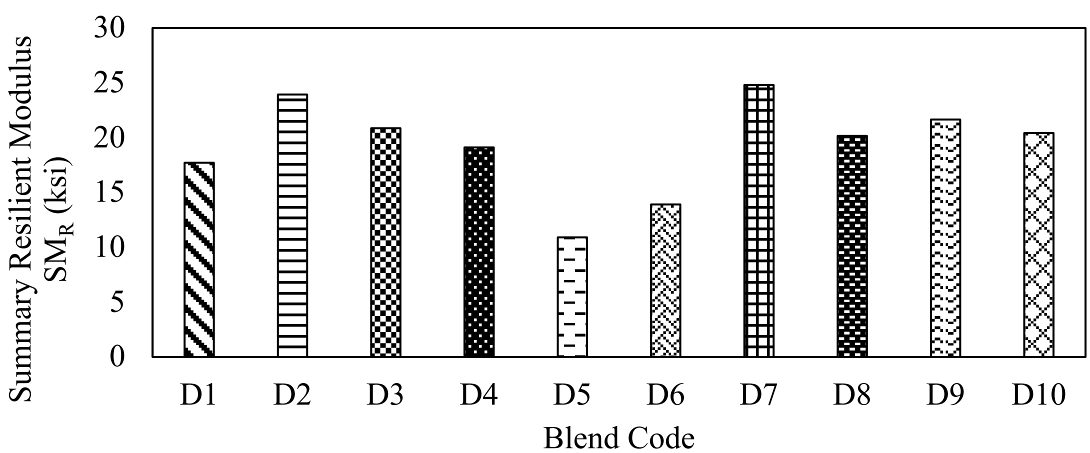


Figure 4.23 Summary M_R results for all Douglas blends.

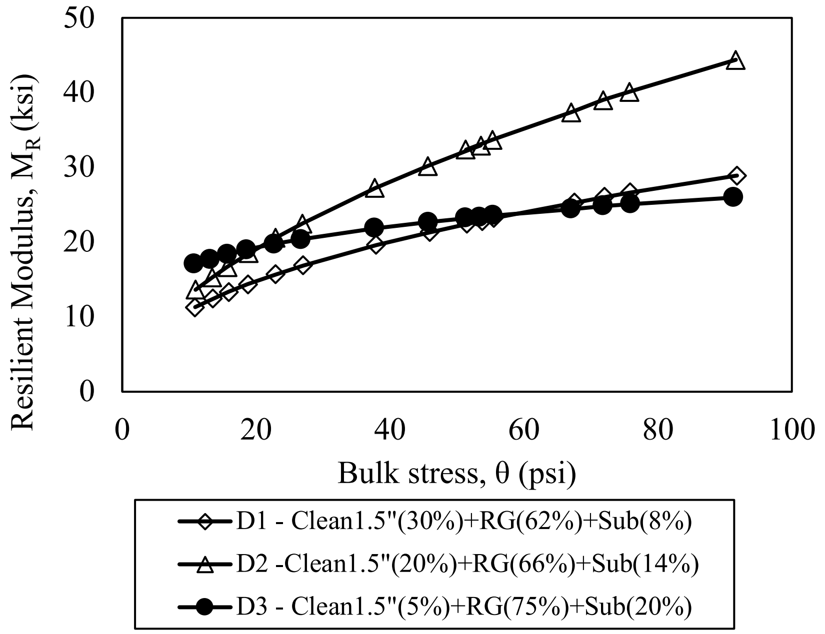


Figure 4.24 M_R vs θ, model fit results of Douglas County blends – D1, D2, and D3.

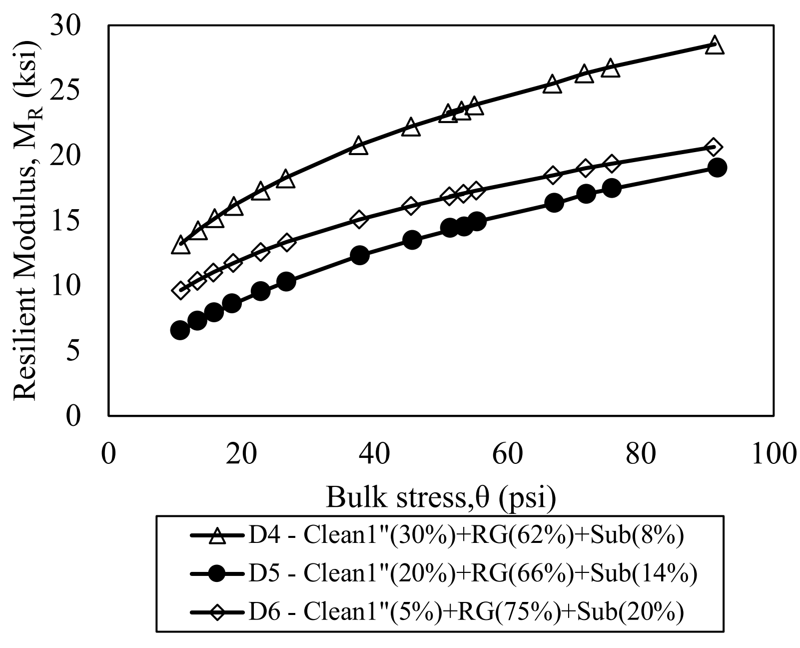


Figure 4.25 M_R vs θ, model fit results of Douglas County blends – D4, D5, and D6.

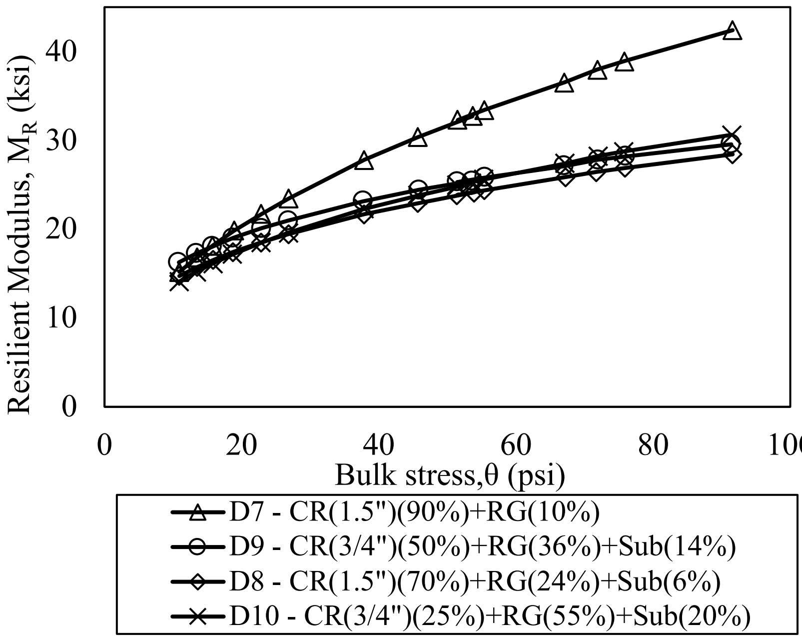


Figure 4.26 M_R vs θ, model fit results of Douglas County blends – D7, D8, D9, and D10.

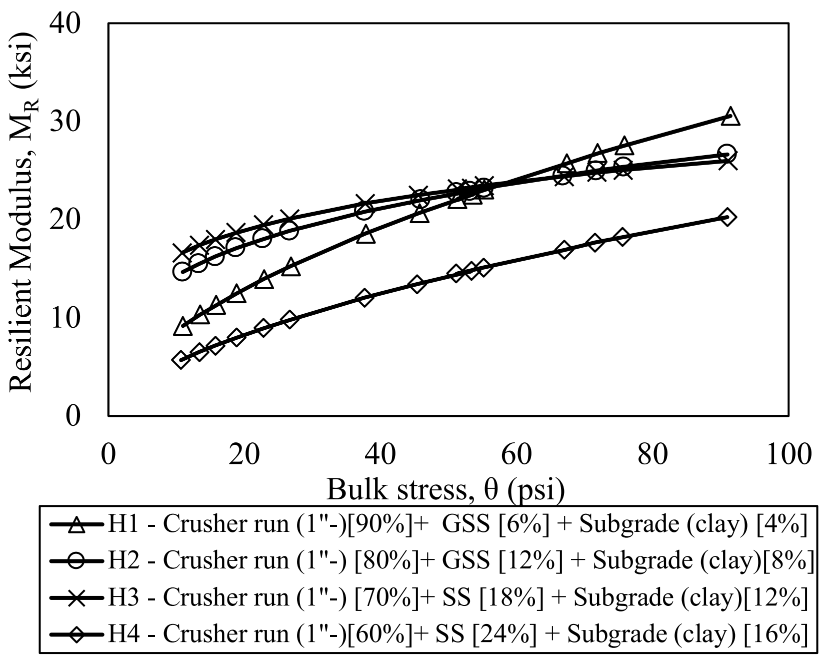


Figure 4.27 M_R vs θ, model fit results of Harlan County blends – H1, H2, H3, and H4.

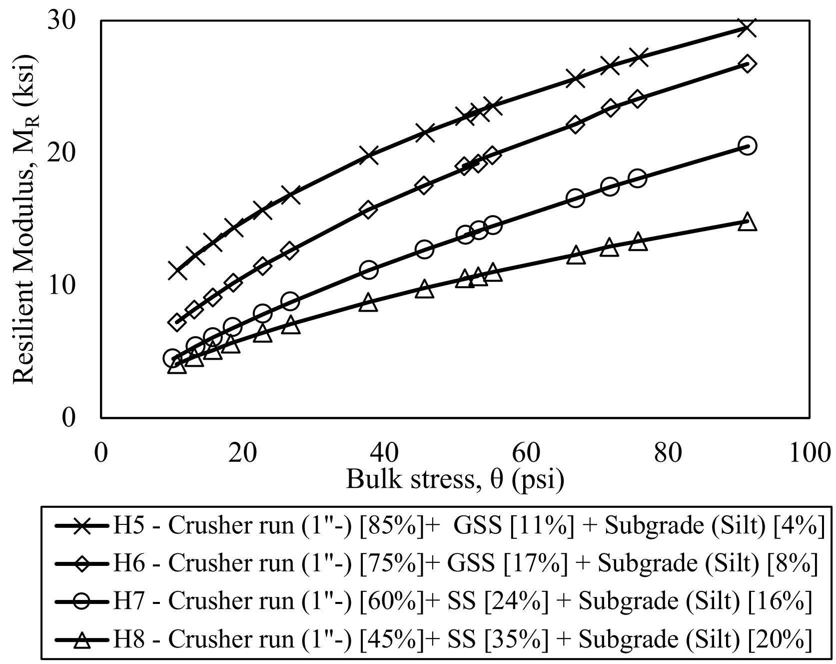


Figure 4.28 M_R vs θ, model fit results of Harlan County blends – H5, H6, H7, and H8.

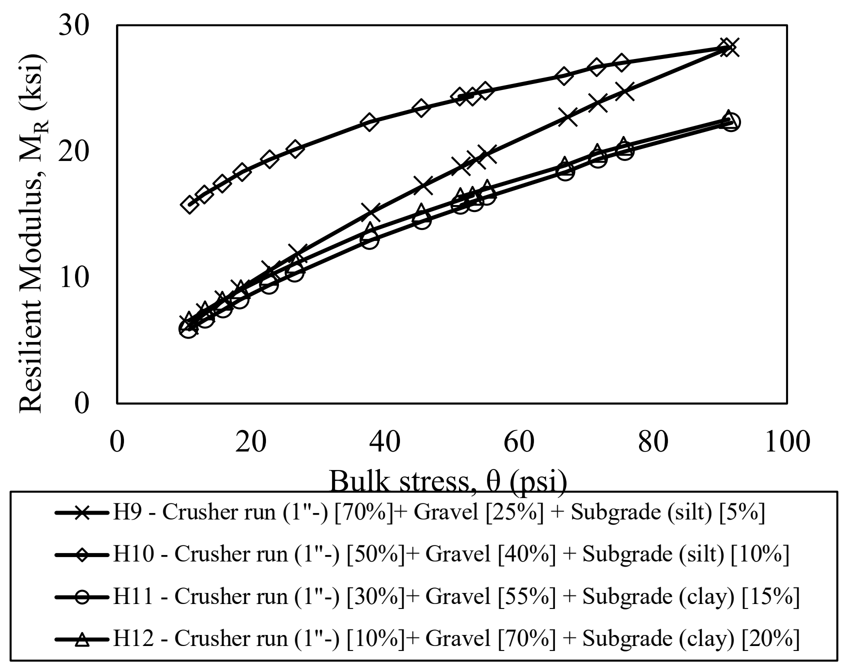


Figure 4.29 M_R vs θ, model fit results of Harlan County blends – H9, H10, H11, and H12.

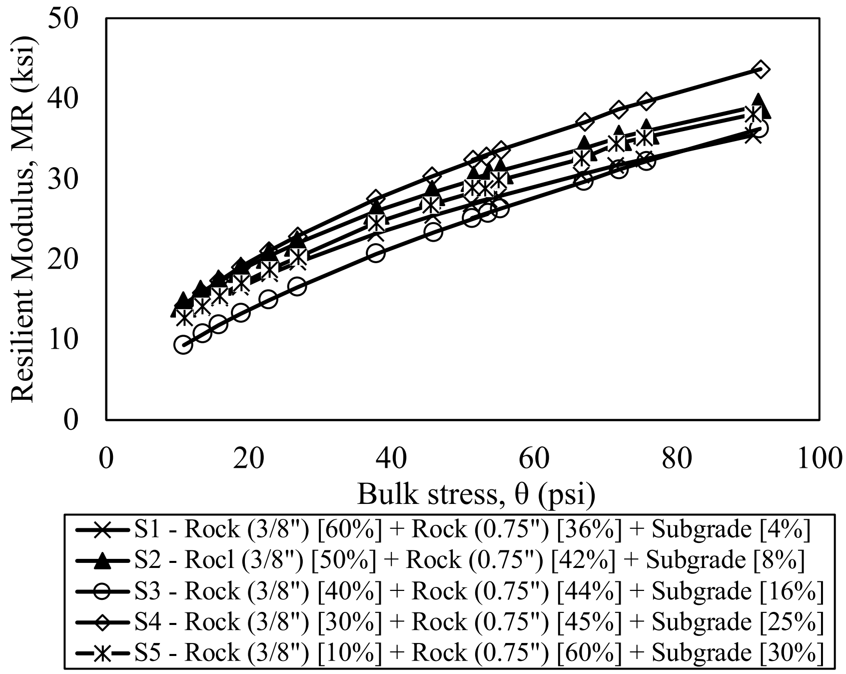


Figure 4.30 M_R vs θ, model fit results of Scottsbluff County blends – S1, S2, S3, S4 and S5.

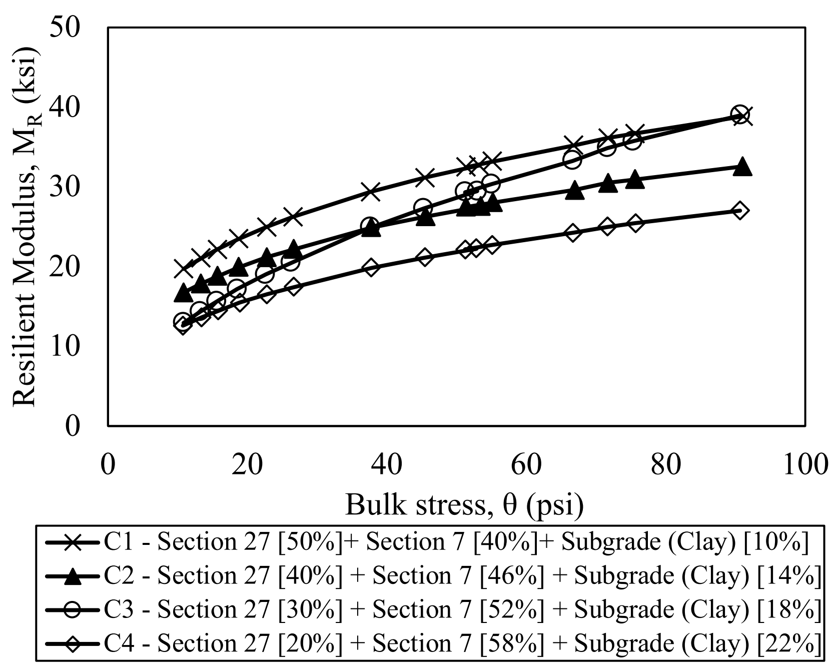


Figure 4.31 M_R vs θ, model fit results of Cherry County blends – C1, C2, C3, and C4.

The model parameters offer a significant understanding of material behavior. The k_1 coefficient followed a similar trend, with C1 (1483.2) having the highest value and H8 (336.8) the lowest. Higher k_1 values suggest better initial resistance to load-induced deformations. Blends with k_1 values exceeding 1000, such as D3, D7, and S4, are expected to perform well, while those below 500, like H4 and H7, may be more susceptible to early-stage deformations.

The k_2 fitting parameter varied significantly across the dataset. Blends H9 (0.71) and H7 (0.70) showed the highest k_2 values, indicating material stiffness is highly dependent on stress changes. Such materials might perform well under moderate loads but could be unpredictable under heavy traffic. The k_3 parameter, or bulk stress exponent, is negative across all blends, meaning that stiffness decreases as bulk stress increases. The most negative k_3 value is observed in S5 (-0.07), which suggests a rapid reduction in modulus under increased loading. This characteristic may lead to early deterioration and higher maintenance needs. Most other blends have k_3 values between -0.01 and -0.04, indicating a more moderate reduction in stiffness under load. Most R^2 values are relatively high, generally exceeding 0.75, indicating good model fits. However, a few cases, such as D3 (0.25) and H3 (0.28), suggest weaker predictive relationships. These lower R^2 values highlight the need for further refinement in material blending or testing procedures to ensure more reliable M_R estimations.

Results showed that the gravel-to-sand ratio also plays a crucial role in yielding higher M_R values, as this ratio is above 1.5. This aligns with the literature, indicating that a well-graded mix with a balanced distribution of coarse and fine aggregates enhances structural resilience. For high-performance applications, blends with a high gravel content (50-70%), moderate sand content (30-40%), and low fines content (<15%) are recommended. Clay subgrades are preferable over silty loam, as they provide better cohesion and stability. If silty loam must be used, the proportion of crusher run or gravel should be increased to compensate for the reduced stiffness. Blends with fines content exceeding 15-20% should be avoided, as they tend to reduce M_R and overall performance.

4.2.3 Permanent Deformation results of blends

In this section, the PD test outcomes are discussed. This test was conducted in accordance with the AASHTO procedures detailed in Section 3.3.2. The plastic strain (%) values for all 31 blends are provided in Table 4.9. Blends failed the testing if they exceeded the total permanent strain of five percent of the total height of the specimen.

The plastic strain value ranged from 0.08% to as high as 3.78%. Plots depicting the relationship between permanent axial strain and the number of loading cycles for all blends, organized by county, are presented. In this section, only the plot for Douglas County blends are shown in Figure 4.32 and the other locations plots are reported in Appendix B. Following the Werkmeister Model, all 31 mixtures were characterized into Ranges A, B, and C, based on their strain rates.

The test results indicate that most blends fall within the B range (plastic shakedown range), suggesting strong resistance to PD. Blends like S5 (0.08%), S4 (0.14%), and H9 (0.24%) exhibit very low plastic strain values, meaning excellent stability and resistance to rutting. These blends are well-suited for use in gravel roads, particularly in areas with moderate traffic loads.

Table 4.9 Permanent deformation results of all blends

S. No	Blend Code	Plastic strain (%)	PD (Range)
1	D1	0.49	B
2	D2	NA	NA
3	D3	1.55	B
4	D4	0.5	B
5	D5	NA	NA
6	D6	NA	NA
7	D7	0.48	B
8	D8	0.6	C
9	D9	1.2	C
10	D10	0.98	B
11	H1	0.38	B
12	H2	0.46	B
13	H3	0.44	B
14	H4	1.39	C
15	H5	0.43	B
16	H6	0.49	B
17	H7	1.59	C
18	H8	NA	NA
19	H9	0.24	B
20	H10	0.43	B
21	H11	3.78	C
22	H12	NA	NA
23	S1	1.08	C
24	S2	0.75	B
25	S3	0.37	B
26	S4	0.14	B
27	S5	0.08	B
28	C1	1.32	C
29	C2	NA	NA
30	C3	NA	NA
31	C4	NA	NA

NA – Not applicable (Specimen Failed in PD test)

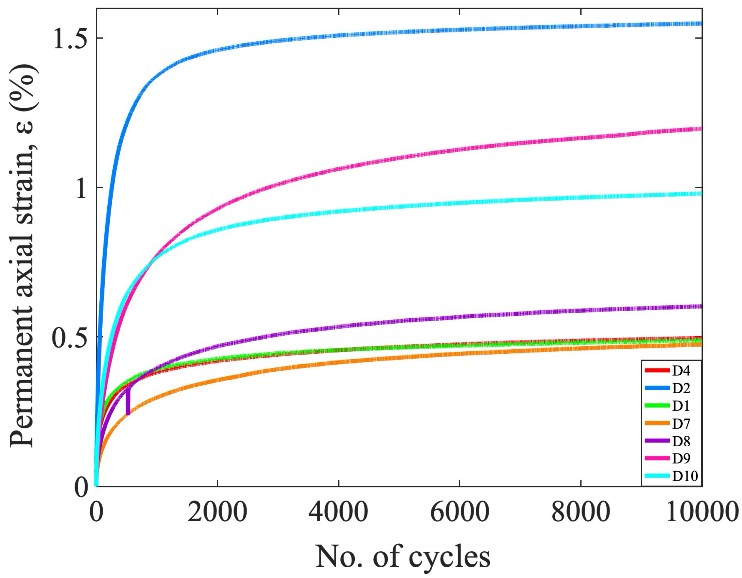


Figure 4.32 Cumulative permanent axial strain versus number of load repetitions for Douglas County blends

However, some blends in the B range, such as D1 (0.49%), D4 (0.5%), and H6 (0.49%), have plastic strain values closer to the upper limit of this category. While they still perform acceptably, they should be monitored closely, as they are nearing the threshold for the C range, which indicates poorer performance. Blends in Range C (incremental collapse) demonstrate higher susceptibility to PD, with plastic strain values exceeding 1.0%. Examples include D8 (0.6%), D9 (1.2%), H4 (1.39%), H7 (1.59%), H11 (3.78%), S1 (1.08%), and C1 (1.32%). Among these, H11 stands out with the highest plastic strain of 3.78%, indicating significant deformation issues.

Several blends are marked as "NA" because the specimens failed during the PD test. These include D2, D5, D6, H8, H12, C2, C3, and C4. Notably, these blends have similar gravel, sand, and fines content. Low gravel fraction and excessive fines content combined with up to

50% sand are likely responsible for the high plastic strains. The failure of these specimens suggests potential issues with their composition, structural integrity, or testing conditions.

The data highlights the importance of material composition, particularly the proportions of gravel, sand, and fines, in determining the plastic deformation behavior of blends. High gravel content, a balanced gravel-to-sand ratio, and low fines content are key to achieving low strain values and high performance. Subgrade type also plays a critical role, with clay subgrades generally outperforming silty loam. Blends like H9, D7, and S4 stand out as top performers, while blends like H11 and C1 demonstrate the detrimental effects of high fines and poor subgrade conditions. By carefully optimizing these factors, it is possible to design high-performing material blends for granular road applications.

4.2.4 Freeze-thaw results of blends

Environmental conditions and traffic loads influence the surface elastic modulus and shear strength, making an understanding of these mechanistic factors essential for assessing road performance. Previous studies have highlighted the substantial impact of FT cycles in deteriorating the mechanical properties of granular layers (White & Vennapusa, 2013).

The influence of FT cycles on the stiffness properties of unpaved road materials is conducted to understand the performance of the materials that undergo environmental changes during the service life of the roads. The experiment was designed to simulate field conditions. To assess the degradation in stiffness due to FT cycles, each material was subjected to five open system FT cycles following the guidelines mentioned in Section 3.3.3. For selected materials M_R, and PD tests were carried out to evaluate the long-term performance. From each county, mixtures that fully completed the M_R and PD tests were analyzed and ranked using the

Technique for Order Preference by Similarity to the Ideal Solution (TOPSIS), a well-established Multi-Criteria Decision-Making (MCDM) method.

TOPSIS is widely used for ranking and selecting alternatives when multiple and often conflicting criteria are involved. This method helps identify the best alternative by comparing each option against an ideal solution (Uzun et al., 2021). Mixtures were selected from the extreme ends of the mixture rankings and one from the middle to see how different materials behave under exposure to FT cycles. However, blends that did not fail in permanent deformation were only considered for the ranking.

Out of seven blends from Douglas County, three blends—D7, D1, and D9—ranked 1st, 4th, and 6th, respectively, were selected based on their performance. From Harlan County, four blends—H10, H5, H1, and H7—ranked 1st, 4th, 5th, and 9th, were chosen due to their stability and gradation properties. In Scottsbluff County, blends S4 and S2 demonstrated superior durability and were selected accordingly. From Cherry County, C1 was the only blend among the four that did not fail the PD test and was therefore selected for FT testing. These selected blends exhibited promising characteristics for optimized gravel road performance and were further analyzed for their mechanical properties.

The presented Table 4.10 summarizes the results of FT testing on selected blends from different counties. The primary focus is on the modulus (M_R) and the percentage of PD after exposure to five FT cycles. Additionally, the coefficients (k_1, k_2, k_3) and the R^2 values provide insights into the material behavior under cyclic loading.

Table 4.10 Resilient Modulus (SM_R) and PD results after 5FT cycles

Specimen code	TOPSIS Rank	5FT M_R					5FT Plastic strain (%)	PD Range
		SM_R (ksi)	k_1	k_2	k_3	R^2		
D7	1	36.8	2301.27	0.13	-0.01	0.19	0.78	C
D1	4	20.90	974.44	0.53	-0.01	0.99	1.01	C
D9	6	22.86	1043.98	0.56	-0.03	0.94	0.4	B
H10	1	22.92	1158.44	0.43	-0.02	0.91	0.61	B
H5	4	18.24	919.33	0.44	-0.04	0.94	1.12	B
H1	5	15.31	707.38	0.55	-0.03	0.98	0.65	B
H7	9	18.44	814.47	0.56	-0.02	0.93	0.35	B
S4	1	NA	NA	NA	NA	NA	NA	NA
S2	4	23.34	1191.08	0.41	-0.02	0.93	3.84	C
C1	1	16.31	795.76	0.481	-0.03	0.92	2.77	C

k_1, k_2, k_3 – fitting parameters; 5FT – five cycles of freeze-thaw; SM_R – Summary M_R; PD range - permanent deformation range;

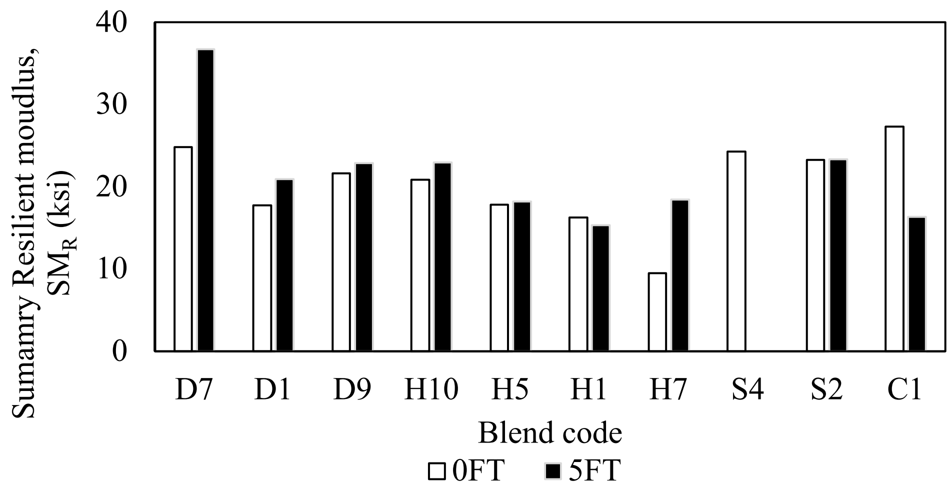


Figure 4.33 Comparison of SMR test values for 0FT and 5FT for all blends

4.2.4.1 Freeze-thaw effects on resilient modulus

A comparison of results before and after exposure to FT cycles highlighted significant changes in material performance. The variations in SM_R values for all blends exposed to FT cycles are illustrated in Figure 4.33. Regression analysis was conducted to fit the MEPDG model to the laboratory-generated M_R data. Results were shown as the material's respective regression constants (k_1, k_2, k_3) and the coefficient of determination. These curves were plotted for selected blends within each county by comparing before and after FT exposure tests. The performance variations among different blends are illustrated in Figure 4.34 (Douglas County), Figure 4.35 (Harlan County), Figure 4.36 (Scottsbluff County), and Figure 4.37 (Cherry County).

In Douglas County, blend D7 initially exhibited an SM_R of 24.8 ksi; however, after FT exposure, this value increased significantly to 36.8 ksi, indicating enhanced stiffness. Blends D1 and D9 showed moderate increases, with SM_R rising from 17.71 ksi to 20.90 ksi and from 21.65 ksi to 22.86 ksi, respectively, suggesting improved FT resilience.

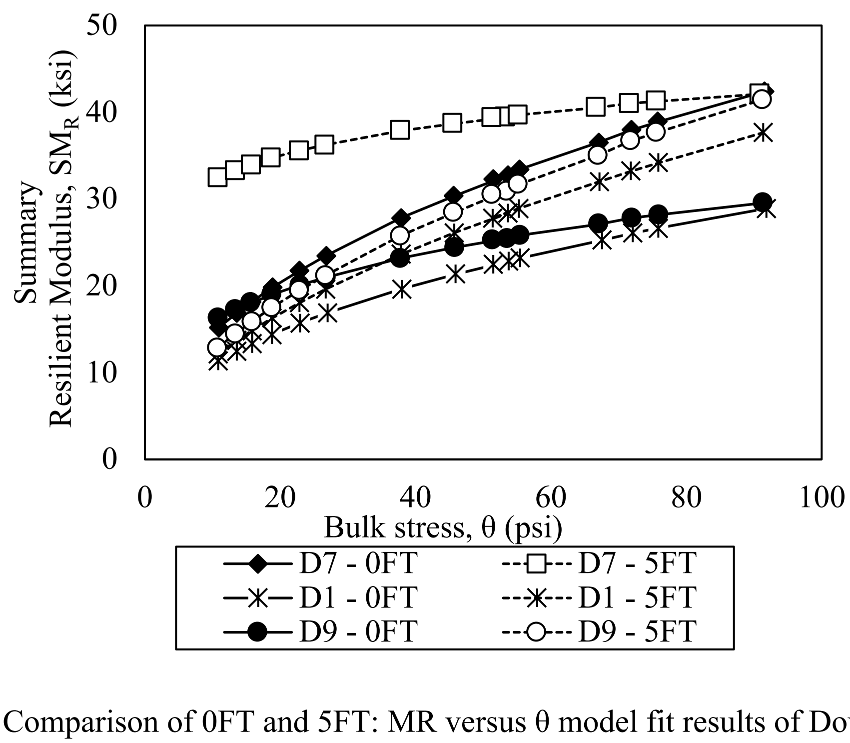


Figure 4.34 Comparison of 0FT and 5FT: MR versus θ model fit results of Douglas County blends D7, D1 and D9

In Harlan County, blend H10 maintained relatively stable stiffness, with an SM_R of 20.85 ksi before FT exposure and 22.92 ksi afterward. Blends H5 and H1 exhibited mixed behavior, with H5 showing a slight increase from 17.79 ksi to 18.24 ksi, whereas H1 experienced a reduction from 17.55 ksi to 15.31 ksi.

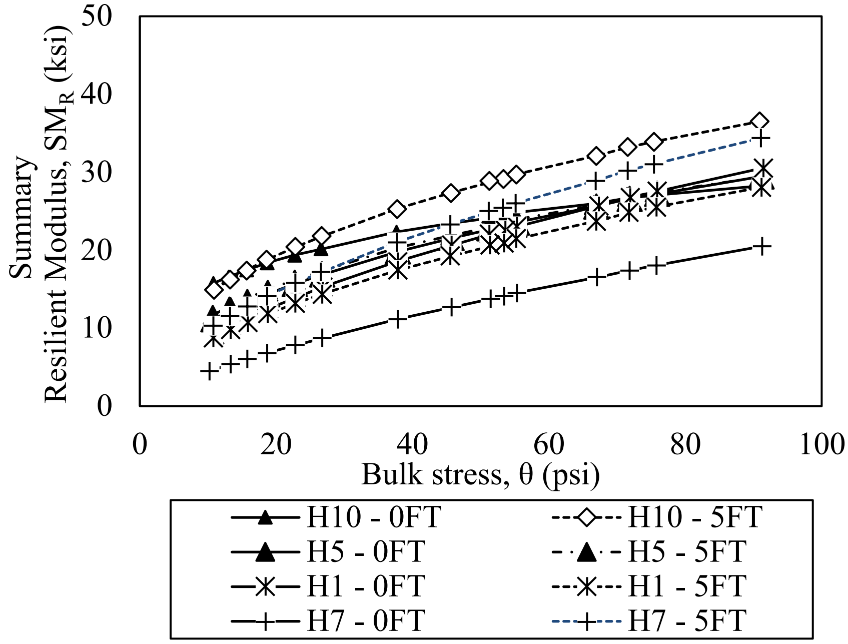


Figure 4.35 Comparison of 0FT and 5FT: MR versus θ model fit results of Harlan County blends H10, H5, H1 and H7

For Scottsbluff County, blend S2 exhibited a minor improvement, with SM_R increasing slightly from 23.23 ksi to 23.34 ksi. However, S4 had no available post-FT data, as the material failed after five FT cycles while being mounted on the loading frame.

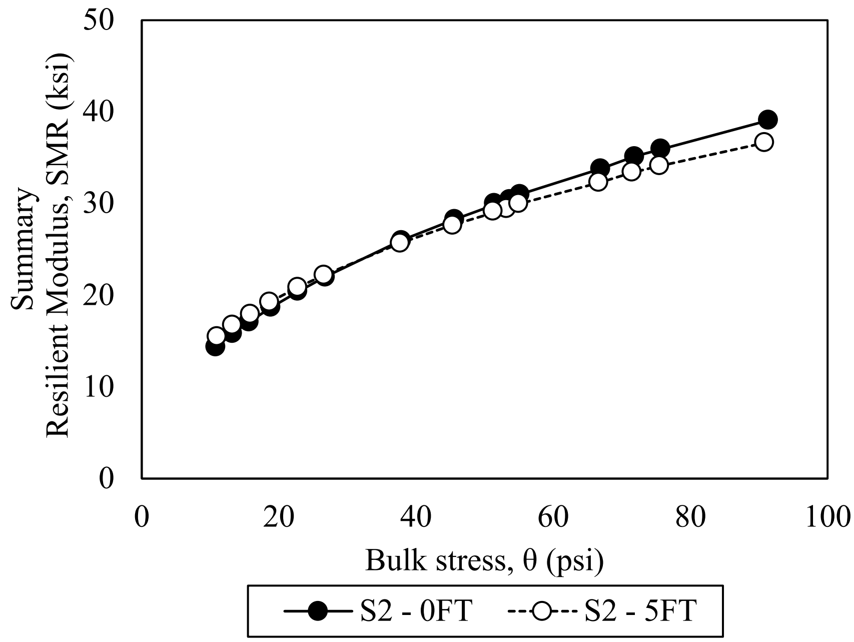


Figure 4.36 Comparison of 0FT and 5FT: MR versus θ model fit results of Scottsbluff County blend S2

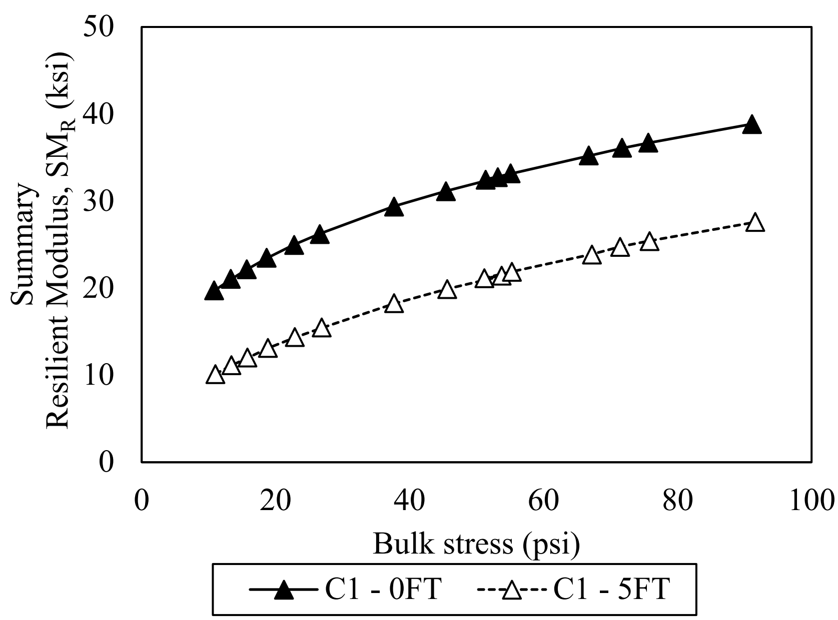


Figure 4.37 Comparison of 0FT and 5FT: MR versus θ model fit results of Cherry County blend C2

Conversely, blend C1 experienced a significant reduction in stiffness, with SM_R decreasing from 27.3 ksi to 16.31 ksi, indicating a major loss in resilience and making it more vulnerable to FT damage.

A comparison of M_R test results for samples exposed to five FT cycles and those with zero FT cycles revealed distinct trends. Blends with high gravel content, low fines content, and balanced sand content (e.g., D7, H10) demonstrated the best performance, exhibiting increased stiffness and reduced deformation. In contrast, blends with high fines content (e.g., C1, D9) tended to perform poorly, particularly under FT conditions. For the selected blends from four counties, exposure to FT cycles did not significantly affect M_R values. Based on the test results, it can be concluded that blended materials show minimal sensitivity to FT cycles, indicating their potential suitability for freezing-thawing prone environments.

4.2.4.2 Freeze-thaw effects on permanent deformation

The PD test was conducted and plastic strain values of selected blends after 10,000 cycles are presented in Table 4.10. The accumulated axial strain values of both the 0FT and 5FT samples are plotted versus number of cycles in Figure 4.38, Figure 4.39 and Figure 4.40 for Douglas County blends D7, D1, and D9, respectively. Figure 4.41

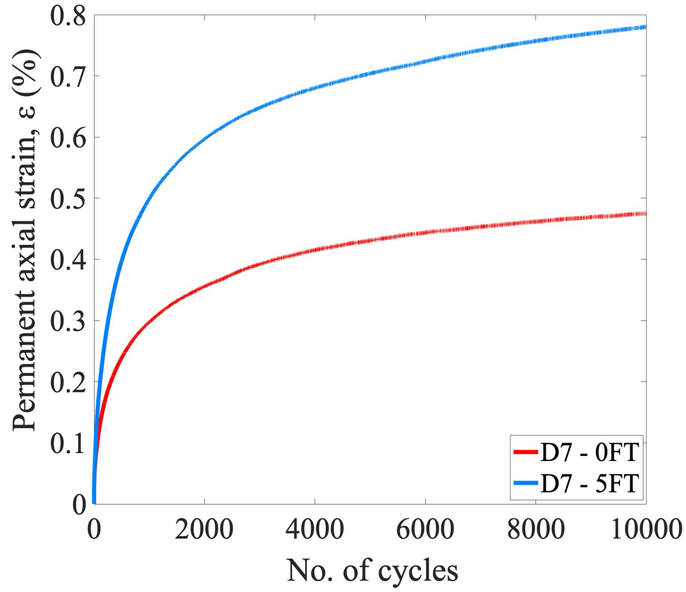


Figure 4.38 Comparison of 0FT & 5FT: Permanent axial strain versus number of cycles for blend D7

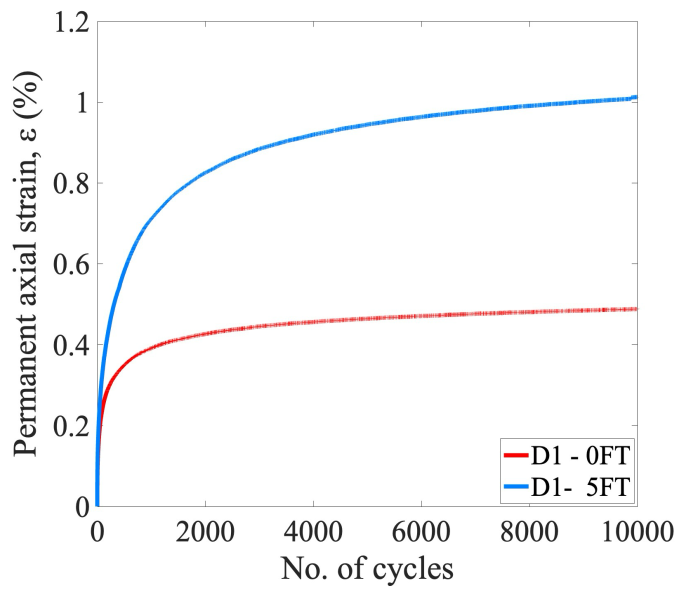


Figure 4.39 Comparison of 0FT & 5FT: Permanent axial strain versus the number of cycles for blend D1

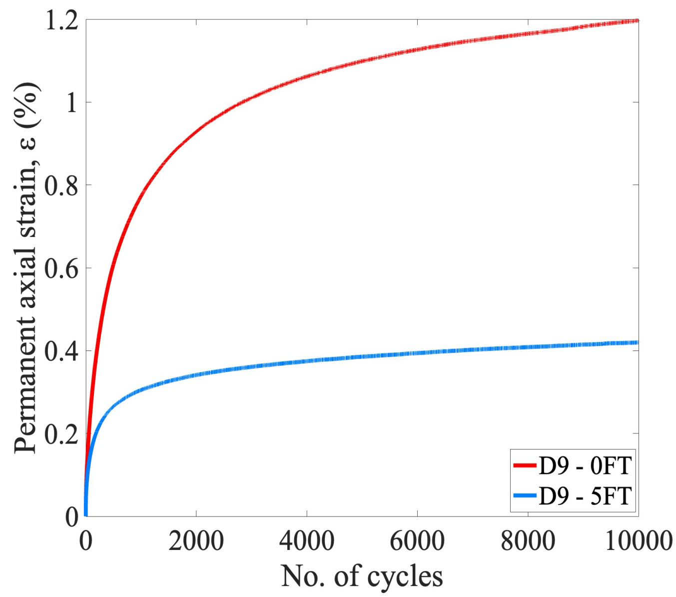


Figure 4.40 Comparison of 0FT & 5FT: Permanent axial strain versus the number of cycles for blend D9

Blend D7 had an accumulated plastic strain of 0.48%, which increased slightly to 0.78% after FT cycle exposure, demonstrating good resistance to deformation, whereas D1 had an 0FT plastic strain of 0.49%, which increased to 1.01% after the exposure, indicating a moderate increase in susceptibility. The impact of FT cycle exposure on these two blends was evident, as they led to increased PD. According to Werkmeister's classification, these materials exhibited a transition from Range B to Range C, indicating a shift in behavior from plastic shakedown to incremental collapse.

Conversely, blend D9 exhibited a unique response after exposure to FT cycles, shifting its behavior from Range C to Range B, indicating improved performance or increased resistance to PD after exposure. This improvement may be attributed to the presence of a high amount of plastic fines (16%), where plasticity had a positive impact, enhancing resistance to PD.

In Harlan County, four blends (H10, H5, H1 and H7) were selected and tested for PD. The accumulated plastic strain plotted against the number of cycles is presented in Figure 4.41, Figure 4.42, Figure 4.43, and Figure 4.44.

Harlan County blend H10 demonstrated strong deformation resistance Its initial (0FT) plastic strain measured 0.43% and increased slightly to 0.61% after exposure to 5FT. This minimal increase suggests that H10 maintained stability under loading conditions, exhibiting favorable resistance to plastic deformation.

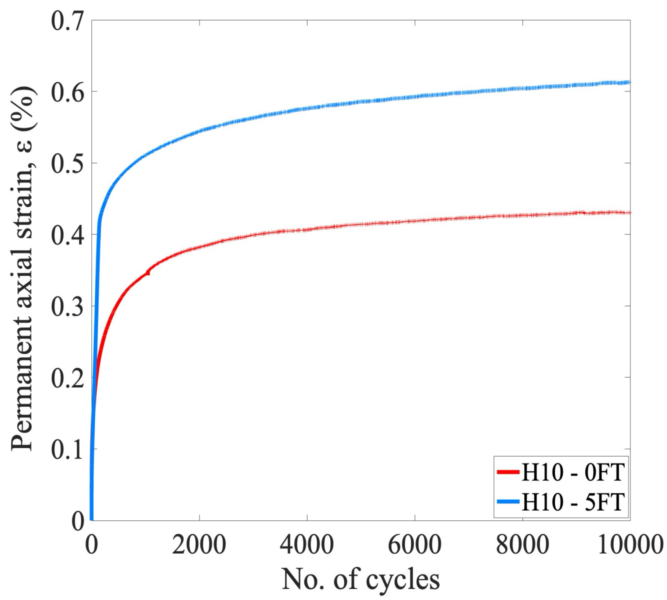


Figure 4.41 Comparison of 0FT & 5FT: Permanent axial strain versus number of cycles for blend H10

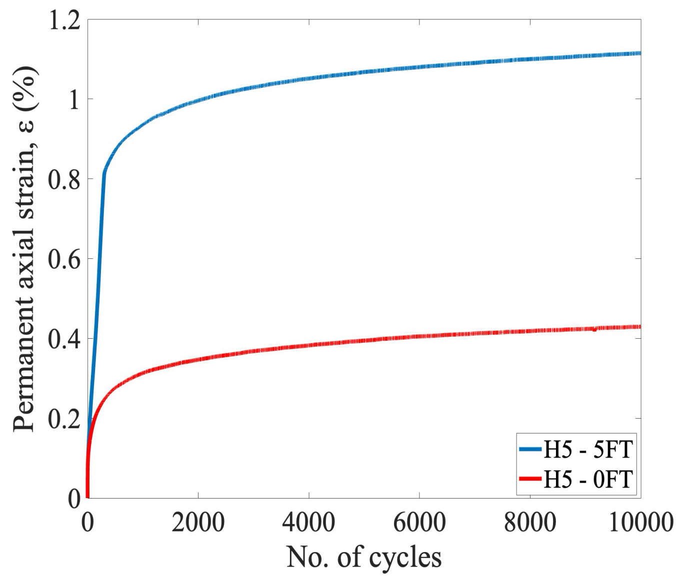


Figure 4.42 Comparison of 0FT & 5FT: Permanent axial strain versus number of cycles for blend H5

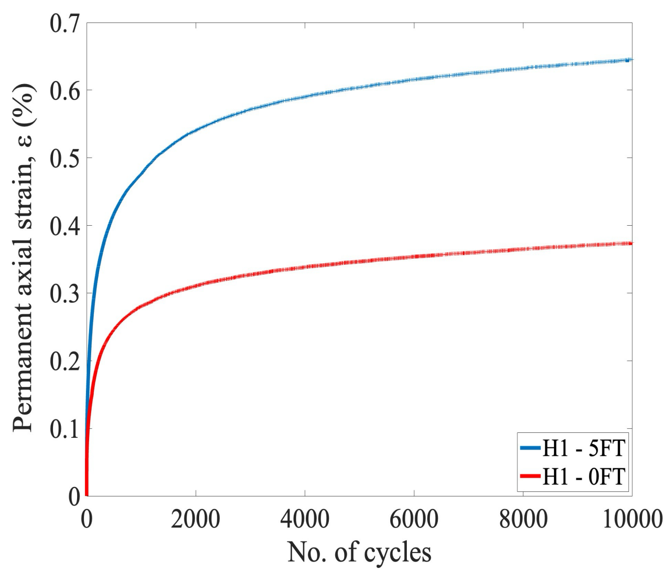


Figure 4.43 Comparison of 0FT & 5FT: Permanent axial strain versus number of cycles for blend H1

Similarly, H5 and H1 with similar gravel, sand, and fines content experienced moderate increases in PD. H5's plastic strain rose from 0.43% to 1.12%, while H1's plastic strain increased from 0.38% to 0.65%. Although these increases were slightly higher than those observed in H10, they still indicated reasonable stability and deformation resistance. Notably, H10 had a balanced gradation, which may have contributed to its better performance.

Furthermore, all these blends remained within Range B of the Werkmeister criteria. Materials in Range B are considered to have high resistance to PD, suggesting that these blends can sustain repeated traffic loading without significant structural deterioration. This classification reinforces the suitability of these blends for use in gravel road applications where long-term durability and minimal maintenance are essential.

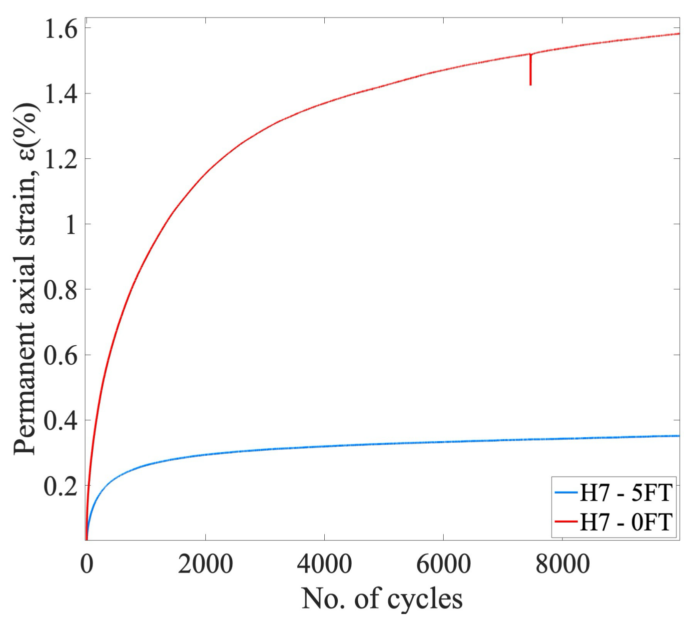


Figure 4.44 Comparison of 0FT & 5FT: Permanent axial strain versus number of cycles for blend H7

Out of all samples, blend H7 showed a significant increase in PD performance. The sample exposed to 5FT cycles showed better performance than the sample without exposure. This blend contained a high amount of fines; due to the higher binding capacity, the specimen underwent densification or improved interlocking.

From Scottsbluff County, two blends S4 and S2 were selected according to their ranking. Blend S4 failed before testing despite multiple attempts. This failure was attributed to its high sand content (44%) and fines content (16%). Notably, the fines used in this blend had a low plasticity index (PI), resulting in inadequate binding capacity. Additionally, the load-bearing capacity was primarily governed by a finer portion which likely contributed to the failure.

The S2 blend before FT exposure had a significantly high plastic strain of 0.75%, which escalated to 3.84% after the 5FT exposure, suggesting substantial FT vulnerability. Figure 4.45 presents the PD test results for one of the selected blends (S2) from Scottsbluff County. According to Werkmeister's criteria, this blend transitioned from Range B to Range C after the exposure, signifying a decline in performance. The observed behavior suggests that fines content (6%) plays a crucial role in determining material performance.

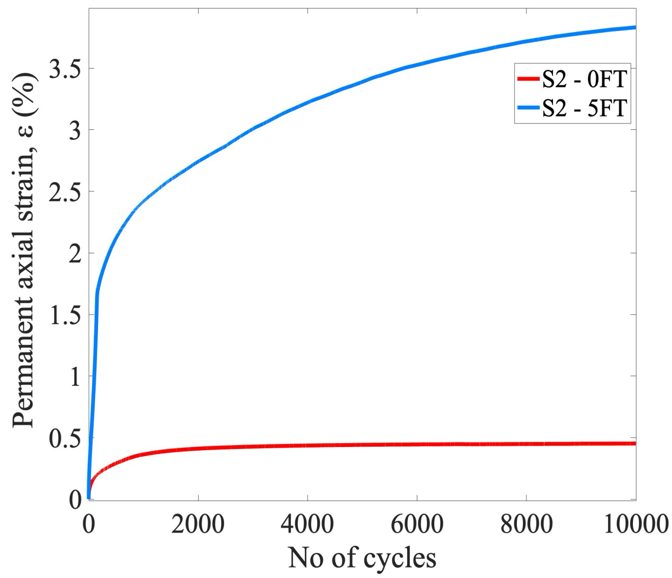


Figure 4.45 Comparison of 0FT & 5FT: Permanent axial strain versus the number of cycles for blend S2

Both excessive and insufficient fines resulted in poor performance, emphasizing the need for a well-balanced proportion of gravel, sand, and fines to optimize the mechanical properties of the material.

In Cherry County, only one blend successfully completed the PD test without FT exposure. As a result, this blend was selected for further testing under FT exposure to assess its performance.

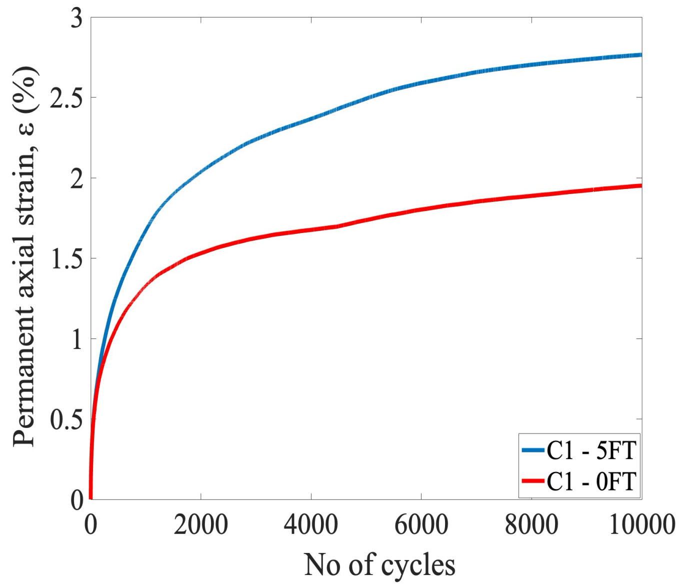


Figure 4.46 Comparison of 0FT & 5FT: Permanent axial strain versus the number of cycles for blend C1

The permanent accumulated strain of 0FT and 5FT are plotted versus the number of cycles for blend C1 in Figure 4.46. After testing, blend C1 exhibited an accumulated plastic strain of 1.32%, which increased to 2.77% after FT exposure. This indicates a notable increase in deformation, confirming its susceptibility to FT effects. It fell in Range C of the Werkmeister criteria, indicating a tendency toward incremental collapse and confirming its poor resistance to FT cycles. This behavior was attributed to the gradation effects, as blend C1 contained a moderate fines content (9%) and high gravel content. The lack of sufficient binding capacity and interparticle contact further contributed to its poor performance under FT conditions.

Table 4.11 Summary of M_R and PD test results before and after exposure to 5FT cycles

Blend Code	**Gravel (%)**	**Sand (%)**	**Fines (%)**	**SM_R (ksi)**		**Plastic strain (%)**		**Range**	
				0FT	**5FT**	**0FT**	**5FT**	**0FT**	**5FT**
D7	65	28	7	24.8	36.8	0.48	0.78	B	C
D1	47	45	8	17.7	20.9	0.49	1.01	B	C
D9	46	37	16	21.6	22.8	1.2	0.4	C	B
H10	55	36	8	20.8	22.9	0.43	0.61	B	B
H5	66	26	7	17.8	18.2	0.43	1.12	B	B
H1	69	23	7	16.3	15.3	0.38	0.65	B	B
H7	49	37	13	9.5	18.4	1.59	0.35	C	B
S4	40	44	16	24.3	NA	0.14	NA	B	NA
S2	56	37	6	23.2	23.3	0.75	3.84	B	C
C1	59	30	9	27.3	16.3	1.32	2.77	C	C

In conclusion, FT cycles did not have a significant impact on material stiffness (M_R), but blended materials showed increased plastic strain that leads to a lower resistance to PD.

Blends with higher and lower fines content were more vulnerable to FT effects, while well-graded blends with balanced gravel and sand content tended to perform better. Some blends exhibited severe deterioration (C1, S2), whereas others (H10, H5, H1) remained relatively stable even after FT cycles. The obtained results indicate that materials containing a high amount of fines (>16%) and a low amount of fines (< 6%) are highly sensitive to freeze-thaw cycles regarding permanent deformation.

Although the results of this experimental study provide new insight related to the M_R and permanent deformation characteristics of blended materials exposed to FT cycles, it is important to note that these conclusions can be extrapolated only to mixtures with similar characteristics to those investigated. For broader conclusions, further research should focus on investigating the effects of FT conditioning on the M_R and permanent deformation on a wider range of materials, considering different gradations and material origins.

Chapter 5 Statistical Analysis and Optimization of Aggregate Gradation

This chapter outlines the statistical analysis framework used to optimize aggregate gradation for improved granular road performance. A forward stepwise regression model was developed to predict (1) resilient modulus (M_R) and (2) permanent deformation (PD) of gravel blends based on key gradation and index properties.

Multivariate regression was used to evaluate material behavior in granular roads due to its clarity and ease of interpretation. The analysis was conducted using engineered blends, which were systematically developed by adjusting the proportions of coarse aggregates, fine aggregates, and binder materials collected from various regions across Nebraska to create well-graded compositions. To enhance statistical robustness, previous research data were integrated, increasing variability and improving model generalizability across different granular road conditions.

To ensure model reliability, the dataset was split into training (80%) and testing (20%) sets. The training data developed regression equations and identified significant variables, while the testing data validated performance on new observations. This validation minimized overfitting, ensuring the models captured meaningful patterns rather than noise. Independent variables were evaluated for statistical significance using p-values, with a threshold of 0.05. Variables meeting this criterion were retained, while others were excluded unless a strong theoretical rationale justified their inclusion. Residual normality was confirmed using histograms and P-P plots.

Data processing, statistical analysis, and model development were carried out using Microsoft Excel, which facilitated descriptive statistics, visualization, and regression implementation. This structured methodology ensured a systematic evaluation of how gradation and index properties influence MR and PD. Refining the dataset and validating assumptions ensured a robust framework for optimizing granular road performance.

The MR model included key independent variables such as oven-dry specific gravity (OD GS), particle size distribution parameters (D10, D30, D50), and silt content, as summarized in Table 5.1. D30 values ranged from 0.19 mm to 7.42 mm, indicating variation in mid-sized aggregates, while silt content varied from 3.23% to 20.61%. For the PD model, independent variables including clay content, coefficient of curvature (Cc), medium sand content, and fines content were used as presented in Table 5.2. C_c ranged from 0.14 to 45.51, reflecting variations in gradation uniformity, while fines content ranged from 3.98% to 25.29%. The variability in these parameters ensures that the dataset captures a diverse range of material conditions, providing a comprehensive basis for regression modeling.

Table 5.1 Descriptive Statistics for M_R Model Variables

Parameter	Mean	SD	Min.	Max.
OD G_s	2.45	0.11	2.25	2.60
D_{30}	2.30	1.82	0.19	7.42
D_{10}	0.13	0.15	0.01	0.68
D_{50}	5.77	3.00	1.53	12.10
Silt (%)	9.83	4.78	3.23	20.61

OD G_s = oven-dry specific gravity, D_{10} = the diameter at which 10% of the soil particles are finer (mm), D_{30} = the diameter at which 30% of the soil particles are finer (mm), D_{50} = the diameter at which 50% of the soil particles are finer (mm), Silt (%) = Silt content, SD = Standard deviation

Table 5.2 Descriptive Statistics for PD Model Variables

Parameter	Mean	SD	Min.	Max.
Clay (%)	3.25	1.70	0.62	6.00
C_c	11.65	14.38	0.14	45.51
M. Sand (%)	11.34	4.59	2.17	23.64
Fine (%)	13.33	6.51	3.98	25.29

Clay (%) = Clay content, C_c = coefficient of curvature, M. Sand (%) = medium sand content Fine (%) = fine content

5.1 Statistical Analysis

A multiple linear regression model was developed to examine the influence of key index properties on the M_R of gravel blends. As shown in Table 5.3, the model demonstrated a strong fit, explaining approximately 72% of the variance in M_R ($R^2 = 0.72$). The overall model was highly significant ($F(5, 37) = 19.11$, $p < 0.001$), confirming that the predictors explain variations in M_R, as detailed in Table 5.4.

Table 5.3 Regression Summary for M_R Model

Statistics	**Values**
Multiple R	0.85
R Square	0.72
Adjusted R Square	0.68
Standard Error	3.16
Observations	600

Table 5.4 ANOVA Results for M_R Model

Source	**Df**	**SS**	**MS**	**F**	**Significance F**
Regression	5	952.63	190.53	19.11	2.3E-09
Residual	37	368.88	9.97		
Total	42	1321.51			

Table 5.5 shows the regression coefficients, quantifying each predictor's effect on M_R. OD G_s exhibited the strongest positive association ($\beta = 43.90$, $p < 0.001$), indicating that higher density improved M_R. In contrast, D_{30} had a negative effect ($\beta = -1.53$, $p < 0.003$), suggesting that excess mid-sized particles reduced stiffness. Furthermore, both D_{10} ($\beta = 11.57$, $p = 0.00881$) and D_{50} ($\beta = 2.94$, $p < 0.001$) were positively associated with M_R, reflecting that increases in

both the fine (D_{10}) and coarser (D_{50}) fractions contribute to enhanced stiffness. Silt content also had a significant positive influence ($\beta = 1.37$, $p < 0.001$).

A separate regression model quantified the effect of index properties on PD of the gravel blends. As presented in Table 5.6, the PD model shows an R2 of 0.70, indicating that 70% of the variation in PD is explained by the selected predictors. The model is statistically significant ($F(4, 28) = 16.67$, $p < 0.001$) as shown in Table 5.7.

Table 5.5 Regression Coefficients for M_R Model

Variable	**Coefficient**	**Standard Error**	**t-Statistic**	**P-value**	**95% Confidence Interval**
Intercept	-117.59	17.49	-6.72	6.65E-08	[-153.03, -82.153]
OD G_s	43.90	6.14	7.16	1.76E-08	[31.47, 56.34]
D_{30}	-1.53	0.47	-3.24	2.53E-03	[-2.48, -0.57]
D_{10}	11.57	4.18	2.77	8.81E-03	[3.09, 20.05]
D_{50}	2.94	0.35	8.50	3.15E-10	[2.24, 3.65]
Silt (%)	1.37	0.21	6.52	1.26E-07	[0.95, 1.8]

Table 5.6 Regression Summary for PD Model

Statistics	**Values**
Multiple R	0.84
R Square	0.70
Adjusted R Square	0.66
Standard Error	0.24
Observations	33

Table 5.7 ANOVA Results for PD Model

Source	**Df**	**SS**	**MS**	**F**	**Significance F**
Regression	4	3.87	0.97	16.67	4.2E-07
Residual	28	1.62	0.06		
Total	32	5.49			

The regression coefficients in Table 5.8 provide insight into how each variable influences PD. Clay content increased PD (β = 0.31, p = 2.62E-04), indicating that higher plasticity leads to greater deformation. Similarly, the coefficient of curvature (C_c) is positive (β = 0.01, p = 1.29E-03), implying that a broader spread in particle sizes (or less uniform gradation) contributes to higher deformation. Medium sand content also exhibits a positive effect (β = 0.06, p = 1.98E-05), indicating that increased proportions of medium sand may reduce packing efficiency and elevate PD. In contrast, fines content shows a negative relationship (β = – 0.08, p = 1.81E-04), meaning that higher fines levels likely improve particle interlock and load distribution, thereby reducing PD.

Table 5.8 Regression Coefficients for PD Model

Variable	**Coefficient**	**Standard Error**	**t-Statistic**	**P-value**	**95% Confidence Interval**
Intercept	-0.18	0.17	-1.06	2.96E-01	[-0.53, -0.17]
Clay (%)	0.31	0.07	4.18	2.62E-04	[0.16, 0.46]
Cc	0.01	0.00	3.58	1.29E-03	[0.01, 0.02]
M. Sand (%)	0.06	0.01	5.12	1.98E-05	[0.03, 0.08]
Fine (%)	-0.08	0.02	-4.31	1.81E-04	[-0.12, -0.04]

To quantify the influence of gradation and index properties on MR and PD, the The regression equation is expressed as Equation (5.1):

$$Y_i = \beta_0 + \beta_1 X_1 + \ \beta_2 X_2 + \cdots + \beta_i X_i + \varepsilon_i \quad 5.1$$

where Y represents the dependent variable—either MR (ksi) or PD (%), and X_1, X_2, X, …, X_i are the independent variables (e.g., OD Gs, Fine (%), D_{10}). β_0 is the intercept, β_i are the estimated coefficients, and the error term ε accounts for unexplained variability in the model. Each β_i

quantify the change in M_R or PD for a one-unit change in the corresponding independent variable, holding other factors constant.

Table 5.9 presents the regression equations for M_R and PD, summarizing their respective R2 and standard error values. These equations provide a quantitative framework for estimating the mechanical performance of gravel blends based on key material properties.

Table 5.9 Regression equations for M_R and PD Models

Model	Equation	R^2	Std. Error
M_R (ksi)	$-117.59 + 43.90\text{*OD } G_s - 1.53\text{*}D_{30} + 11.57\text{*}D_{10} + 2.94\text{*}D_{50} + 1.37\text{*Silt (\%)}$	0.72	3.16
PD (%)	$-0.18 + 0.31\text{*Clay (\%)} + 0.015\text{*}C_c + 0.058\text{*M. Sand (\%)} - 0.08\text{*Fine (\%)}$	0.70	0.24

Figures 5.1 and 5.2 present the predicted versus actual values of M_R for the training and test sets, respectively, while Figures 5.3 and 5.4 illustrate the corresponding results for PD. The training set exhibits R^2 values of 0.72 for M_R and 0.70 for PD, while the corresponding test sets yield slightly lower R^2 values of 0.67 (M_R) and 0.62 (PD). However, a reduction in R^2 is expected when evaluating model performance on new data, the close alignment between predicted and actual values in both sets underscore the robustness of the regression approach. These findings confirm that the identified independent variables and derived equations not only capture a substantial portion of the variability within the training dataset but also maintain predictive accuracy when applied to previously unseen samples.

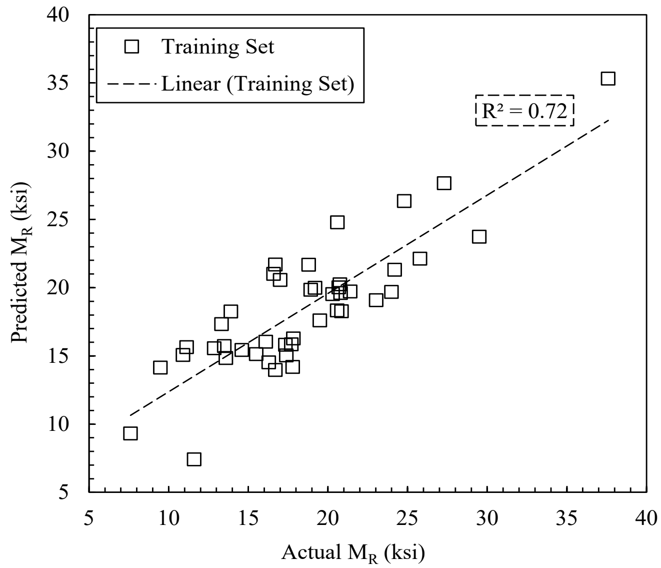


Figure 5.1 Comparison of predicted and actual M_R values for the training set

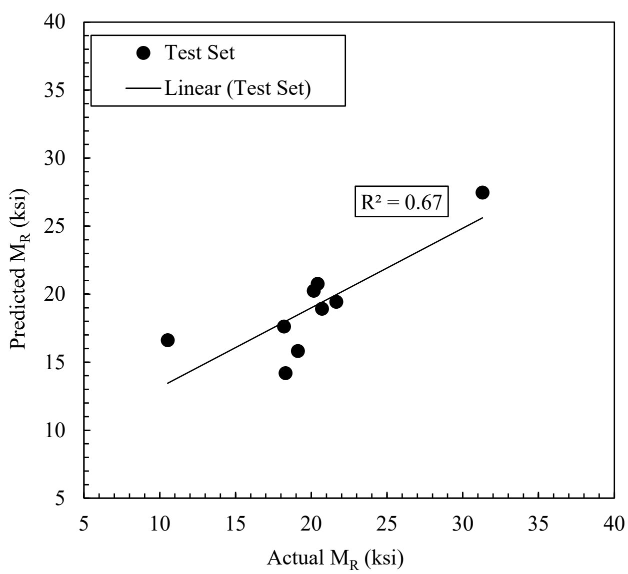


Figure 5.2 Comparison of predicted and actual M_R values for the test set

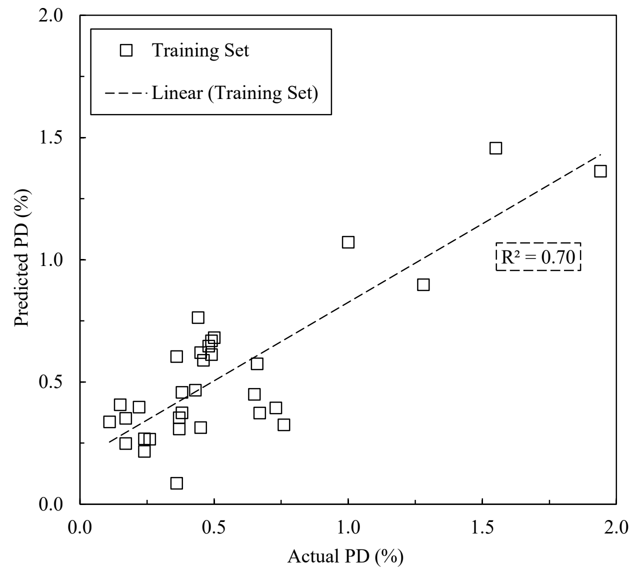


Figure 5.3 Comparison of predicted and actual PD values for the training set

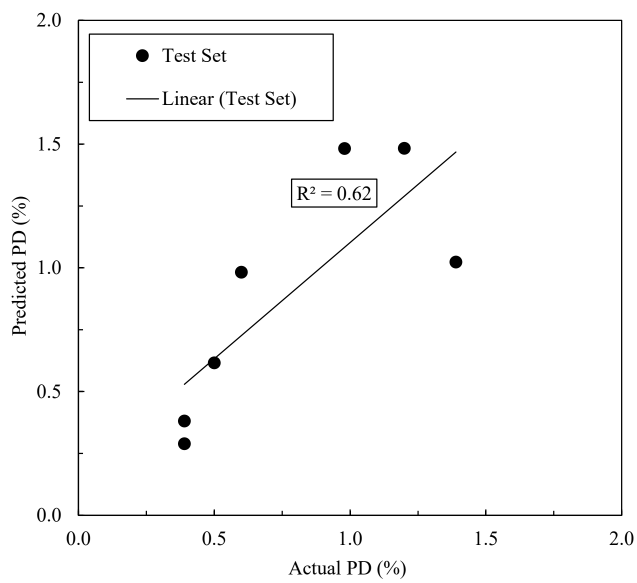


Figure 5.4 Comparison of predicted and actual PD values for the test set

5.2 Gradation Optimization Tool

An Excel-based tool with a user-friendly interface was developed to apply research findings for optimizing gravel road construction by improving material selection and gradation. The tool assists engineers in determining the optimal blend of existing and fresh aggregates to achieve an optimized gradation that enhances road performance.

Users provide key data, including the grain size distribution of the existing gravel surface, base aggregates, and subgrade, along with material properties such as specific gravity and particle size parameters. Additionally, they input road dimensions, including length, width, and thicknesses of the existing gravel and base layers.

Using these inputs, the tool calculates the required proportions of existing materials and fresh aggregates to achieve an optimized gradation range. This range, developed through statistical analysis and an extensive literature review, is defined by upper and lower limits that maximize resilient modulus (M_R) while minimizing PD. An integrated solver analyzes the gradation characteristics of both existing and fresh aggregates to determine the required volume of new material. If the existing gravel contains excessive fines, the tool calculates the necessary amount of coarser fresh aggregate. Conversely, if a finer blend is needed, it determines the required addition of finer material or subgrade. When subgrade addition is necessary, the tool computes the required thickness; otherwise, the subgrade contribution is set to zero.

The tool generates key outputs, including the volume of existing materials, the required quantities of new aggregates, and the final mixture thickness. It also produces visual gradation curves that compare the final blend with the optimized gradation range and illustrates the gradations of both existing and fresh aggregates. Additionally, users can specify a desired thickness of fresh aggregates based on practical constraints such as budget or material availability. The tool then evaluates how closely the selected blend aligns with the optimized gradation. By integrating material optimization with user-defined constraints, the tool provides county engineers and DOT personnel with a reliable, data-driven approach for gravel road design and optimization.

Chapter 6 Summary and Conclusions

This study examined granular surface road materials in Nebraska to optimize gradation for enhanced durability and cost efficiency. Tests included index property analysis, compaction, resilient modulus evaluation, permanent deformation, and freeze-thaw assessments on both virgin and blended materials from Douglas, Harlan, Scottsbluff, and Cherry counties. Survey responses identified raveling, poor drainage, and aggregate loss as key issues, contributing to frequent maintenance needs. Laboratory test results indicated that well-graded materials with a balanced gravel-to-sand ratio provided higher stiffness and better load-bearing capacity, whereas materials with high fines content exhibited reduced stability and increased susceptibility to rutting. Additionally, FT cycle exposure had a significant impact on material performance, further influencing long-term durability. Key conclusions include:

- The survey results showed that the most common distresses affecting gravel roads include raveling (21%), loss of crown (13%), improper drainage (12%), dust (12%), and rutting (11%). The severity of these distresses varies, but raveling, potholes, and frost damage were identified as the costliest to maintain due to their significant material and labor demands.
- For typical maintenance, nearly 35% of respondents reported reshaping existing materials with a motor grader, while 29% maintained proper drainage systems, and 28% made an addition of virgin or recycled materials with blading. The least used method was the application of stabilizers, suggesting that cost-effective and traditional techniques remain the preferred approach. Maintenance was mostly initiated when aggregate loss was observed, followed by excessive fines from surface degradation.

- Regarding design and stabilization, 70% of counties follow NDOT specifications, with AASHTO and FHWA guidelines serving as secondary references. Stabilization is rarely implemented, as only a few counties use proprietary or non-proprietary stabilizers. Challenges in meeting gradation specifications stem from unavailability of high-quality materials, inconsistent contractor performance, and moisture-related issues. While 69% of respondents can adjust NDOT specifications to accommodate locally sourced materials, 31% face limitations due to regulatory constraints or technical capacity.
- The research team, in collaboration with county engineers, identified diverse quarry locations and some sites for subgrade collection. Seventeen materials were carefully selected from four different counties across Nebraska, representing various regions of the state to ensure a comprehensive evaluation of material properties. The materials included 1.5-inch, 1-inch, 3/4-inch, and 3/8-inch subbase materials, crusher run, and river gravels. Additionally, four subgrades including clay and silt were collected to incorporate locally available fines in the gradation improvement process.
- Comprehensive laboratory testing methodologies were employed to accurately determine the index properties of natural soils and aggregates, thereby enhancing the understanding of their physical characteristics. Additionally, to assess the quality of materials, gyratory compaction was utilized, as validated by previous studies. This method helped in evaluating the abrasion resistance of virgin materials.
- Virgin material index properties, including sieve analysis, Atterberg limits, specific gravity, absorption, and moisture content tests, were conducted to determine the physical soil properties. According to the USCS classification, the surface materials were classified as GP, SP, GC, GM, SM, and a dual classification of SW-SM. The road gravel

material from Scottsbluff County was the only surface material classified as GW (well-graded gravel). All collected subgrade materials were classified as ML.

- Gyratory compaction tests conducted in accordance with AASHTO T-312 provided additional insights into the mechanical degradation of aggregates. The results showed that Clean 1.5" and Section 27 materials exhibited the highest total breakage values, indicating greater susceptibility to fragmentation under stress. In contrast, well-graded materials such as crusher run and road gravel demonstrated greater durability and resistance to particle degradation.
- Proctor compaction tests revealed that crusher-run materials exhibited the highest maximum dry unit weight (MDU). The MDU values of all collected surface materials ranged from 111 pcf to 138 pcf, with corresponding OMC values ranging from 4.4% to 11.3%. For two clean materials, vibratory hammer compaction was used to determine the MDU. The MDU value for Clean 1.5-inch was 107 pcf, and for Clean 1-inch, it was 109 pcf. The optimum moisture content (OMC) was higher for subgrades, as they required higher moisture levels to achieve maximum density due to their high specific surface area. As expected, the MDU values for subgrades were the lowest, ranging from 93 pcf to 107 pcf.
- Open-graded materials from Douglas County (DV1, DV2) exhibited higher SMR values, while subgrade materials had the lowest due to fine particle bearing stress. Crusher run 1.5-inch and 3/4-inch materials showed minimal variation, ranging from 15 ksi to 17 ksi. DV3 (Road Gravel), with 70% sand, had the lowest SMR value at 12 ksi. Harlan virgin materials, particularly Crusher Run 1-inch with the highest gravel content, exhibited higher M_R values. Green Surface Sand had an SMR value of 10 ksi, while Surface Sand

(HV3) failed due to excessive fine particles. In Scottsbluff and Cherry counties, SV2 and CV1, which have high gravel contents, exhibited higher M_R values of 17 ksi and 18 ksi, respectively. Meanwhile, SV1 and CV2 materials, which contain more sand, had SMR values of 13 ksi and 14 ksi, respectively.

- The PD test was initiated after applying 500 conditioning cycles, followed by 10,000 loading cycles under 4 psi confining pressure, which simulated field conditions. All the virgin surface materials fell within Range C of the Werkmeister criteria, except for the Harlan crusher run 1.5-inch which was classified in Range B. The results indicated virgin materials require gradation improvements to enhance their resilient behavior and reduce PD, ensuring better overall performance.
- To enhance material behavior and performance, a stabilization technique was employed that blended locally available materials to achieve optimal gradations. Systematic combinations of coarse, fine, and binder materials were used to develop various gradation curves. Ten blends were created in Douglas County, twelve in Harlan County, and four each in Scottsbluff and Cherry counties. These blends exhibited significant differences, allowing for a clear evaluation of mechanical performance variations.
- The MDU and OMC of the blended materials were obtained using the standard Proctor test. MDU values ranged from 125 pcf to 144 pcf, with most blends falling within this range, indicating improved density. OMC values varied between 5.4% and 8.6%. These results highlight the influence of material gradation on compaction characteristics.
- From the M_R test of blended materials, the SM_R values ranged from 7.6 to 27 ksi with most materials falling near the upper limit of this range. Consistent material behavior was

observed from these tests. Particle size distribution and internal structure affected stress-strain behavior.

- The MEPDG model relating M_R to θ was adapted to determine model constants k_1, k_2, k_3 and R^2. Observations revealed that k_1 values ranged from 336 to 1483, k_2 values ranged from 0.32 to 0.7, and k_3 values varied from -0.01 to -0.07. Additionally, the R^2 values ranged from 0.6 to 0.98.
- Permanent deformation tests were conducted to evaluate the long-term deformation properties of blended granular materials and to improve the understanding of their service performance. The results indicated that most blends fell within Range B of the Werkmeister criteria, which is considered desirable for granular surface roads. Compared to virgin materials, the PD test results showed significant improvement, with the blends demonstrating enhanced resistance to plastic deformation.
- A five-cycle FT was performed on selected blends chosen using the TOPSIS method. Out of the 10 blends evaluated, only S4 from Scottsbluff County failed during testing. While the remaining blends showed slightly reduced performance, the adverse effects of FT cycles were minimal.
- Laboratory investigations of granular materials revealed that the granular stabilization technique employed in this study significantly improved the mechanical properties of the materials, leading to enhanced performance. This technique effectively increased the M_R, indicating a higher resistance to deformation under repeated loading. Additionally, stabilized materials exhibited reduced plastic strain, suggesting improved durability and resistance to PD. The stabilization method also contributed to better FT resistance, as some treated materials maintained their structural integrity even after multiple FT cycles.

Overall, the results suggested that granular stabilization enhances load-bearing capacity, minimizes maintenance requirements, and extends the service life of gravel road materials.

- Regression analysis confirmed that all selected independent variables had a statistically significant effect on M_R and PD, reinforcing their relevance in predicting granular road performance. Higher specific gravity, along with larger D_{10} and D_{50} values, positively influenced M_R, suggesting that well-graded materials with an appropriate balance of fine and coarse particles contribute to improved stiffness. In contrast, an increase in D_{30} was associated with reduced M_R, suggesting a negative impact on structural integrity. For PD, greater clay and medium sand content resulted in higher deformation, highlighting the influence of plasticity and particle packing inefficiencies. Conversely, a higher fines fraction reduced PD, likely improving interparticle stability and load distribution. These findings showed that optimizing gradation based on key material properties improves road durability by increasing M_R and reducing PD. This provides a practical approach for selecting and designing aggregate blends.
- The gradation optimization tool developed in this study provides a systematic approach for improving aggregate selection and blending in granular road construction. It calculates the required proportions of existing and new aggregates based on grain size distribution, material properties, and road dimensions, incorporating practical constraints like budget and material availability. The tool generates key outputs like material quantities, final mixture thickness, and visual gradation curves, integrating statistical analysis with a user-friendly interface for reliable and data-driven material usage optimization and decision-making in road design and maintenance.

Chapter 7 Recommendations and Implementations

This chapter presents recommendations based on the study's findings to enhance material characterization, optimize gradation, and improve the performance of granular surface roads. It outlines strategies for practical implementation to ensure the effective application of these findings in road construction and maintenance.

Comprehensive laboratory testing should continue to evaluate key material properties, including particle shape, size, and mineralogy, to better understand variations in stiffness. The role of plastic fines, particularly clay, should be further examined as a potential binder, as it has demonstrated greater resistance to FT cycles than silt. Its inclusion in gravel road surfacing may also mitigate common distresses such as washboarding and raveling.

To enhance material selection, additional testing of various geomaterials and blended mixtures should be conducted to identify optimal combinations that improve resistance to plastic strain and FT cycles. Key physical properties, such as grain shape, angularity, surface roughness, and specific surface area, should be analyzed further, as they directly influence interparticle friction, interlocking, and overall material stability. Expanding the scope of research to include hydraulic conductivity and matric suction testing is recommended, as these properties affect drainage and long-term material performance.

While laboratory testing provides controlled conditions for material evaluation, field implementation is essential to validate findings under real-world conditions. Constructing test sections and conducting field tests, such as Light Weight Deflectometer (LWD), Falling Weight Deflectometer (FWD), nuclear density, and Dynamic Cone Penetrometer (DCP) tests, will provide critical insights into in-field performance. Correlating laboratory results with field data will improve the reliability of material characterization, leading to more effective design and maintenance strategies.

Additionally, implementing the gradation tool in test road sections with continuous monitoring will refine its applicability. Real-world performance data should be collected to refine the tool iteratively, ensuring it accurately adapts to field conditions. This process will optimize its effectiveness, ultimately leading to more efficient, durable, and cost-effective gravel road construction and maintenance. Collaboration between academic institutions, research centers, government agencies, and industry stakeholders is essential to developing a comprehensive material database. Expanding the training dataset for the gradation tool with diverse geomaterial properties and loading conditions will improve its predictive accuracy and reliability.

It is also recommended that the application of this tool be validated across a range of construction projects to ensure its adaptability. Future research should focus on refining the tool by incorporating project-specific construction data, allowing for further customization of material selection and gradation optimization. For these recommendations to be effectively implemented, agencies should adopt a data-driven approach to material selection by integrating both laboratory and field data. Establishing standardized material testing protocols across agencies will enhance consistency in material characterization and performance evaluation. Furthermore, leveraging existing technological advancements such as automated data collection and machine learning applications in gradation optimization could further refine predictive models and improve road performance outcomes.

By adopting these strategies, road agencies can translate research findings into practical, field-applicable solutions, ensuring long-term improvements in granular road performance, cost efficiency, and sustainability.

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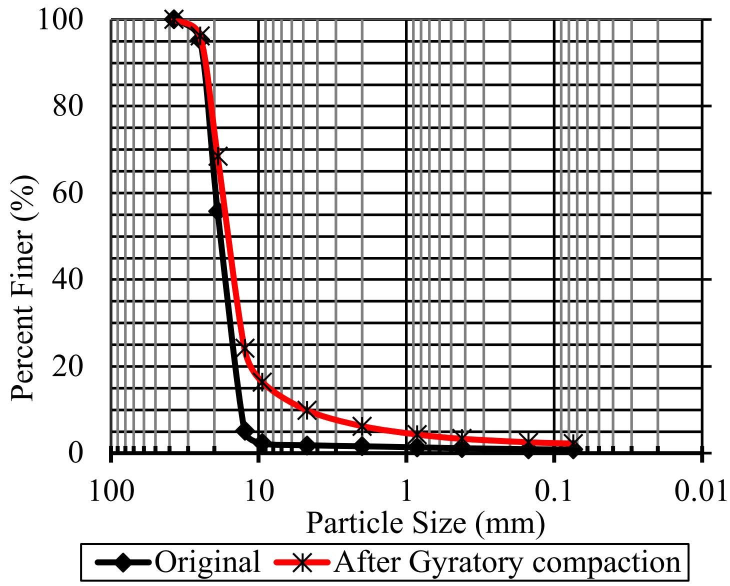


Figure A.1 Degradation of aggregates after gyratory compaction of DV2 (Clean 1.5")

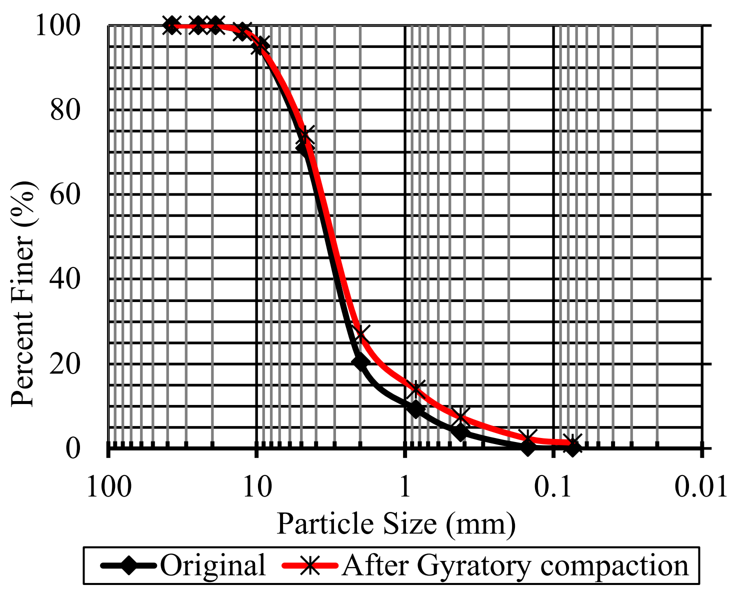


Figure A.2 Degradation of aggregates after gyratory compaction of DV3 (Road Gravel)

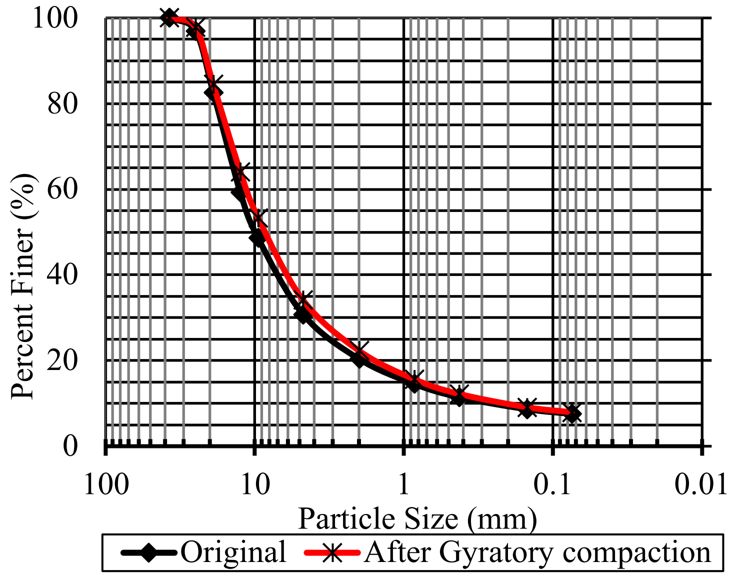


Figure A.3 Degradation of aggregates after gyratory compaction of DV4 (Crusher run 1.5 in)

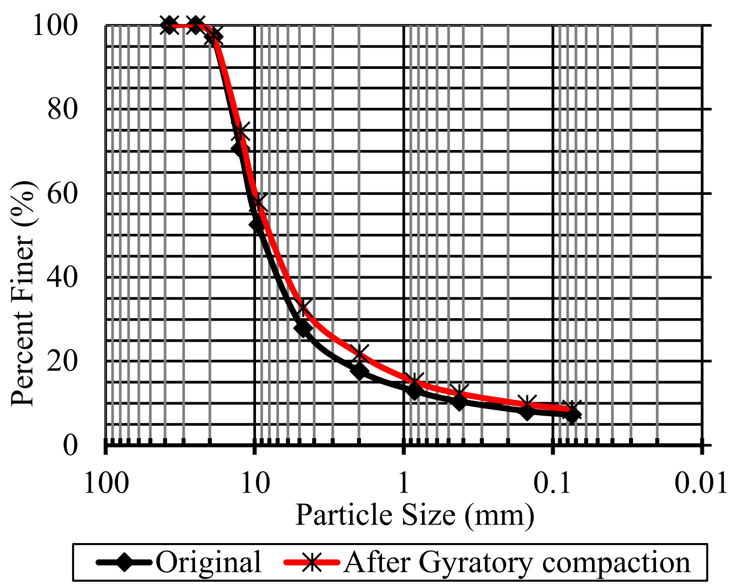


Figure A.4 Degradation of aggregates after gyratory compaction of DV5 (Crusher run 0.75 in)

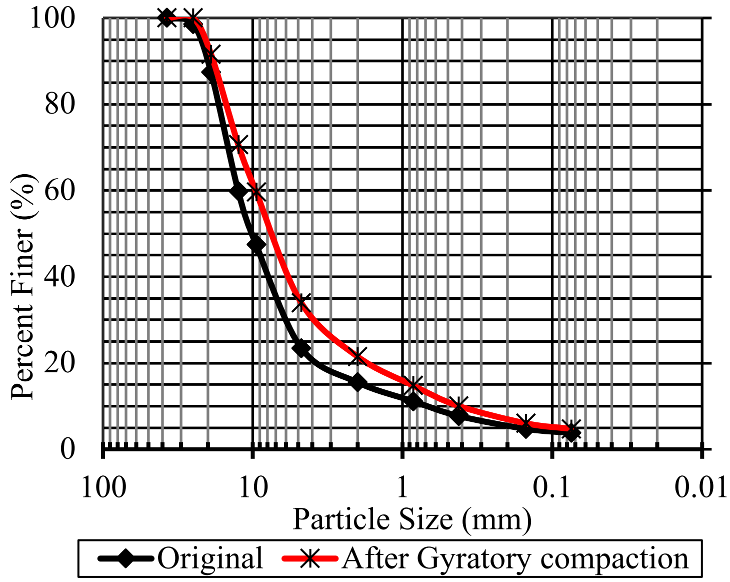


Figure A.5 Degradation of aggregates after gyratory compaction of HV1 (Crusher run 1 in)

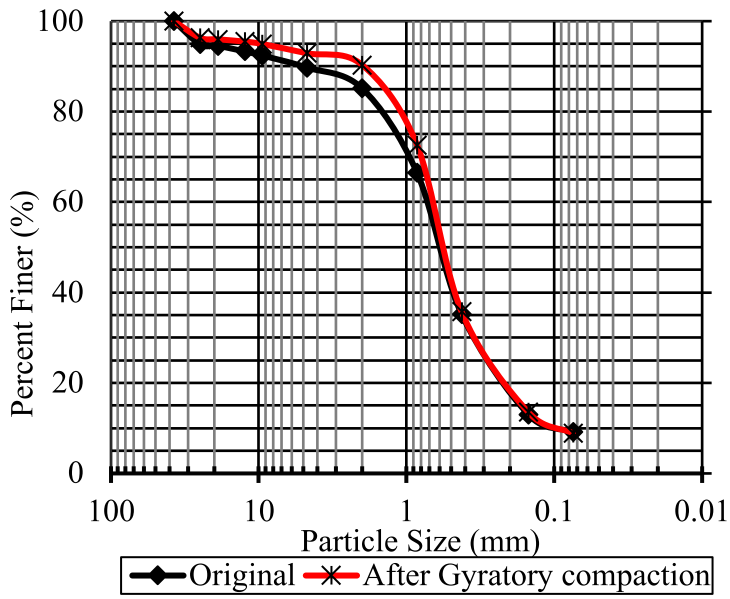


Figure A.6 Degradation of aggregates after gyratory compaction of HV2 (Surface sand)

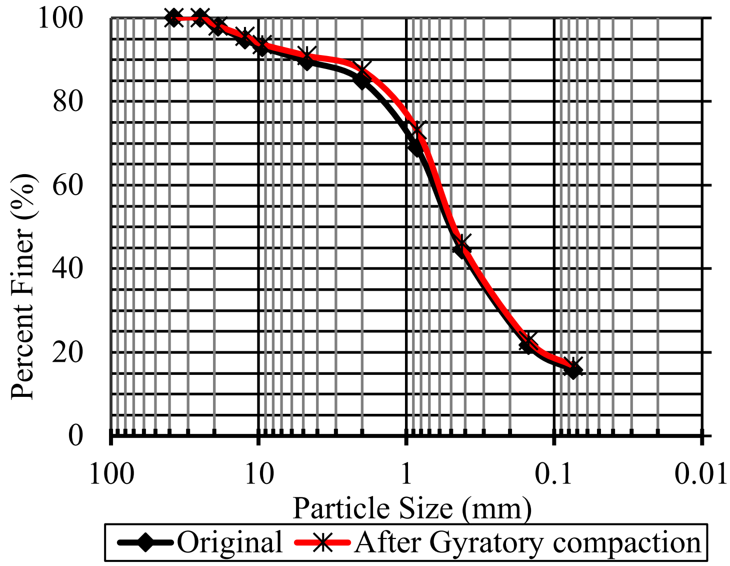


Figure A.7 Degradation of aggregates after gyratory compaction of HV3 (Green surface sand)

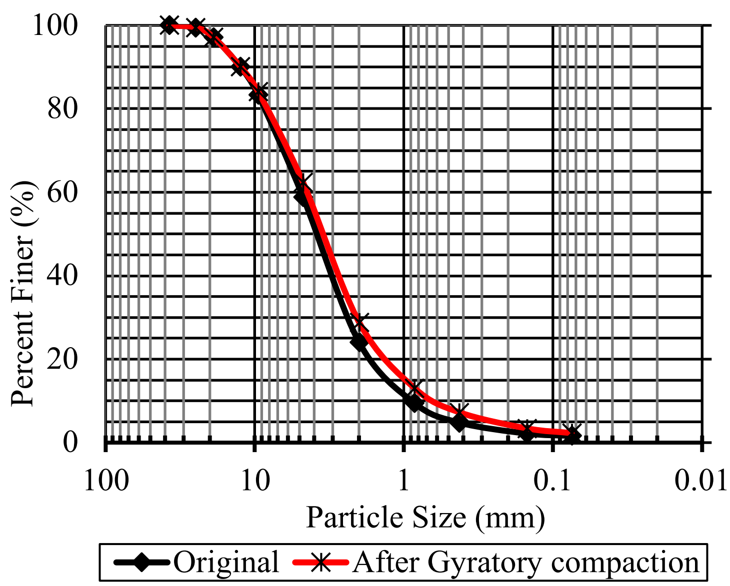


Figure A.8 Degradation of aggregates after gyratory compaction of HV4 (Gravel)

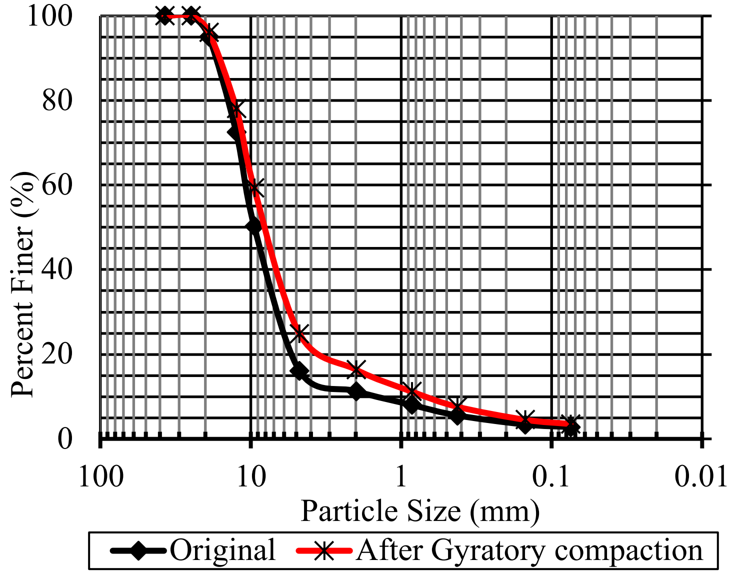


Figure A.9 Degradation of aggregates after gyratory compaction of SV1 (Rock 3/8 in)

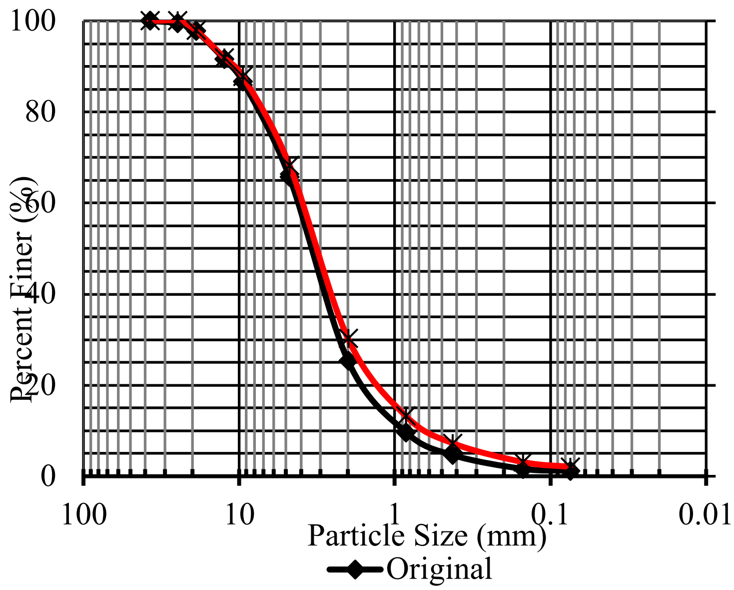


Figure A.10 Degradation of aggregates after gyratory compaction of SV2 (Road gravel)

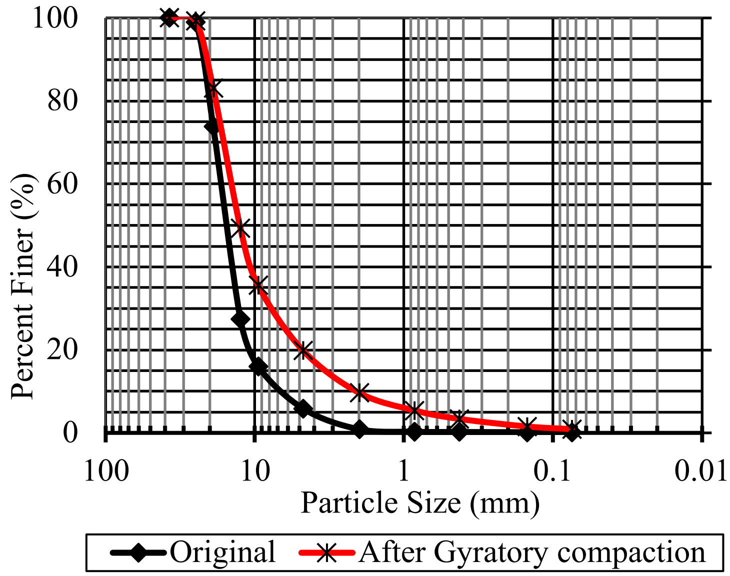


Figure A.11 Degradation of aggregates after gyratory compaction of CV1 (Section 27)

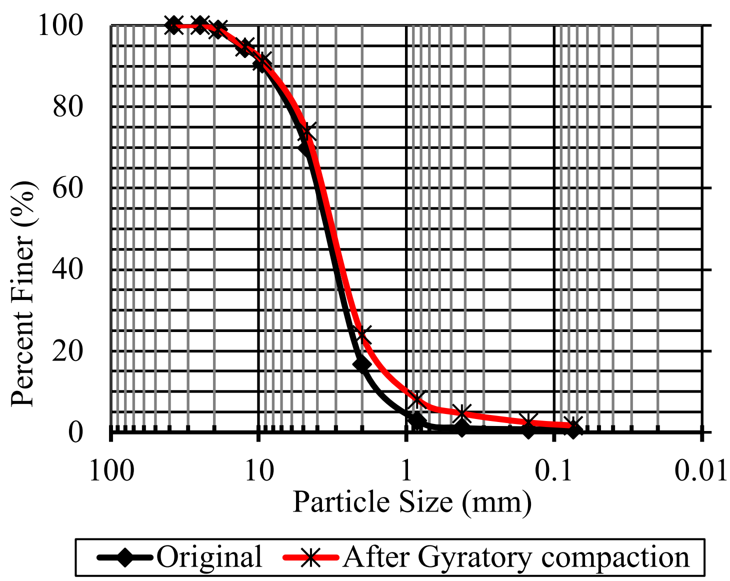


Figure A.12 Degradation of aggregates after gyratory compaction of CV2 (Section 7)

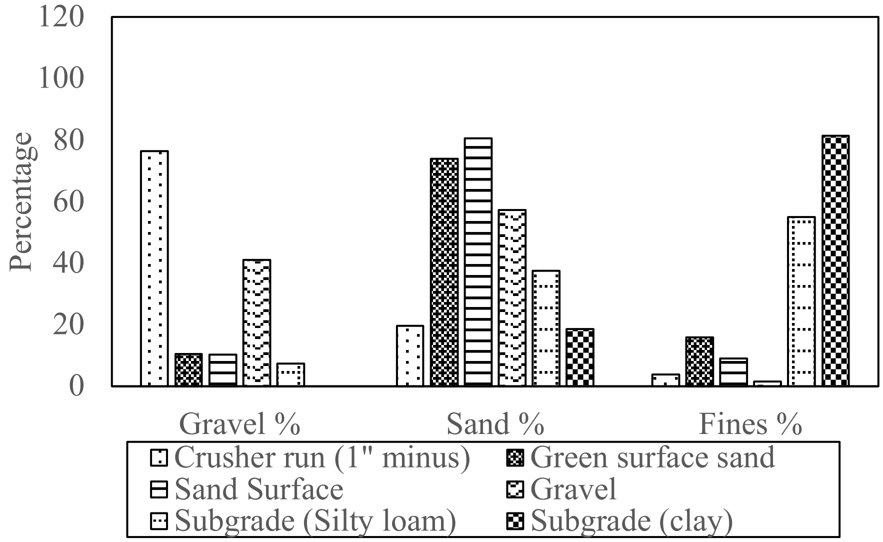


Figure A.13 Gravel, Sand and fines content varaitions of the Harlan county virigin materials

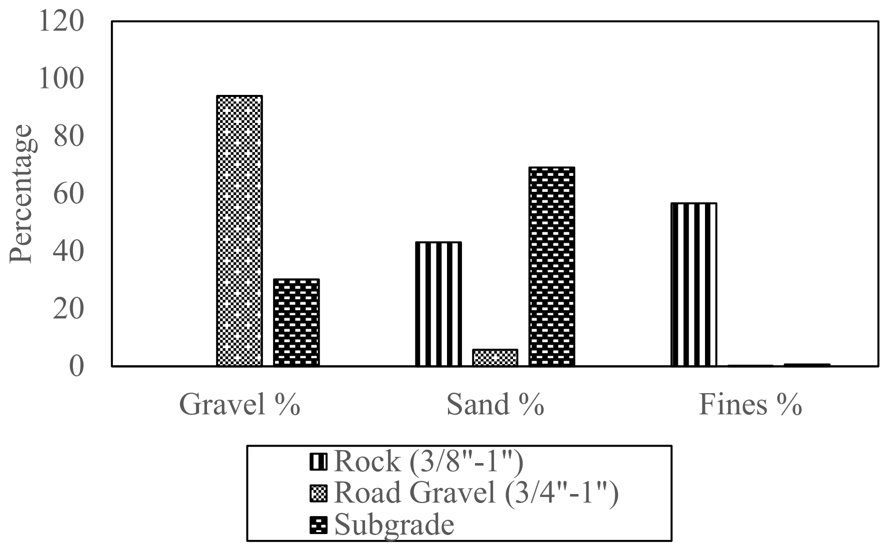


Figure A.14 Gravel, Sand and fines content varaitions of the Scottsbluff county virigin materials

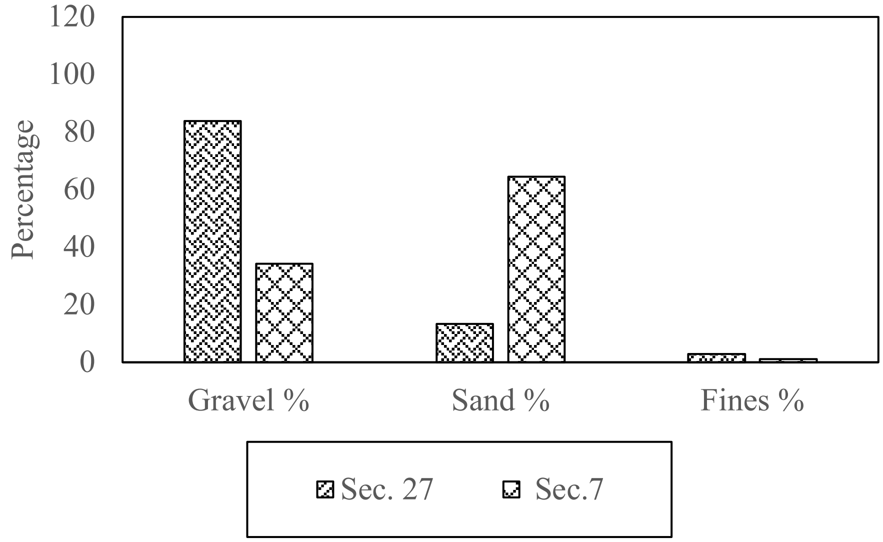


Figure A.15 Gravel, Sand and fines content varaitions of the Cherry county virigin materials

Table A.1 Resilient Modulus testing sequence for subgrade materials

Seq. No.	Confining Pressure (kPa)	Cyclic Stress (kPa)	No of Load Applications
0	41.4	24.8	500–1000
1	41.4	12.4	100
2	41.4	24.8	100
3	41.4	37.3	100
4	41.4	49.7	100
5	41.4	62.0	100
6	27.6	12.4	100
7	27.6	24.8	100
8	27.6	37.3	100
9	27.6	49.7	100
10	27.6	62.0	100
11	13.8	12.4	100
12	13.8	24.8	100
13	13.8	37.3	100
14	13.8	49.7	100
15	13.8	62.0	100

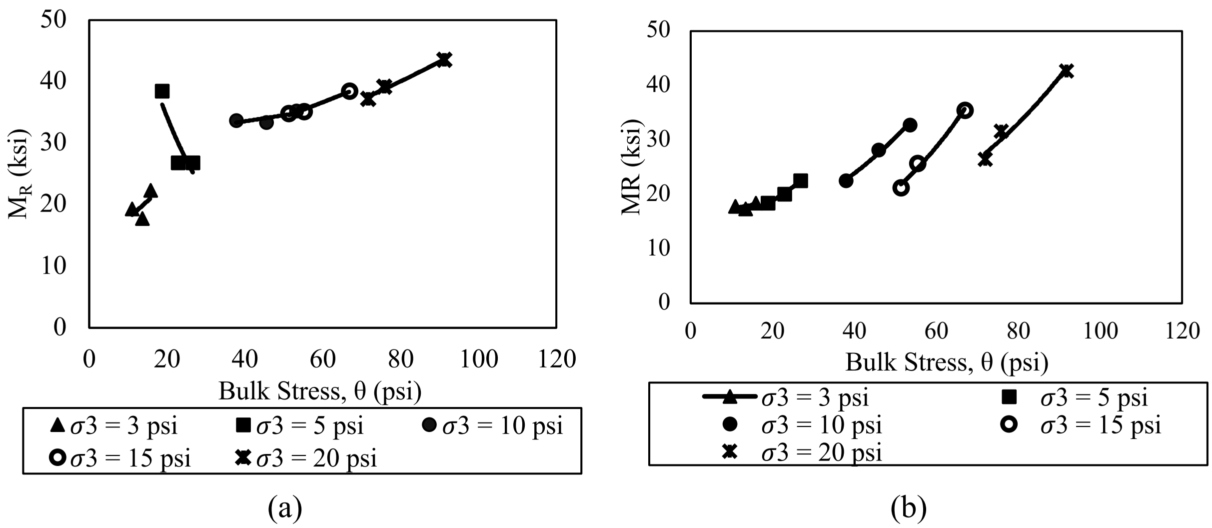


Figure A.16 Resilient modulus varying with bulk stress, θ (a) DV2 – Clean (1”), (b) DV5 – Crusher run (0.75”)

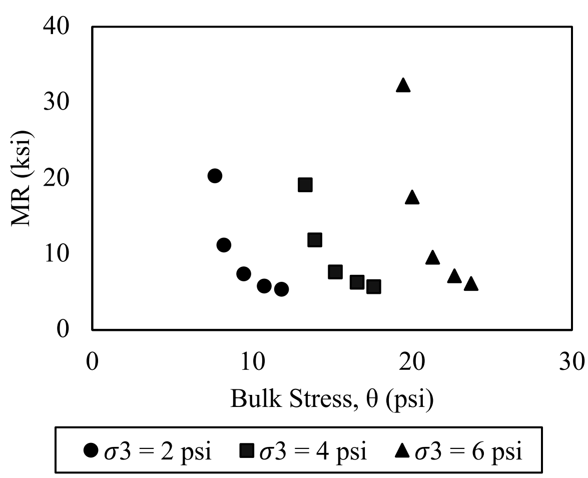


Figure A.17 Resilient modulus varying with bulk stress, HV6 - Subgrade (clay)

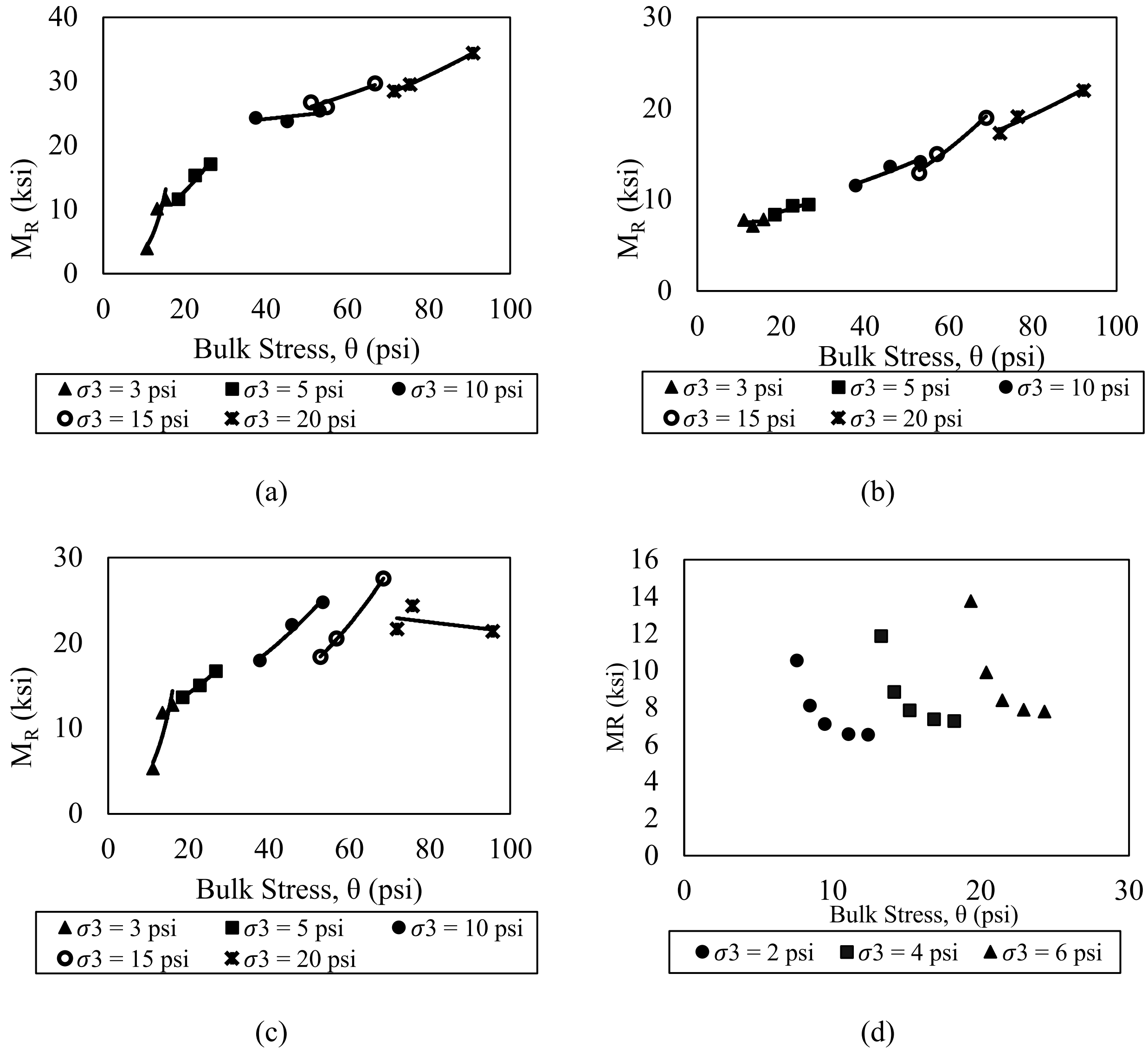
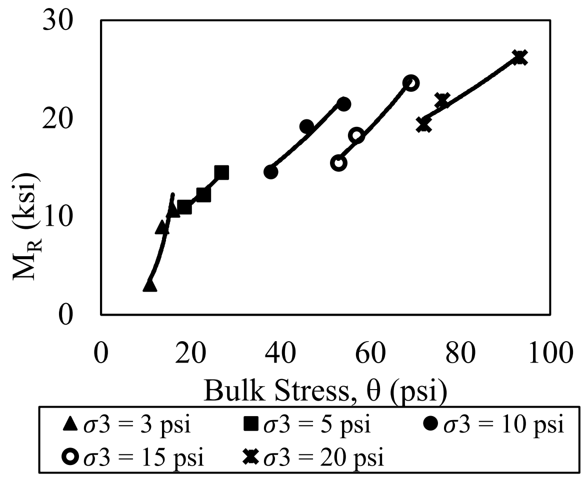
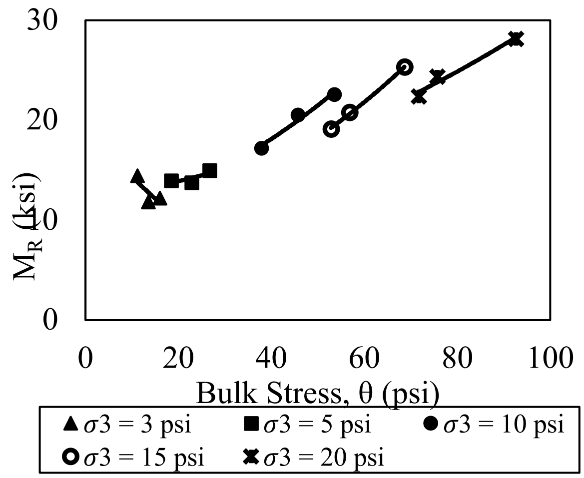


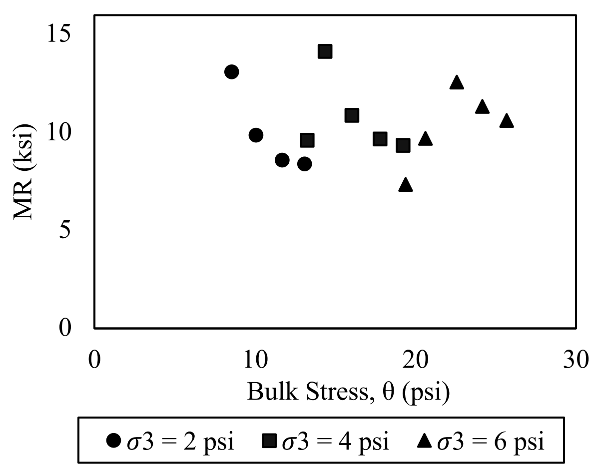
Figure A.18 Resilient modulus varying with bulk stress, θ (a) HV1 – Crusher run (1”-) (b) HV2 – Green surface sand (c) HV4 – Gravel (d) HV5 – Subgrade (Silty loam)



(a)

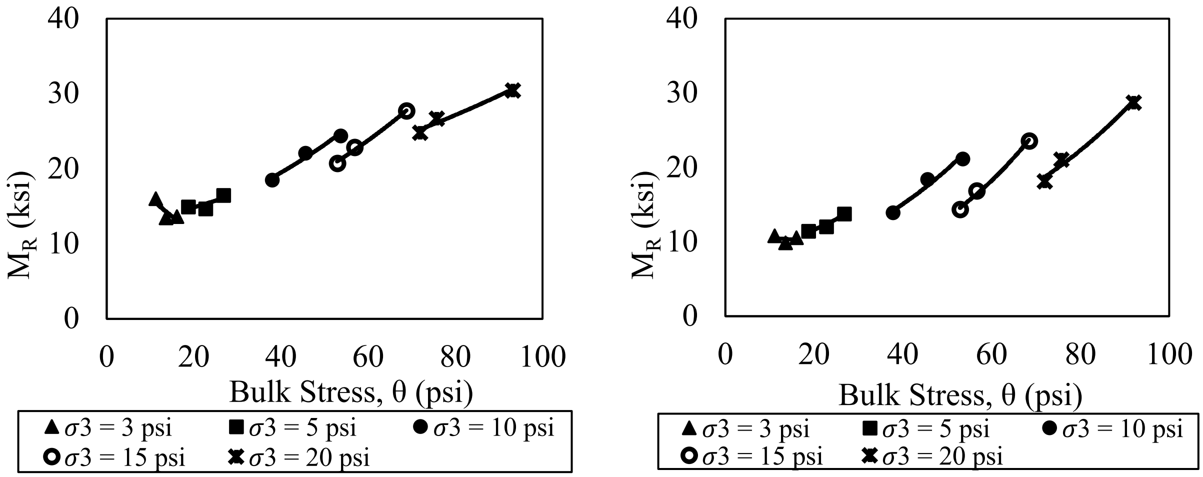


(b)



(c)

Figure A.19 Resilient modulus varying with bulk stress, (a)SV1 – Rock (b) SV2 – Road Gravel (c) SV3 - Subgrade



(a) (b)

Figure A.20 Resilient modulus varying with bulk stress, (a) CV1 – Section 27 (b) CV2 –Section 7

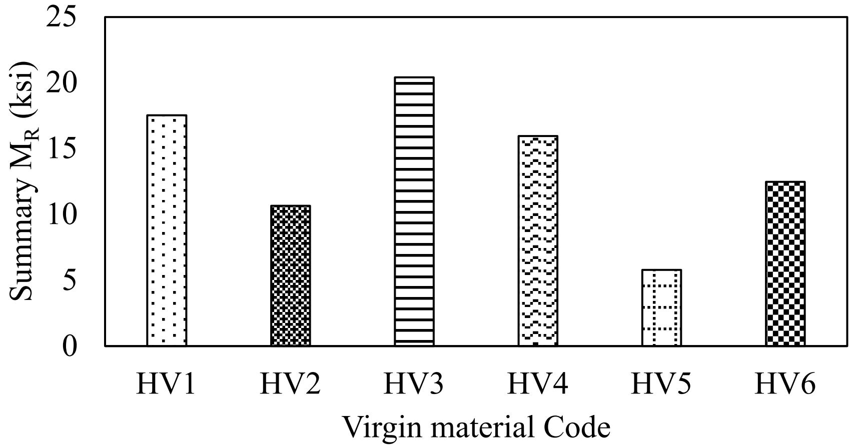


Figure A.21 Summary M_R variation for the Harlan County virgin materials

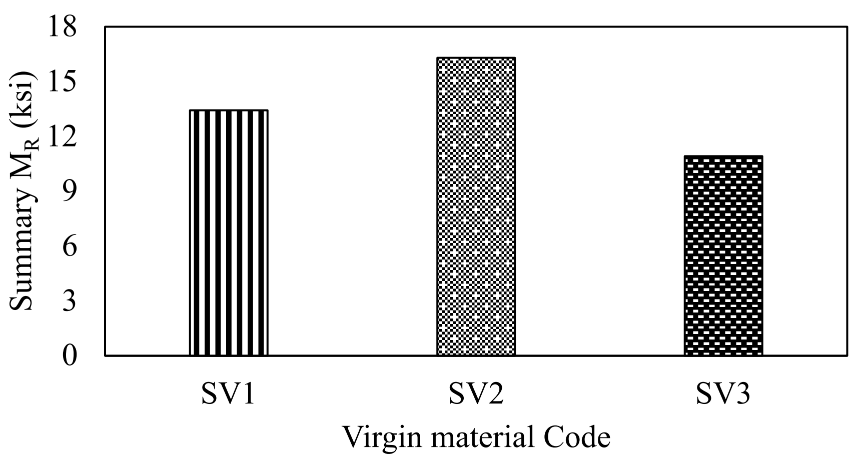


Figure A.22 Summary M_R variation for the Scottsbluff County virgin materials

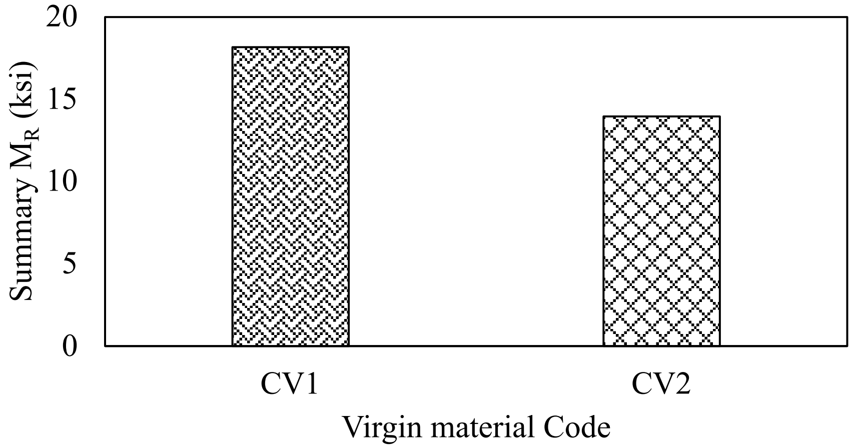


Figure A.23 Summary M_R variation for the Cherry County virgin materials

Appendix B

Table B.1 Details of Harlan County blends H1, H2, H3, and H4

Code	Materials	Gravel (%)	Sand (%)	Fines (%)
H1	CR (1" minus) [90%] + GSS [6%] + Sub (clay) [4%]	69.5	22.9	7.6
H2	CR (1" minus) [80%] + GSS Sand [12%] + Sub (clay) [8%]	62.5	26.1	11.4
H3	CR (1" minus) [70%] + SS [18%] + Sub (clay)[12%]	55.4	30.5	14.0
H4	CR (1" minus) [60%] + SS [24%] + Sub (clay)[16%]	48.4	34.2	17.5

CR = Crusher run, GSS – Green Surface Sand, SS.- Surface sand, Sub - Subgrade

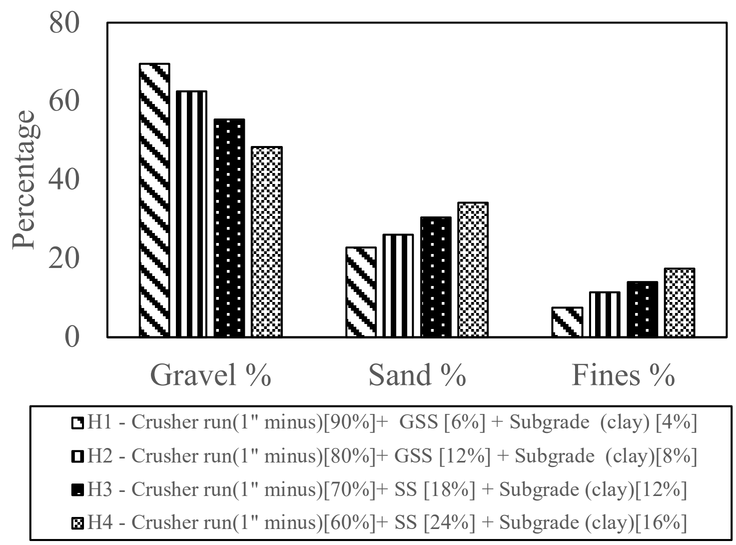


Figure B.1 Gravel, Sand, Fines content variations of Harlan County blends H1, H2, H3, and H4

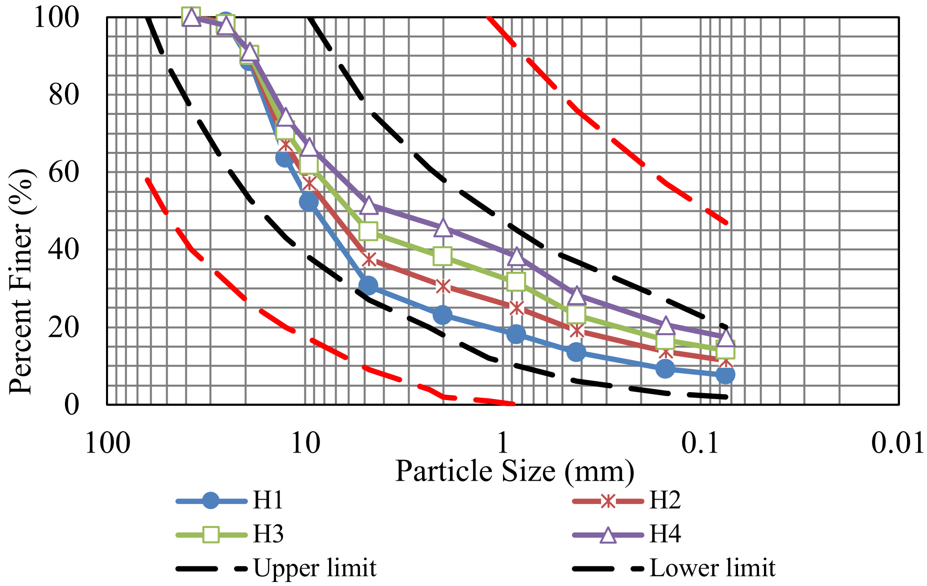


Figure B.2 Gradation curves of the blends H1, H2, H3, and H4 with recommended gradation limits

Table B.2 Details of Harlan County blends H5, H6, H7, and H8

Code	**Materials**	**Gravel (%)**	**Sand (%)**	**Fines (%)**
H5	CR (1" minus) [85%] + GSS [11%] + Sub (Silty loam) [4%]	66.5	26.4	7.2
H6	CR (1" minus) [75%] + GSS [17%] + Sub (Silty loam) [8%]	59.7	30.3	9.9
H7	CR (1" minus) [60%] + SS [24%] + Sub (Silty loam) [16%]	49.6	37.2	13.3
H8	CR (1" minus) [45%] + SS [35%] + Sub (Silty loam) [20%]	39.5	44.6	15.9

CR = Crusher run, GSS – Green Surface Sand, SS.- Surface sand, Sub - Subgrade

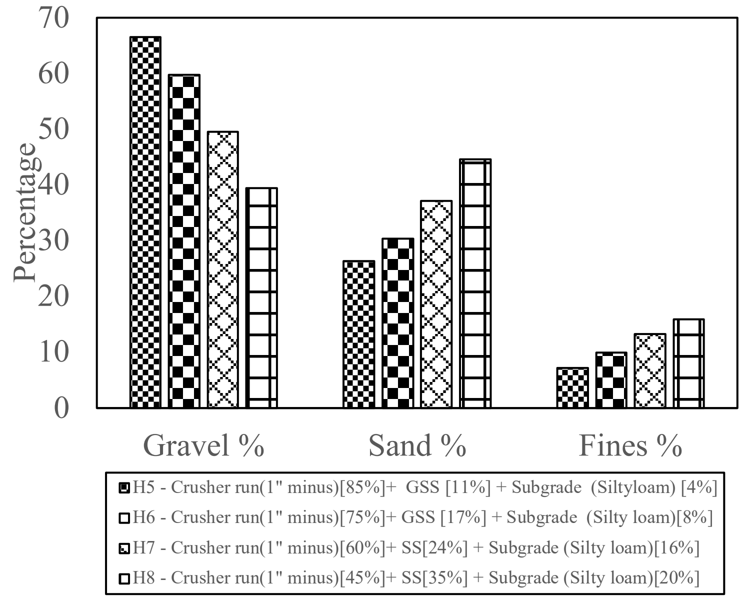


Figure B.3 Gravel, Sand, Fines content variations of Harlan County blends H5, H6, H7, and H8

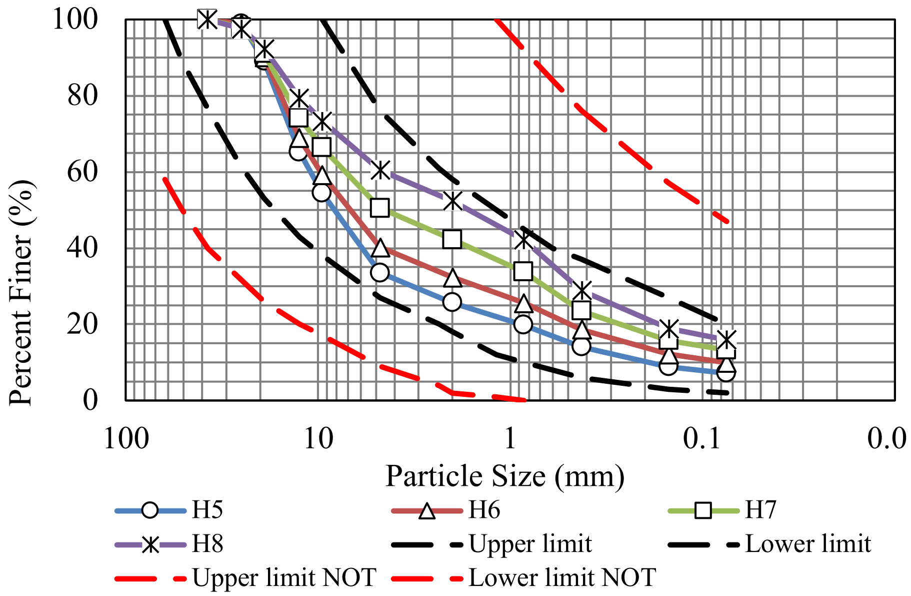


Figure B.4 Gradation curves of the blends H5, H6, H7, and H8 with recommended gradation limits

Table B.3 Details of Harlan County blends H9, H10, H11, and H12

Code	Materials	Gravel (%)	Sand (%)	Fines (%)
H9	CR (1" minus) [70%] + Gravel [25%] + Sub (silt) [5%]	64.2	30.0	5.8
H10	CR (1" minus) [50%] + Gravel [40%] + Sub (silt) [10%]	55.4	36.5	8.0
H11	CR (1" minus) [30%] + Gravel [55%] + Sub (clay) [15%]	45.6	40.2	14.2
H12	CR (1" minus) [10%] + Gravel [70%] + Sub (clay) [20%]	36.4	45.8	17.8

CR = Crusher run, Sub - Subgrade

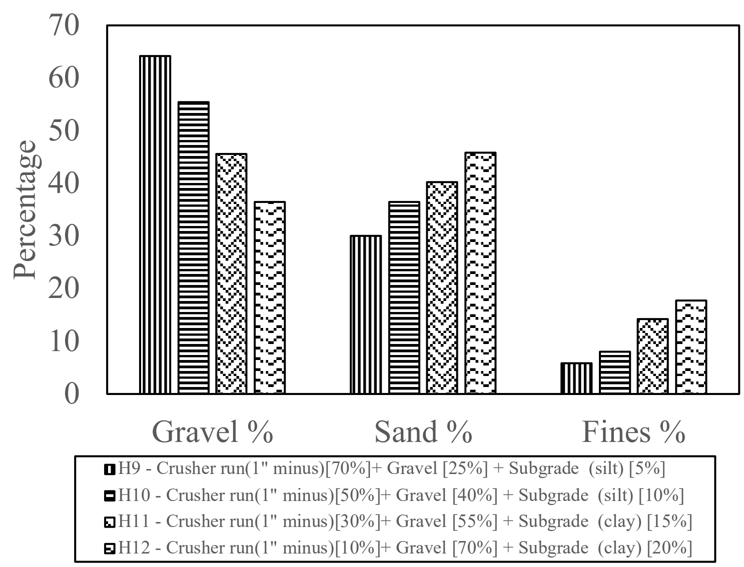


Figure B.5 Gravel, Sand, Fines variation of Harlan County blends H9, H10, H11, and H12

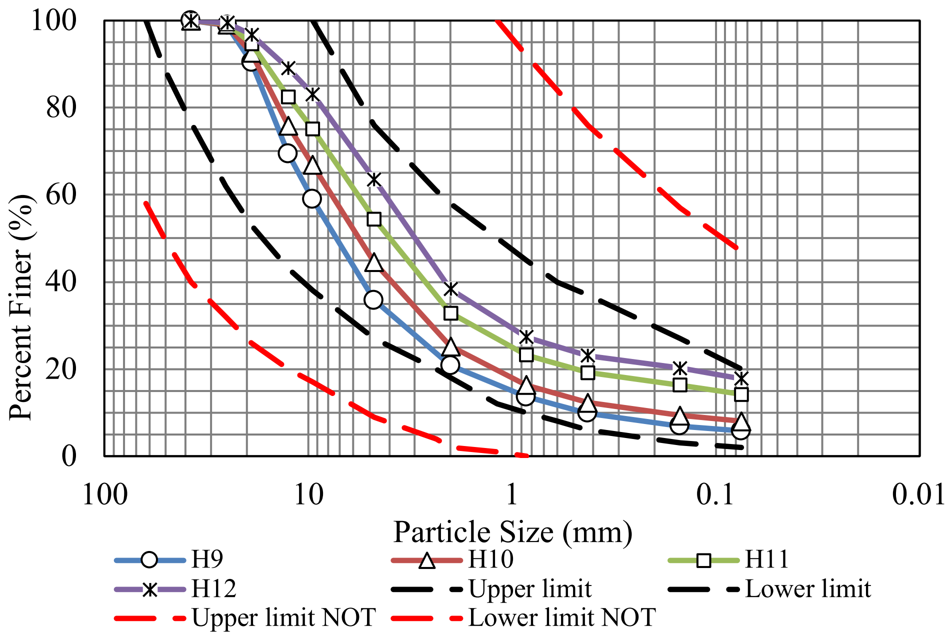


Figure B.6 Gradation curves of the blends H9, H10, H11, and H12 with recommended gradation limits

Table B.4 Proctor Compaction results of the Harlan County Blends

Blend No.	MDU (pcf)	OMC (%)	Corrected MDU (pcf)	Corrected OMC (%)
H1	122.16	9.4	124.41	8.5
H2	126.65	7.8	128.44	7.2
H3	127.35	7.1	129.11	6.6
H4	127.50	8.7	130.04	8.1
H5	122.24	8.9	124.38	8.1
H6	128.37	7.4	129.92	6.8
H7	128.17	7.6	129.84	7.1
H8	129.4	8.3	130.89	7.8
H9	126.1	8.4	127.94	7.7
H10	128.89	7.1	130.29	6.7
H11	133.24	7.2	134.23	6.9
H12	135.02	6.4	NA	NA

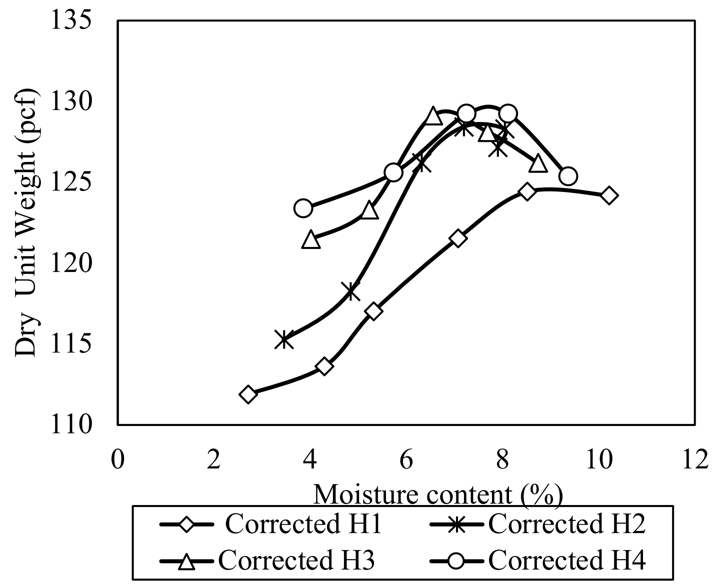


Figure B.7 Proctor Compaction curves for the blends H1, H2, H3, and H4

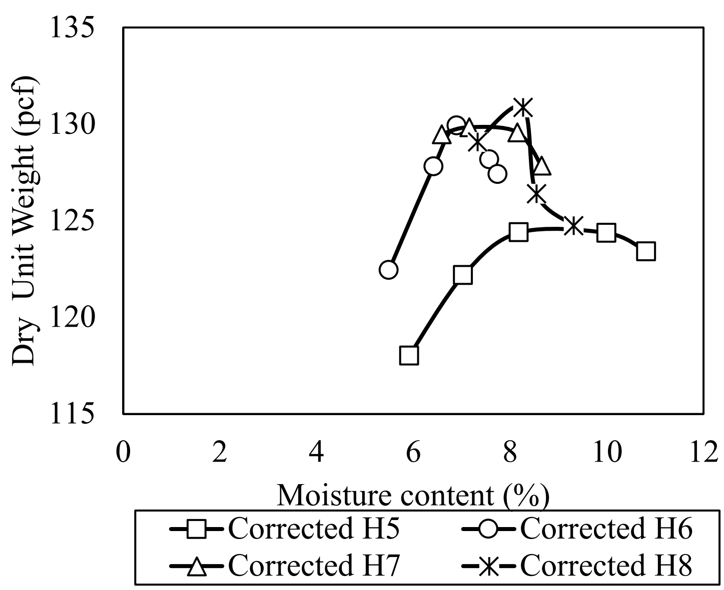


Figure B.8 Proctor Compaction curves for the blends H5, H6, H7, and H8

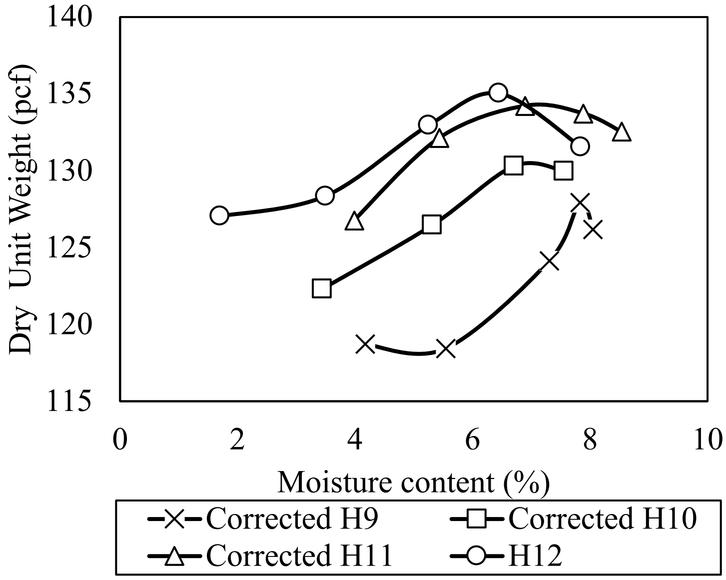


Figure B.9 Proctor Compaction curves for the blends H9, H10, H11, and H12

Table B.5 Details of Scottsbluff County blends

Code	**Materials**	**Gravel (%)**	**Sand (%)**	**Fines (%)**
S1	Rock (3/8") [60%] + RG (0.75') [36%] +Sub [4%]	62.7	32.9	4.4
S2	Rock (3/8") [50%] + RG (0.75') [42%] + Sub [8%]	56.4	37.2	6.5
S3	Rock (3/8") [40%] + RG (0.75') [44%] + Sub [16%]	48.7	40.6	10.7
S4	Rock (3/8") [30%] + RG (0.75') [45%] + Sub [25%]	40.6	43.8	15.6
S5	Rock (3/8") [10%] + RG (0.75') [60%] + Sub [30%]	29.0	53.0	18.0

RG = Road Gravel, Sub - Subgrade

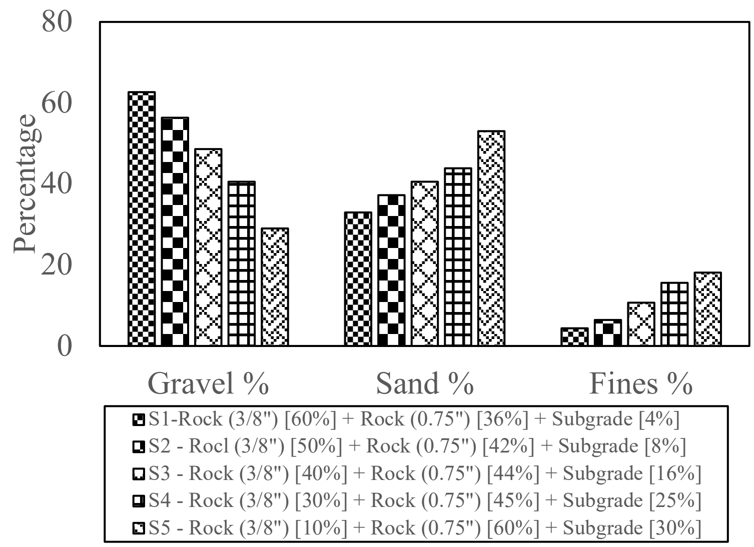


Figure B.10 Gravel, Sand, Fines proportions of Scottsbluff County blends S1, S2, S3, S4, and S5

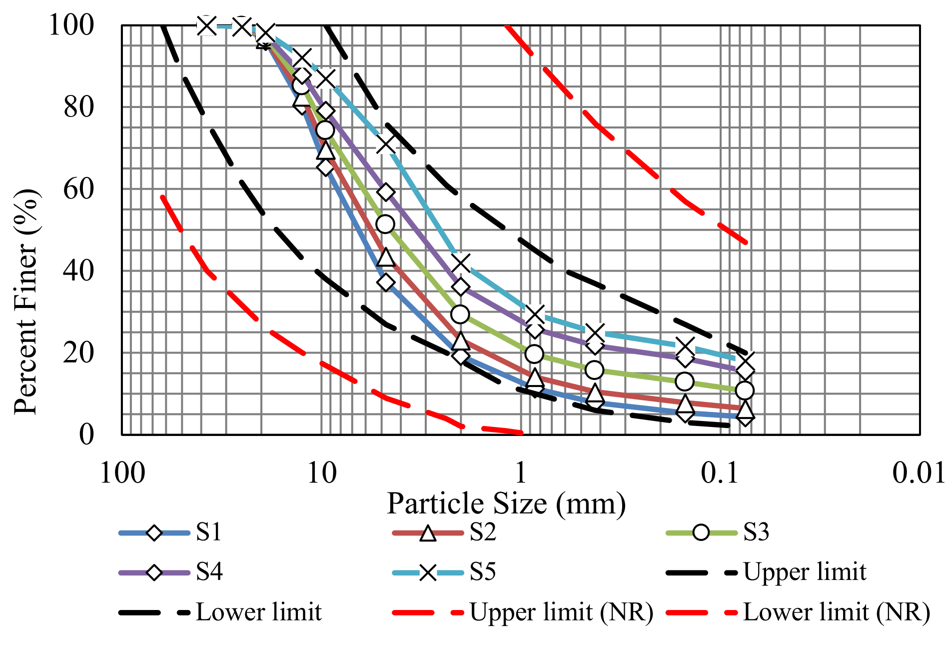


Figure B.11 Gradation curves of the blends S1, S2, S3, S4, and S5 with recommended limits

Table B.6 Compaction result of Scottsbluff County blends

Blend No	MDU (pcf)	OMC (%)
S1	134.3	7.3
S2	135.9	7.2
S3	134.7	6.8
S4	129.4	6.8
S5	126.2	7.8

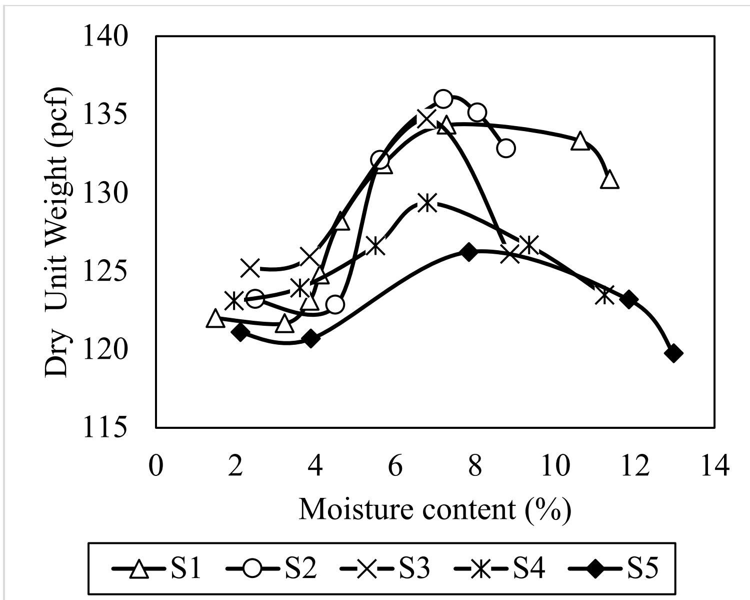


Figure B.12 Proctor Compaction curves for the blends S1, S2, S3, S4, and S5

Table B.7 Cherry County blends

Code	Materials	Gravel (%)	Sand (%)	Fines (%)
C1	Section 27 [50%] + Section 7 [40%] + Sub (Clay) [10%]	59.1	30.5	9.2
C2	Section 27 [40%] + Section 7 [46%] + Sub (Clay) [14%]	51.5	34.1	12.7
C3	Section 27 [30%] + Section 7 [52%] + Sub (Clay) [18%]	43.9	37.7	16.3
C4	Section 27 [20%] + Section 7 [58%] + Sub (Clay) [22%]	36.3	41.3	19.8

CR = Crusher run, Sub - Subgrade

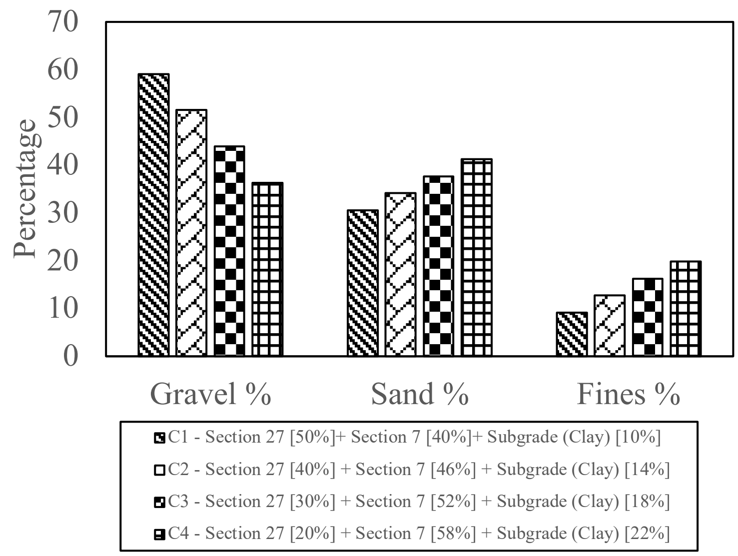


Figure B.13 Gravel, Sand, Fines proportions of Cherry County blends C1, C2, C3, and C4

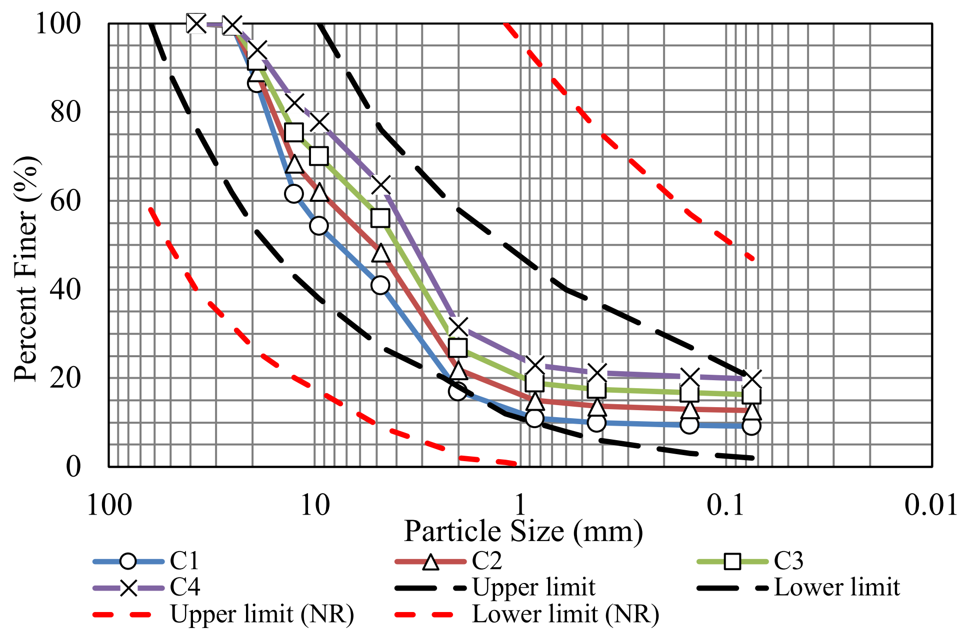


Figure B.14 Gradation curves of the blends C1, C2, C3, and C4 with recommended limits

Table B.8 Proctor Compaction results of Cherry County

Blend No.	MDU (pcf)	OMC (%)	Corrected MDU (pcf)	Corrected OMC (%)
1	135.473	7.2	138.44	6.5
2	134.739	7.7	137.21	7.07
3	132.54	7.4	134.55	6.95
4	132.316	8.1	133.74	7.74

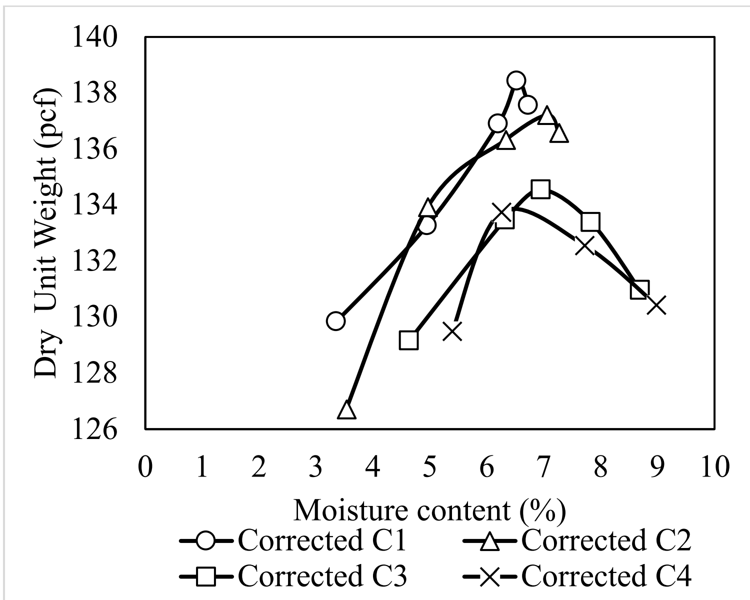


Figure B.15 Compaction curves for the blends C1, C2, C3, and C4

Table B.9 Gradation size distribution of Douglas County blends

Sieves		Percent passing									
Sieve No	**Sieve size (mm)**	**D1**	**D2**	**D3**	**D4**	**D5**	**D6**	**D7**	**D8**	**D9**	**D10**
1.5"	37.5	100	100	100	100	100	100	100	100	100	100
1"	25	85	90	98	99	99	100	97	98	100	100
3/4"	19	71	81	95	87	91	98	84	88	99	99
1/2"	12.5	70	80	94	71	80	94	63	71	85	92
3/8"	9.5	68	77	91	68	77	91	53	63	75	85
4	4.75	53	61	73	53	61	73	35	45	54	66
10	2	21	28	35	21	28	35	20	25	30	35
20	0.85	14	20	26	14	20	26	14	18	23	28
40	0.425	10	16	22	10	16	22	11	15	20	23
100	0.15	8	13	18	8	13	18	8	12	17	20
200	0.075	7	13	18	7	13	18	7	11	16	20

Table B.10 Gradation size distribution of Harlan County blends

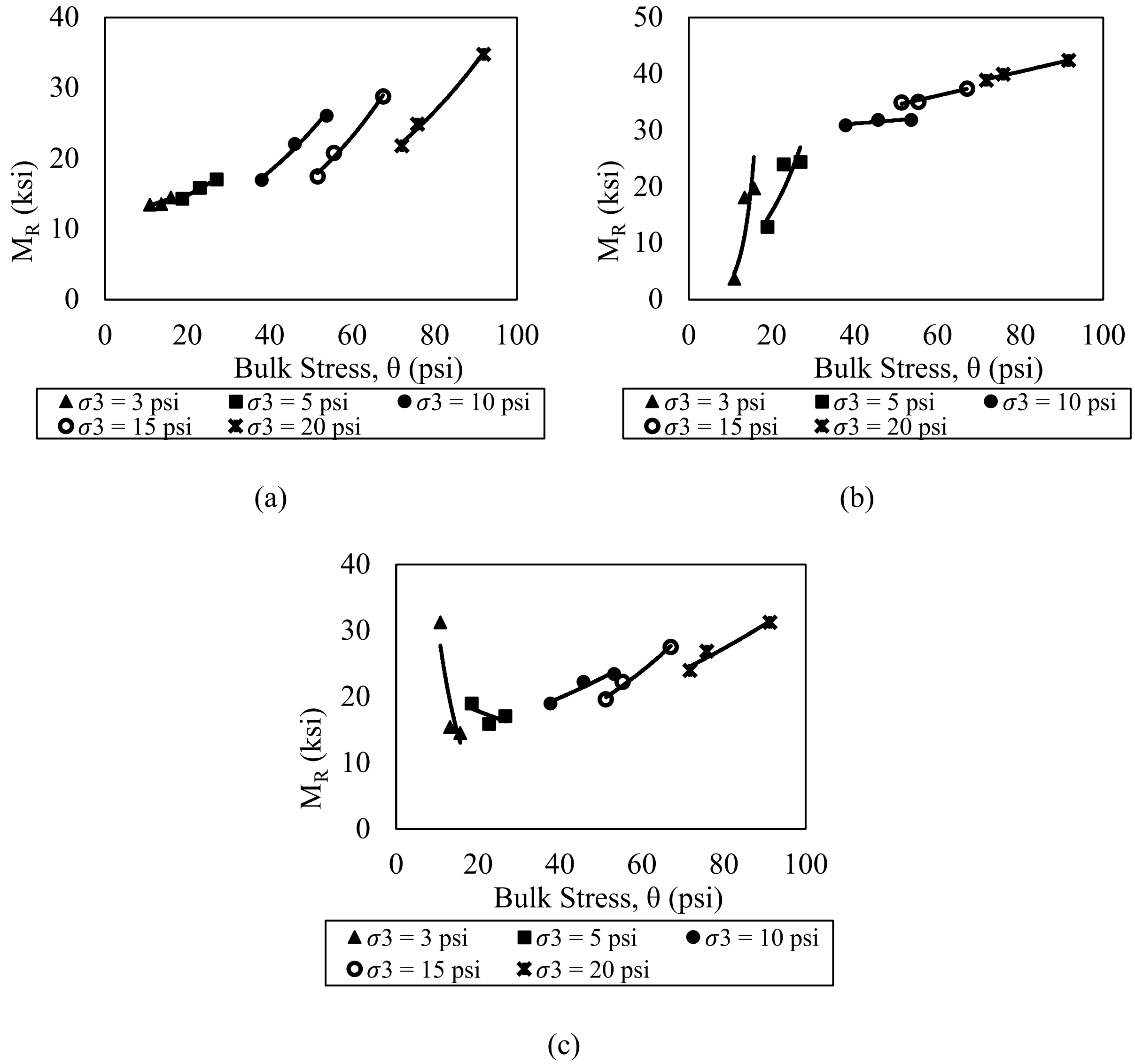
Sieves		Percent passing%											
Sieve No	**Sieve size (mm)**	**H1**	**H2**	**H3**	**H4**	**H5**	**H6**	**H7**	**H8**	**H9**	**H10**	**H11**	**H12**
1.5"	37.5	100	100	100	100	100	100	100	100	100	100	100	100
1"	25	99	99	98	98	99	99	98	98	99	99	99	99
3/4"	19	89	90	90	91	89	90	91	92	90	93	95	97
1/2"	12.5	64	67	71	74	65	69	74	79	69	76	82	89
3/8"	9.5	52	57	62	67	54	59	66	73	59	67	75	83
4	4.75	31	38	45	52	34	40	50	61	36	45	54	64
10	2	23	31	38	46	26	32	42	52	21	25	33	38
20	0.85	18	25	32	38	20	26	34	42	14	16	23	27
40	0.425	13	19	23	28	14	19	24	29	10	12	19	23
100	0.15	9	14	17	21	9	12	16	19	7	9	16	20
200	0.075	8	11	14	17	7	10	13	16	6	8	14	18

Table B.11 Gradation size distribution of Scottsbluff County blends

Sieves		Percent passing%				
Sieve No	**Sieve size (mm)**	**S1**	**S2**	**S3**	**S4**	**S5**
1.5"	37.5	100	100	100	100	100
1"	25	100	100	100	100	100
3/4"	19	96	97	97	98	98
1/2"	12.5	80	83	85	88	92
3/8"	9.5	65	70	74	79	87
4	4.75	37	44	51	59	71
10	2	19	23	29	36	42
20	0.85	11	14	20	26	29
40	0.425	8	11	16	22	25
100	0.15	5	8	13	19	22
200	0.075	4	6	11	16	18

Table B.12 Gradation size distribution of Cherry County blends

Sieves		Percent passing %			
Sieve No	**Sieve size (mm)**	C1	C2	C3	C4
1.5"	37.5	100	100	100	100
1"	25	99	100	100	100
3/4"	19	86	89	92	94
1/2"	12.5	61	68	75	82
3/8"	9.5	54	62	70	78
4	4.75	41	48	56	64
10	2	17	22	27	32
20	0.85	11	15	19	23
40	0.425	10	14	17	21
100	0.15	9	13	17	20
200	0.075	9	13	16	20



(a)

(b)

(c)

Figure B.16 Resilient modulus varying with bulk stress (a) D1, (b) D2, and (c) D3

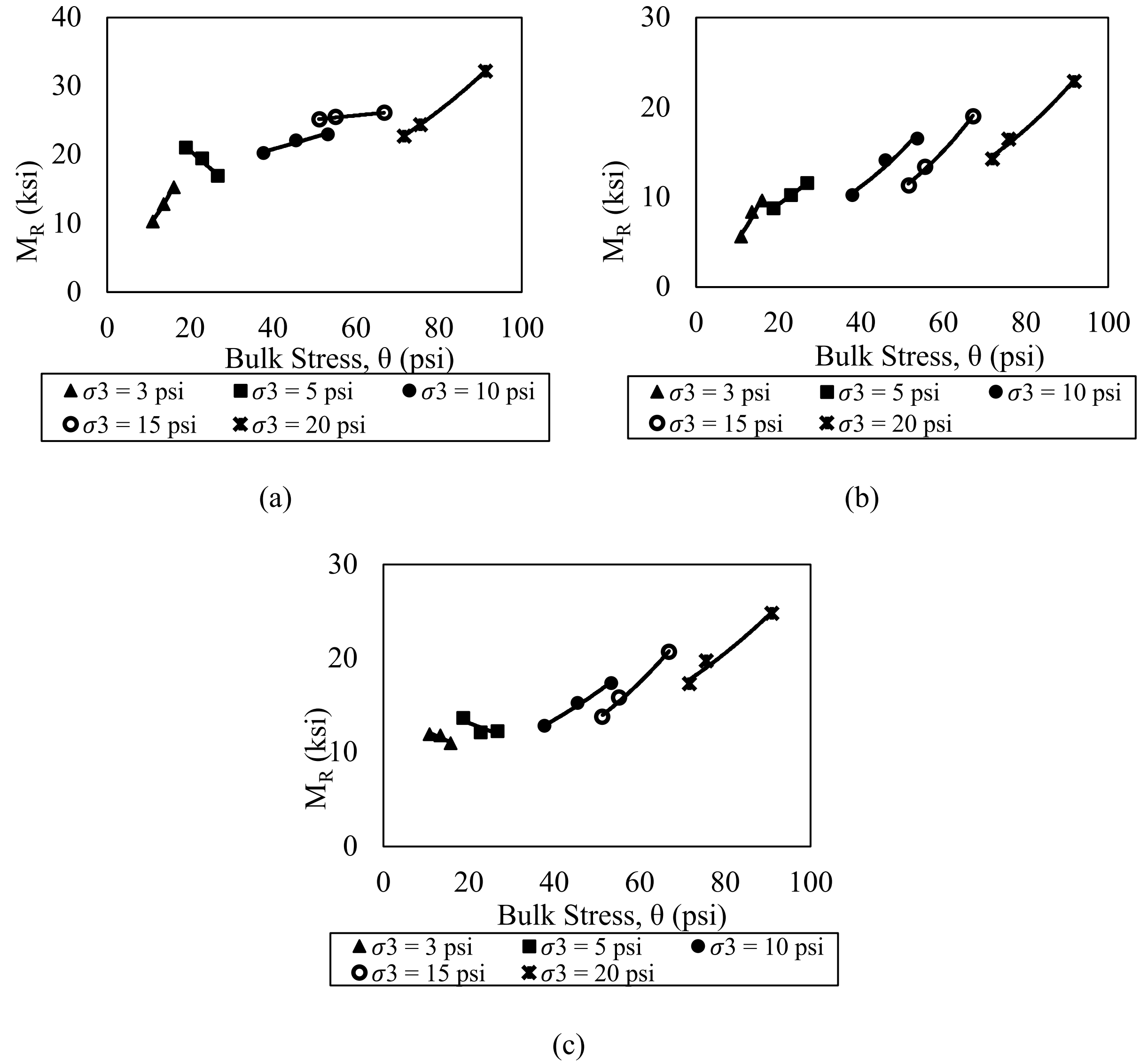


Figure B.17 Resilient modulus varying with bulk stress of Douglas blends (a) D4, (b) D5, and (c) D6

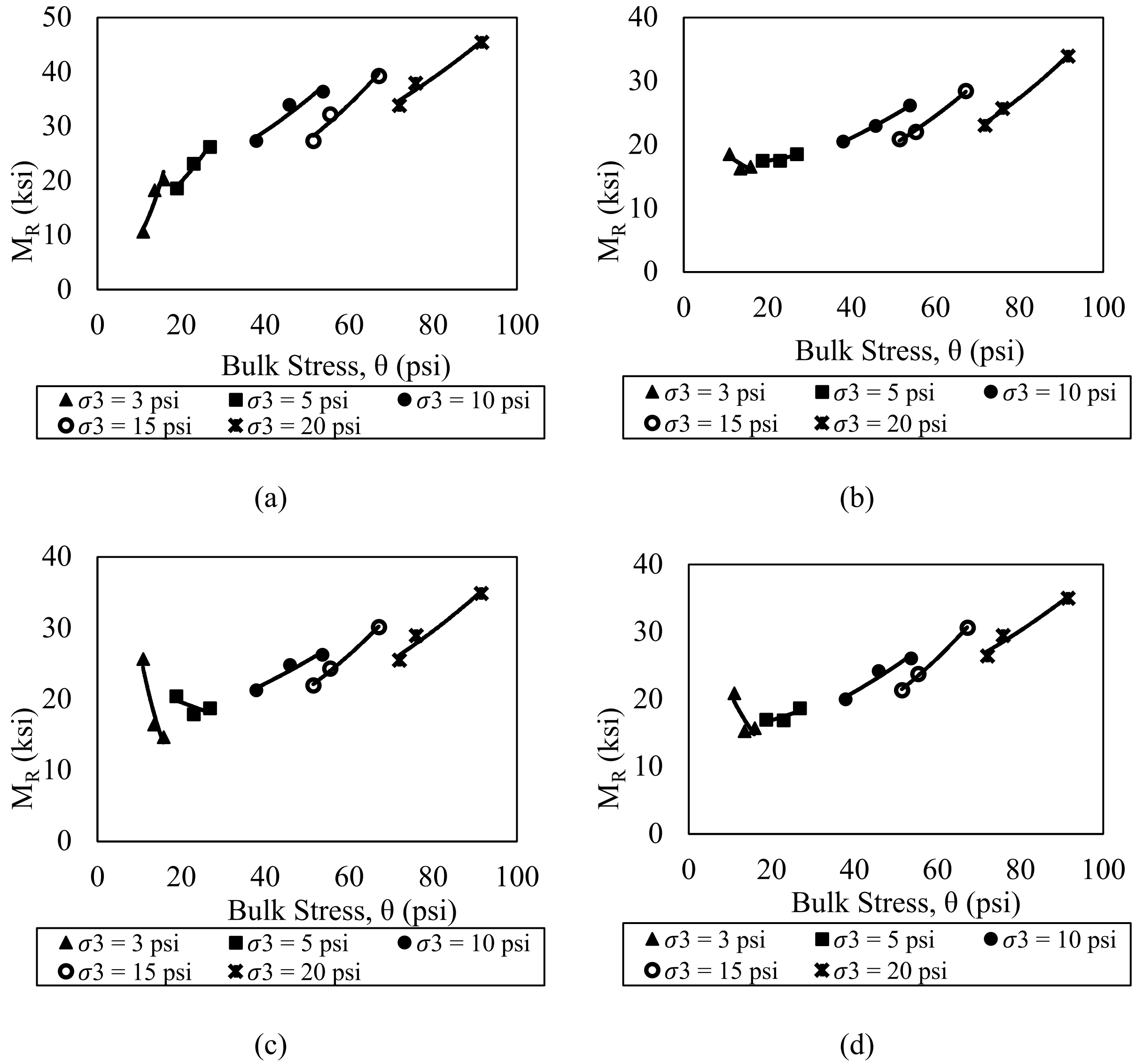


Figure B.18 Resilient modulus varying with bulk stress of Douglas blends (a) D7, (b) D8, (c) D9, and (d) D10

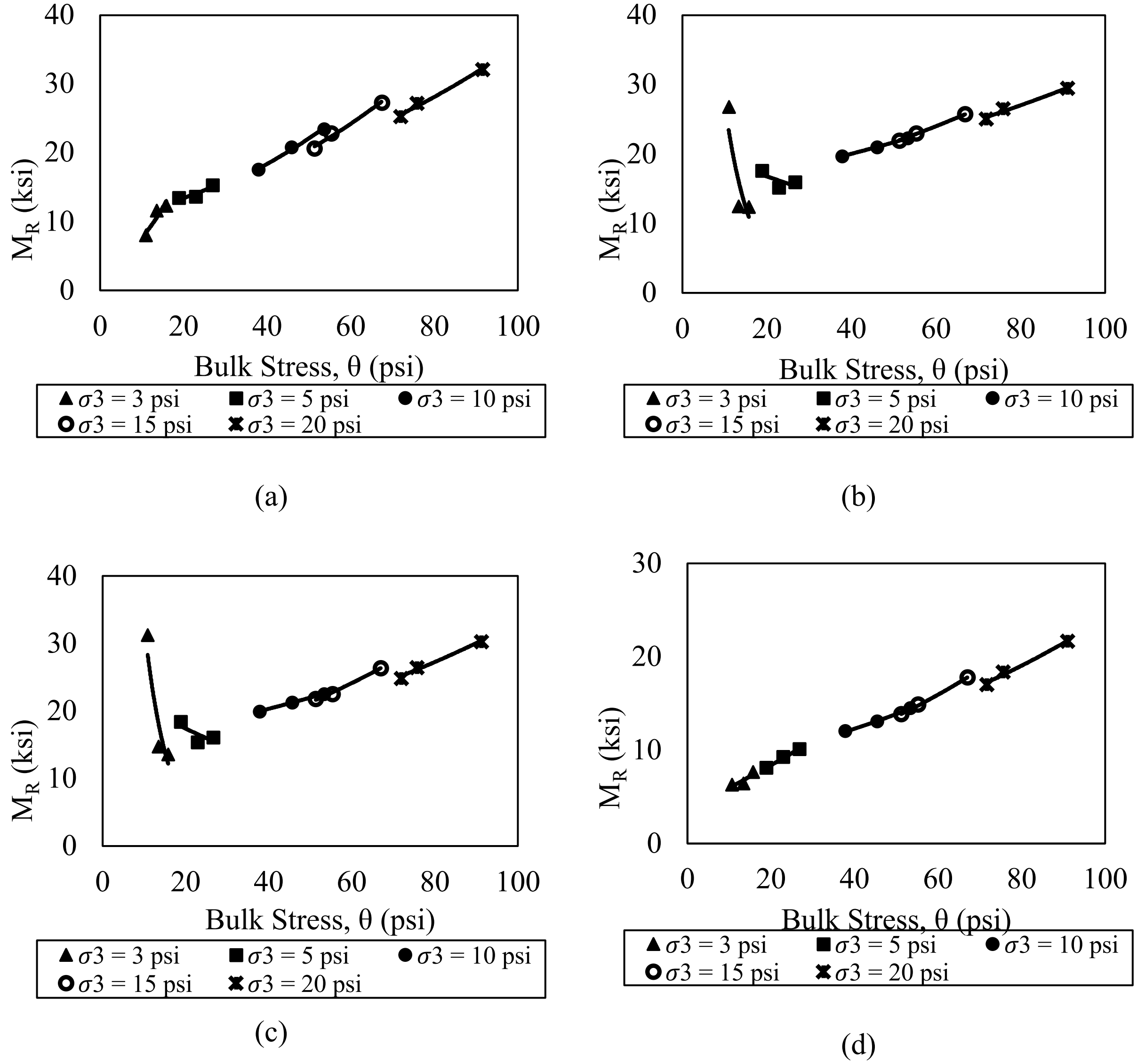


Figure B.19 Resilient modulus varying with bulk stress of Harlan blends (a) H1, (b) H2, (c) H3, and (d) H4

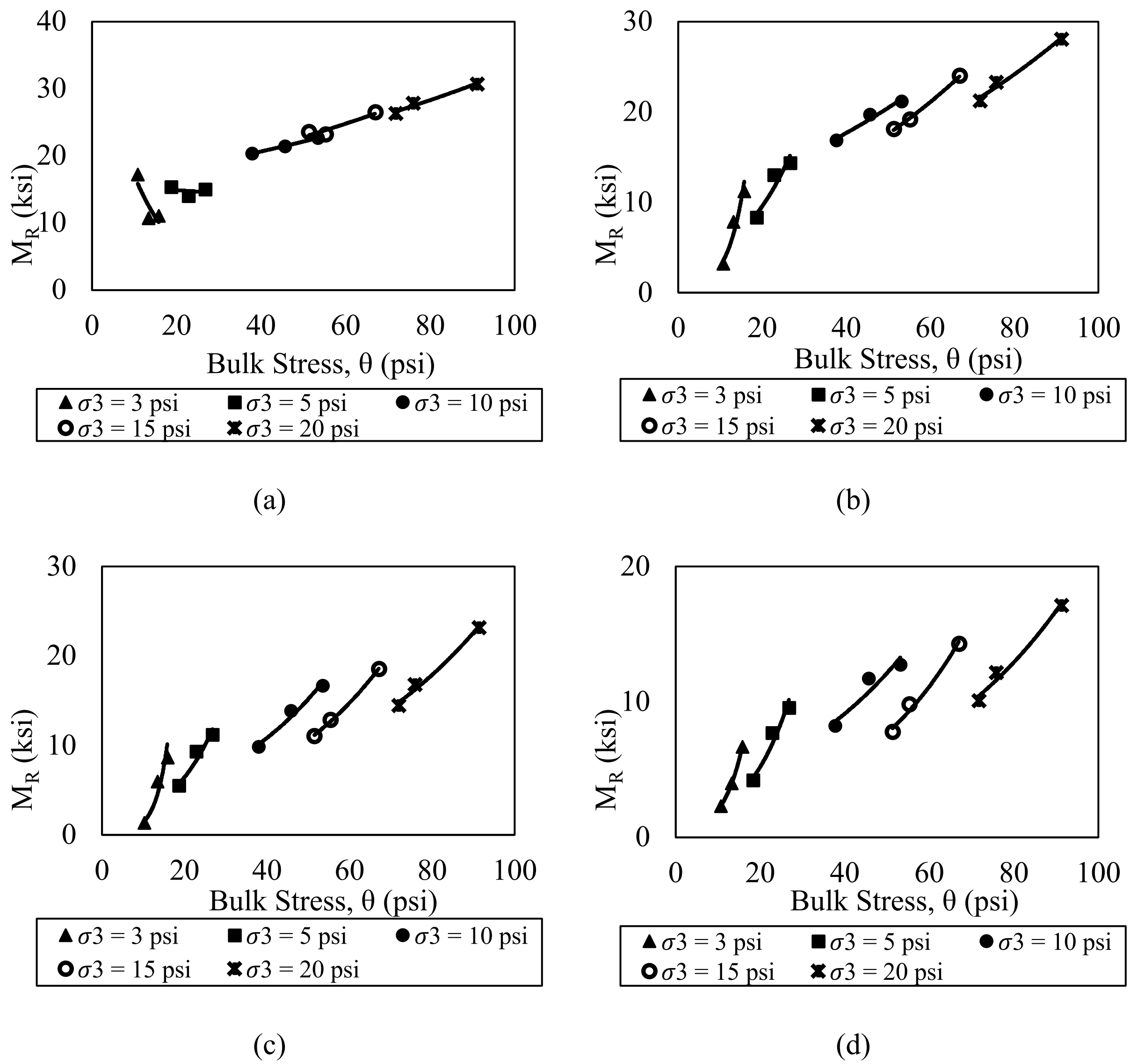


Figure B.20 Resilient modulus varying with bulk stress of Harlan blends (a) H5, (b) H6, (c) H7, and (d) H8

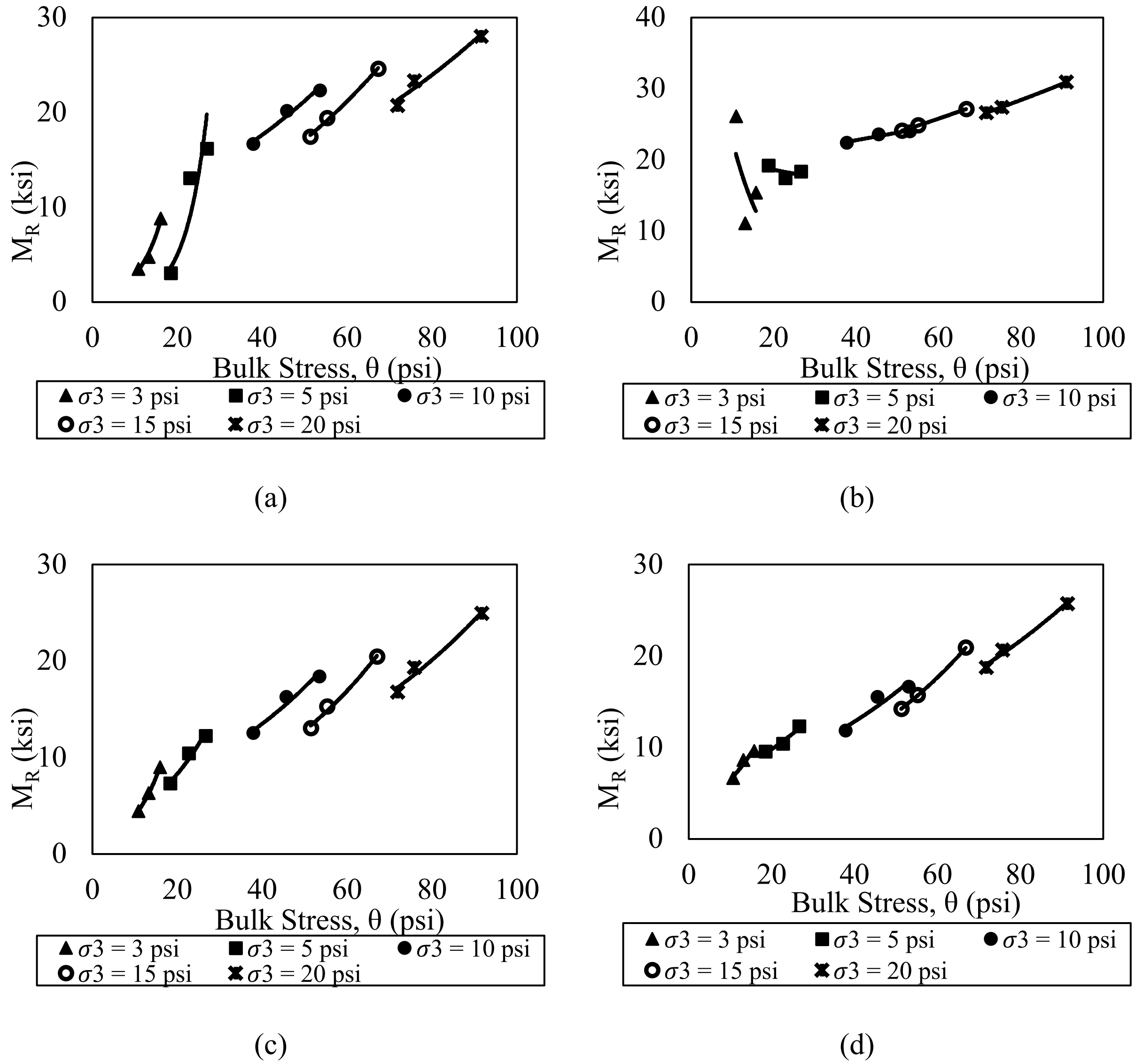
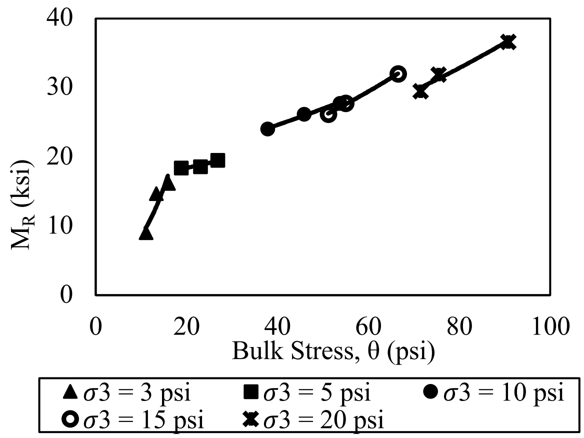
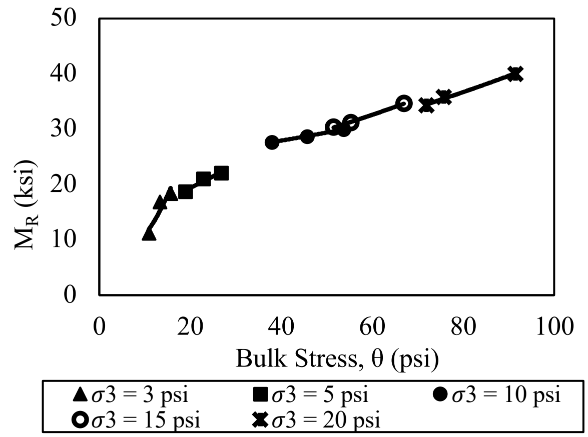


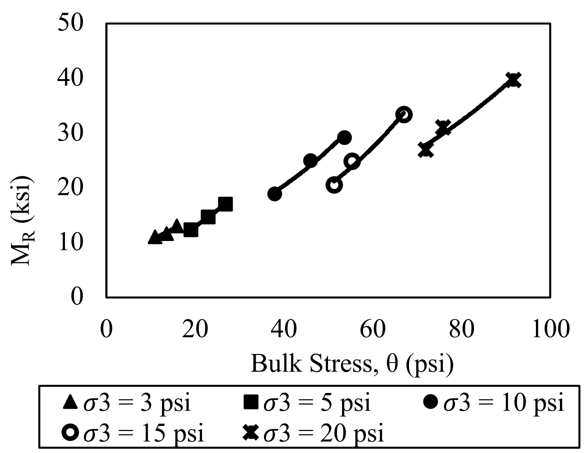
Figure B.21 Resilient modulus varying with bulk stress of Harlan blends (a) H9, (b) H10, (c) H11, and (d) H12



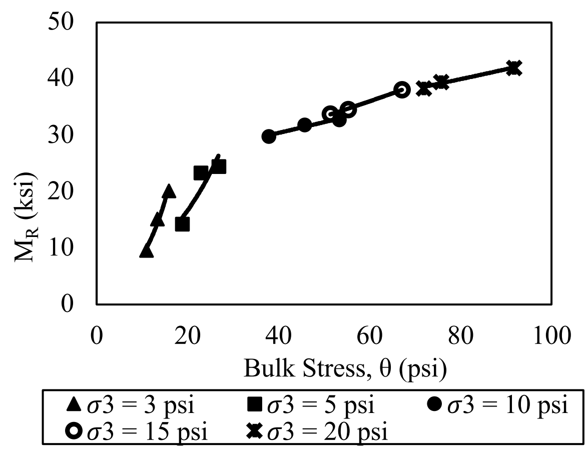
(a)



(b)



(c)



(d)

Figure B.22 Resilient modulus varying with bulk stress of Scottsbluff blends (a) S1, (b) S2, (c) S3, and (d) S4

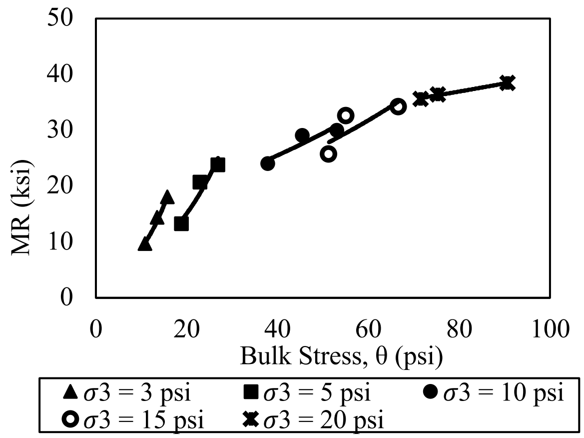


Figure B.23 Resilient modulus varying with bulk stress of Scottsbluff blend - S5

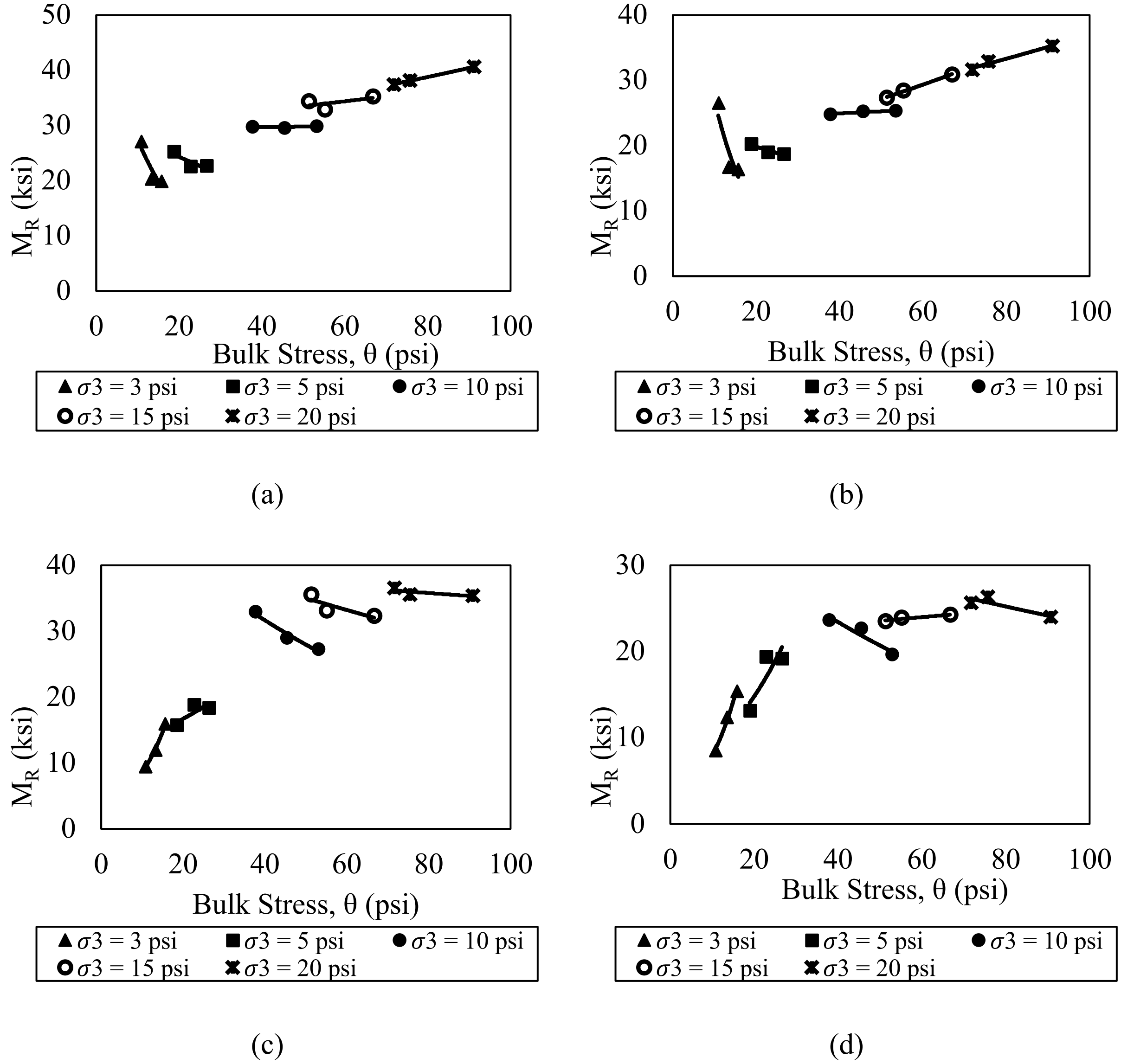


Figure B.24 Resilient modulus varying with bulk stress of Cherry blends (a) C1, (b) C2, (c) C3, and (d) C4

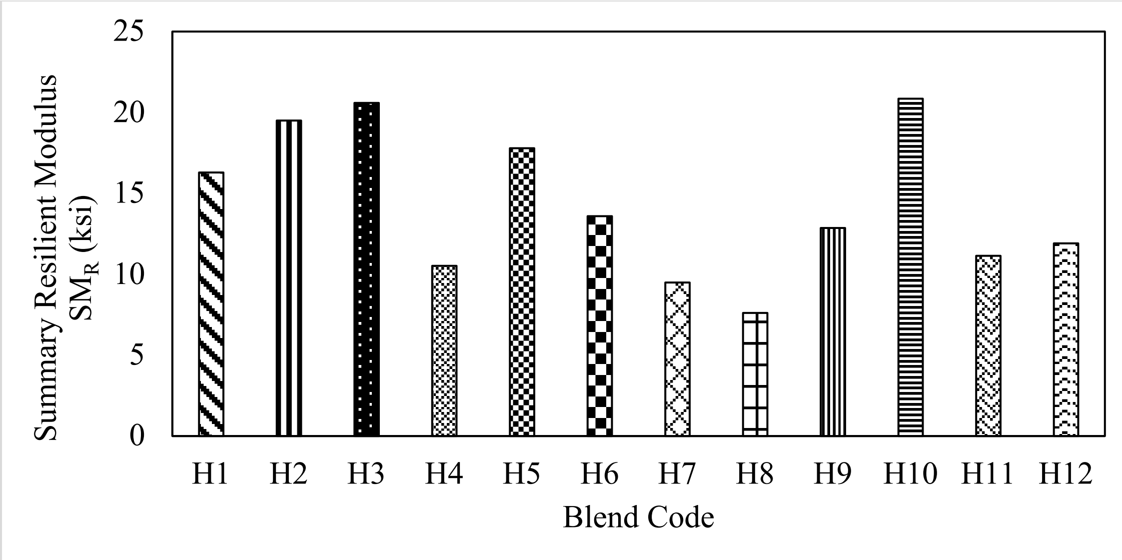


Figure B.25 Summary M_R results for all Harlan blends

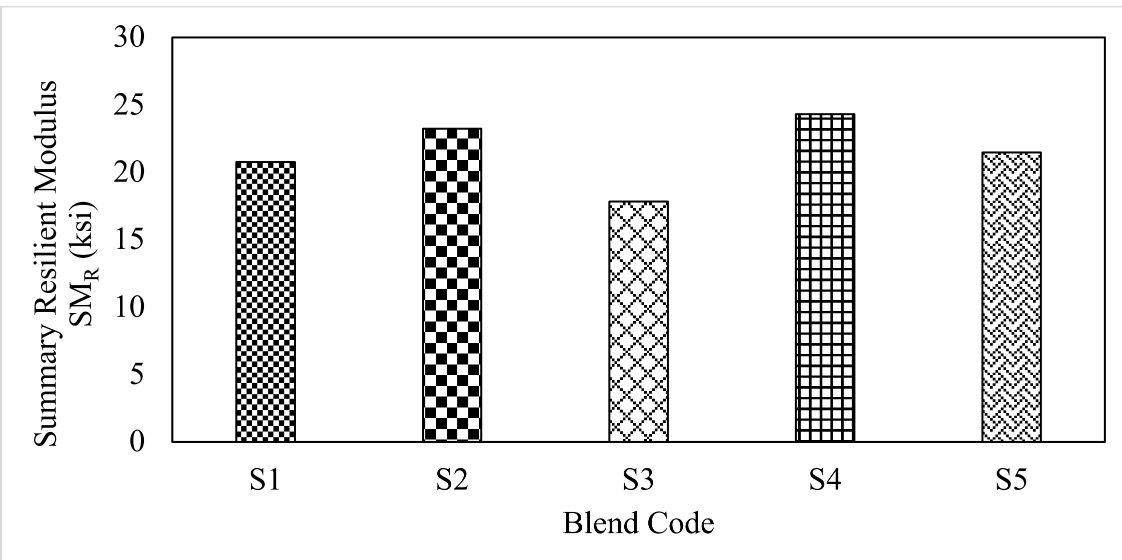


Figure B.26 Summary M_R results for all Scottsbluff blends

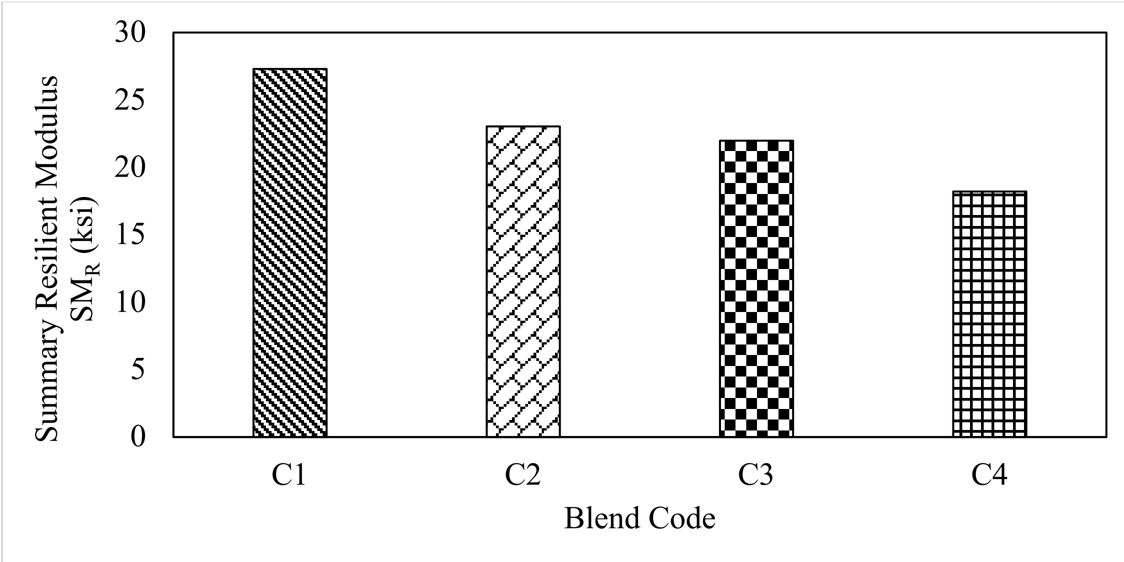


Figure B.27 Summary M_R results for all Cherry blends

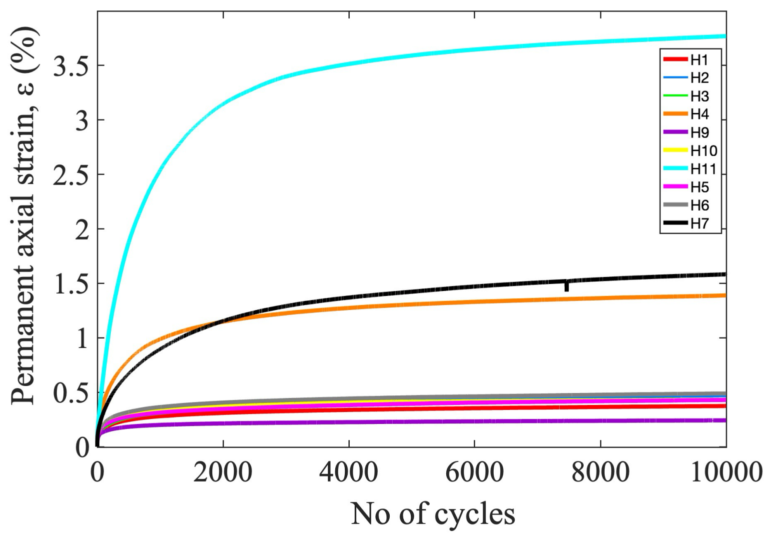


Figure B.28 Cumulative permanent axial strain versus number of load repetitions for Harlan County blends

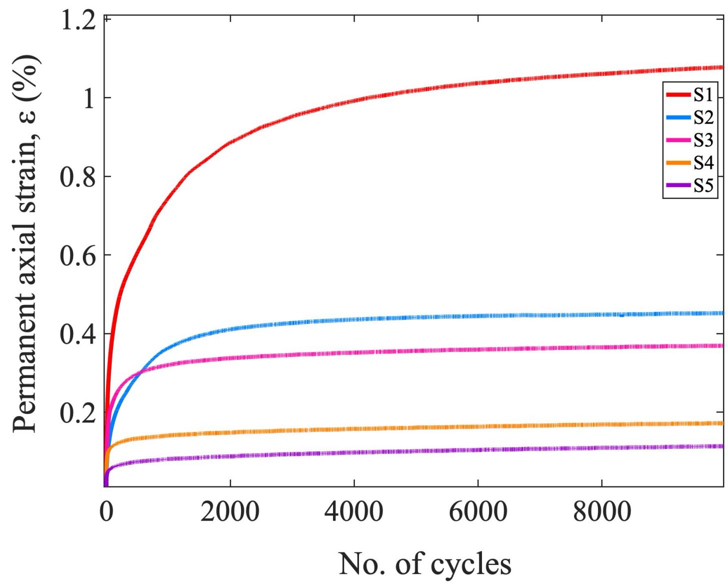


Figure B.29 Cumulative permanent axial strain versus number of load repetitions for Scottsbluff County blends

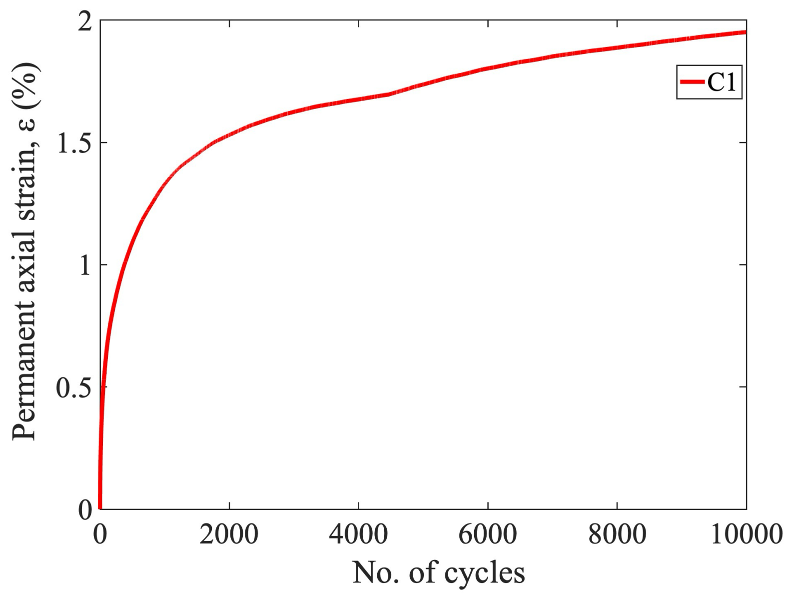
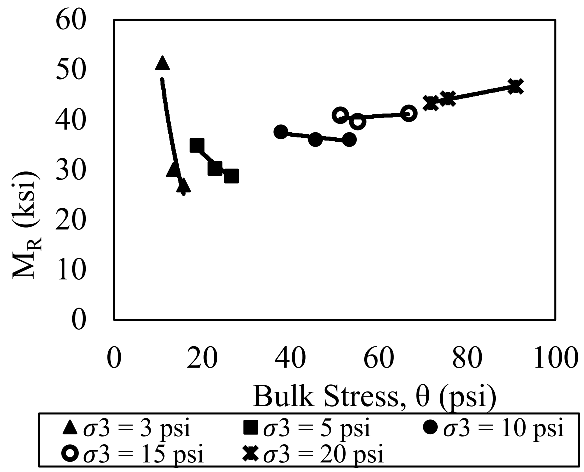
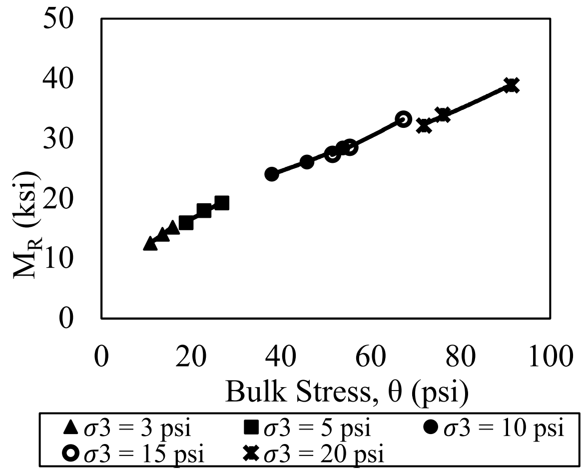


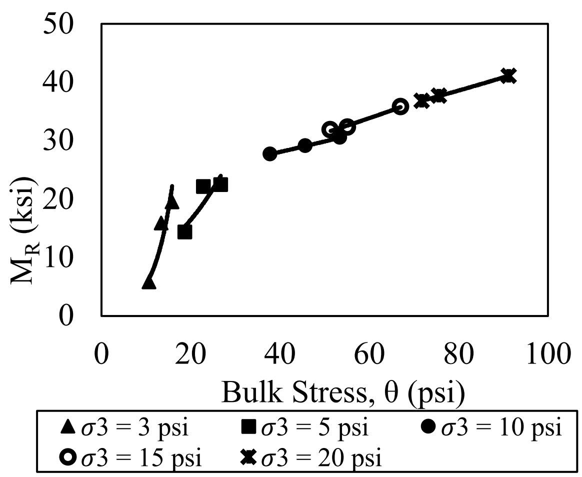
Figure B.30 Cumulative permanent axial strain versus number of load repetitions for Cherry County blends



(a)



(b)



(c)

Figure B.31 Variation of Resilient Modulus with bulk Stress for Douglas blends after 5FT Cycles: (a) D7, (b) D1, and (c) D9

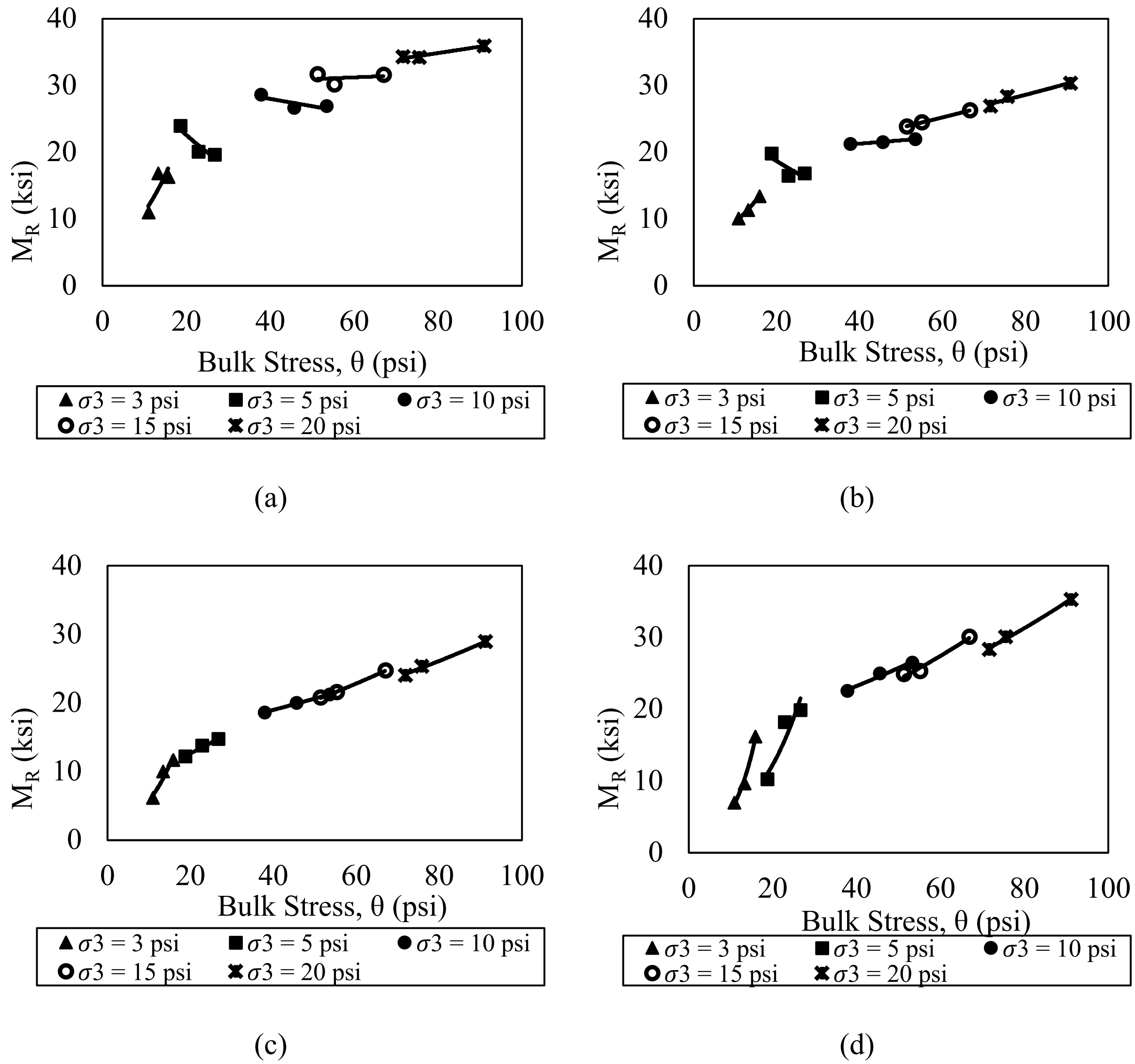


Figure B.32 Variation of Resilient Modulus with bulk Stress for Harlan blends after 5FT Cycles: (a) H10, (b) H5, (c) H1, and (d) H7

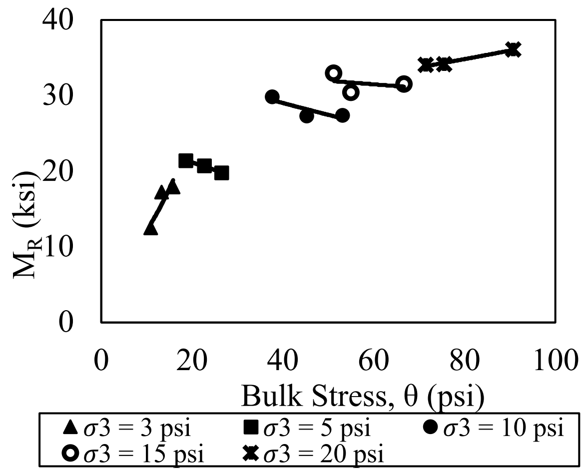


Figure B.33 Variation of Resilient Modulus with bulk Stress for Scottsbluff blend S2 after 5FT Cycles

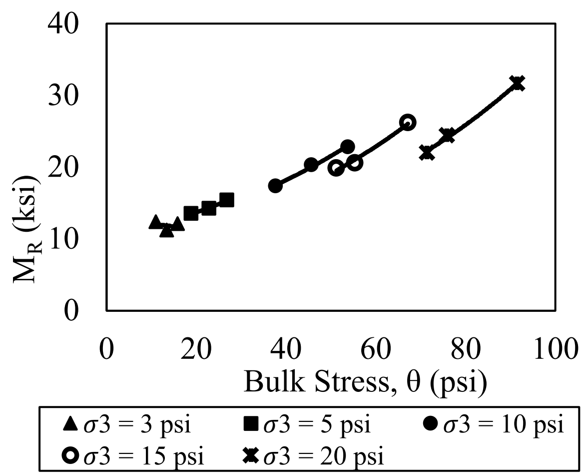


Figure B.34 Variation of Resilient Modulus with bulk Stress for Cherry blend C1 after 5FT