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Structural Design of Unpaved Roads for Solar and Wind Farms

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Abstract

Numerous countries, including Italy, have a substantial portion of their road networks consisting of unpaved roads. In Italy, these roads play a vital role in facilitating transportation for agricultural and market-related purposes, while also offering support to renewable energy farms such as wind and solar farms. In particular, for the establishment of wind and solar farms, the transportation of extremely heavy equipment and components is necessary during the construction stages, despite the low traffic volumes involved in their subsequent operation.

Consequently, it is essential to thoroughly examine the optimal construction practices and maintenance procedures, analyze the mechanisms that can lead to failures, and ensure their proper and careful design.

The primary aim of this thesis is to investigate selected specific approaches for determining the appropriate structural design of the final road thickness for unpaved roads in the context of solar and wind farms. It focuses on the subsequent areas in depth.

- ✓ Literature review on history of unpaved roads along with their prominent categories, construction & maintenance, and main distresses associated with them.
- ✓ Description of the design methodologies of unpaved roads, utilizing AASHTO, AUSTROADS, IRC, and S.A manuals, along with two empirical mechanistic approaches together with assessment and consideration of each distress in these manuals to see the impact in determining the final pavement thickness.
- ✓ The utilization of the aforementioned manuals for conducting a critical analysis of previous case studies related to wind and solar farms, with a particular focus on contrasting and comparing their approaches, aiming to identify any variations that may occur.
- ✓ Performing a sensitivity analysis on key variables to evaluate their significance and level of impact on the outcome.

A comprehensive examination and practical implementation of all the mentioned elements were carried out by employing past case studies that specifically revolved around wind and solar farms.

Keywords: Unpaved roads, Dirt Roads, Gravel roads, Laterite Roads, Ravelling, Corrugation (Washboarding), Rutting, Potholes, Dust, Unpaved Road, Structural Analysis, Sensitivity Analysis, Wind Farms, Solar Farms

Contents

1. Introduction 12
1.1. Problem statement 14
1.2. Scope and Objectives 16
1.3. Thesis Organization 16
2. Literature Review 17
2.1. History of Unpaved roads 17
2.2. Definition Low volume Unpaved roads 18
2.3. Major types of unpaved roads 19
2.3.1. Dirt Roads 19
2.3.2. Gravel Roads 20
2.3.3. Laterite Roads (Murram Roads) 23
2.4. Typical Cross section of Unpaved gravel roads 26
2.4.1. Crown 27
2.4.2. Shoulder 30
2.4.3. Ditches 32
2.5. Surface and base materials for gravel Road 33
2.6. Distresses in Unpaved roads and their Maintenance 36
2.6.1. Dust 37
2.6.2. Ravelling 40
2.6.3. Corrugation (washboarding) 42
2.6.4. Potholes 47
2.6.5. Rutting 50
3. Maintenance and rehabilitation of Unpaved Roads 55
3.1. Reshaping Surface and Shoulder 56
3.2. Reshaping Entire Cross Section 57
4. Structural Design of Unpaved roads using Different Manuals 58
4.1. Pavement Design 58
4.1.1. Australia (AUSTROAD) design Method 59
4.1.2. American Association of State Highway and Transportation Officials (AASHTO) 70

4.1.3. India (IRC) ... 77
4.1.4. South African Design Manual (Draft TRH 20) ... 87
4.1.5. Mechanistic-Empirical approach ... 91
5. Distress Impact on Design: Critical Analysis ... 96
5.1. Dust ... 96
5.2. Ravelling ... 97
5.3. Corrugation ... 97
5.4. Potholes ... 98
5.5. Rutting ... 98
6. Pavement design for wind and solar farm case studies ... 99
6.1. Case Study 1 ... 99
6.1.1. Calculation of number of Equivalent Axes (ESALS) ... 101
6.1.2. Pavement Design ... 104
6.1.3. Sensitivity Analysis ... 106
6.1.4. Comparative Analysis of Design Manuals with Reference to AASHTO ... 110
6.1.5. Comparative Analysis of Design Manuals with Mechanistic-Empirical approaches 111
6.2. Case Study 2 ... 113
6.2.1. Pavement Design ... 114
6.2.2. Comparative Analysis of Design Manuals with Reference to AASHTO ... 115
6.2.3. Comparative Analysis of Design Manuals with Mechanistic-Empirical approaches 116
7. Conclusion and recommendations ... 119
7.1. Conclusions ... 119
7.2. Recommendations ... 121
Bibliography ... 123
Appendix A ... 127
Appendix B ... 134

List of Figures

Figure 1. Wind turbines and solar panels (DGB Group, 2023) 14
Figure 2. Transportation of a blade of a wind turbine (photo credit: Utility dive, 2022) 15
Figure 3. Typical roads of each design environment (Robert A. Douglas, 2016) 19
Figure 4. Dirt road (photo credit.www.rougton.com, 2014) 20
Figure 5. Example of a gravel road in a good shape (FHA, 2015) 21
Figure 6. Example of a gravel road located in wet area (FHA, 2015) 22
Figure 7. Example of a gravel road in a desert area (FHA, 2015) 22
Figure 8. A murram road in Kenya (Photo credit: A. Anyango) 26
Figure 9. Typical Cross-section of a gravel road (TBR, 2021) 27
Figure 10. Different schemes of crown. (PennState, 2019) 28
Figure 11. Centerline crown (PennState, 2019) 29
Figure 12. Out-sloping crown (PennState, 2019) 29
Figure 13. Good example of gravel shoulder (FHA, 2015) 31
Figure 14. Nicely reshaped ditch (FHA, 2015) 33
Figure 15. A very good surface gravel. (FHA, 2015) 35
Figure 16. A road that is distressed with dust. (Légère, 2015) 38
Figure 17. A truck applying liquid chloride on unpaved road (FHA, 2000) 39
Figure 18. Mobile and static dust monitoring apparatus (Rushing 2006) 39
Figure 19. A road distressed with ravelling (Légère, 2015) 41
Figure 20. Measurement of loose material on pavement surface (Austroads, 2009) 41
Figure 21. The "Forced oscillation theory" for the formation of corrugation. (After Heath and Robinson, 1980) 43
Figure 22 Corrugation formation in dry climates (OEDC 1987) 44
Figure 23. Cutting corrugations (Ferry 1986) 45
Figure 24. Corrugation formations in wet climates (OECD 1987) 46
Figure 25. Corrugations in gravel surface. (ARRB, 2020) 46
Figure 26. Low-cost drags 47
Figure 27. Potholes on flat crossfall (ARRB, 2020) 49
Figure 28.Rutting in Wheel paths(Source: ARRB Group) 50
Figure 29 Deformation within granular layers of the pavement, near the surface (A. Dawson & P Kolisoja, 2006) 52

Figure 30. Close up caption of a rut in Scottish forest (Photo: courtesy W Tyrrell) (A. Dawson & P Kolisoja, 2006) 52

Figure 31. Shear deformation within the subgrade (A. Dawson & P Kolisoja, 2006) 53

Figure 32. Advanced case of Structural rutting with weak subgrade (photo: courtesy W Tyrrell) 54

Figure 33. Reestablishing of shoulder line (Photo credit NACE/APWA 2015) 56

Figure 34. A photo of a front dozer equipped with carbide bits maintaining areas affected by washboards. (FHA, 2000) 57

Figure 35. Preferred layers associated with unsealed road (ARRB, 2020) 63

Figure 36. Design chart for granular pavements (80% confidence) (ARRB, 2020) 65

Figure 37. Suggested PSD range for unsealed wearing course (Source based on NAASRA, 1980) 68

Figure 38. Workability attributes of granular materials. Source: Wooltorton (1947) 69

Figure 39. Design chart for aggregate-Surfaced Roads Considering Allowable Serviceability Loss (AASHTO, 1993) 72

Figure 40. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting (AASHTO, 1993) 73

Figure 41. Example Growth of Total Damage Versus Base Layer Thickness for Both Serviceability and Rutting Criteria. (AASHTO, 1993) 74

Figure 42. Chart to convert a Portion of the aggregate Base Layer Thickness to an Equivalent Thickness of Subbase (AASHTO, 1993) 75

Figure 43. The six Climatic Regions in the United States (AASHTO, 1993) 76

Figure 44. Typical Cross-Section of a Gravel Road Pavement 78

Figure 45. Chart to convert a portion of the Gravel/Soil- Aggregate Base Layer Thickness to Equivalent of Subbase Adapted from AASHTO, 1993) 84

Figure 46. Weinert N-values for southern Africa (Weinert, 1980) 90

List of Tables

Table 1. Indicative design traffic volumes for various road classes(based on 10 year design life) 62
Table 2. Suggested CBR values for pavement materials for unsealed roads 66
Table 3.Indicative permeability values (100% standard compaction) 67
Table 4. Typical properties for unsealed road wearing course (source NAASRA, 1980) 67
Table 5. Chart for Computing Total Pavement Damage (both serviceability and based on trial Aggregate Base Thickness (AASHTO, 1993) 70
Table 6. Aggregate Surfaced Road Design Catalog: Recommended Aggregate Base Thickness in inches for the Six U.S. Climatic Regions, Five Relative Qualities of Roadbed Soil and Three Levels of Traffic (AASHTO, 1993) 77
Table 7. Reasonable estimate of design traffic in terms of cumulative ESAL (IRC, 2008) 81
Table 8. Traffic Categories 81
Table 9. Typical Presumptive Design CBR Values 82
Table 10. Subgrade strength classes 82
Table 11. Pavement Design Catalog (IRC, 2008) 83
Table 12. Grading Requirements for Base Course (IRC, 2008) 85
Table 13. Grading Requirements for Surface Course (IRC, 2008) 85
Table 14. Percentage of Gravel, sand and fines (Silt and Clay) in Gradings A, B, C (IRC, 2008) 86
Table 15. Requirements when the requirements with respect to Table 12 grading is not met 86
Table 16. Recommended material specifications for unpaved rural roads 91
Table 17. Serviceability Index and Rut Depth**)** (Margot T, n.d) 95
Table 18. Summary of the LEF for the different manuals utilized in the case study 103
Table 19. Sensitivity Analysis summary for Traffic and sub-grade strength 107
Table 20. Sensitivity Analysis Summary on other parameters 109
Table 21. Summary of Comparative Analysis -Case Study 1 110
Table 22. Comparative Analysis between the Different Manuals and the SDMS Method 112
Table 23. Comparative Analysis between the different manuals and the ARMS method 113
Table 24.Summary of the results of design thickness of case-study 2 for each manual 115
Table 25. Summary of Comparative Analysis -Case Study 2 115

List of Glossary of Acronyms

AASHTO: American Association of Highway and Transportation Officials

IRC: Indian Road Congress

CBR: California Bearing Ratio

M_r: Resilient Modulus

AUSTROADS: Association of Australian and New Zealand Road Transport and Traffic Authorities

SA: South Africa

ESAL: Equivalent single axle load.

ARRB: Australian Road Research Board

LEF: Load equivalent Factor

psi: pounds per square inch

lb : pounds

DESA: Design number of Equivalent Standard Axles

TRH: Technical Recommendations for Highways

FHA: Federal Highway Administration

HV: Heavy vehicle

PSI: Present serviceability Index

SDMS: Surfacing Designing and Management System

USFS: United States Forest Service

ARMS: Analysis Road Materials System

Chapter 1

1. Introduction

Renewable energy has a long history in ancient societies, who used the sun, wind, and water's power for a variety of purposes. Examples include the use of water wheels by the ancient Chinese and Persians to grind grain as well as the use of wind-powered water pumps by the ancient Greeks and Romans for irrigation. (DGB Group, 2023) Each of these methods functions in its own distinct way, whether it involves capturing sunlight through solar panels or utilizing wind turbines and water flow to generate power. The Industrial Revolution and the extensive use of fossil fuels during the 18th and 19th centuries, however, caused a drop in the usage of renewable energy.

Throughout the 20th century, there was a renaissance of interest in renewable energy as the environmental movement gathered steam and concerns about the unfavorable impact of fossil fuels on the environment grew. The 1970s oil crisis served to further emphasize the need for energy independence and the investigation of alternate energy sources. (DGB Group, 2023) As a result, governments all over the world started to spend money on developing and researching new renewable energy technology.

For instance, solar energy grew significantly over the 20th century. Solar panels are becoming more widely available and effective because of technological advancements and price reductions. This, together with a growing understanding of environmental issues, pushed authorities, organizations, and people to adopt solar energy as a competitive alternative to conventional fossil fuels.

In a comparable manner, during this time wind energy became a significant renewable energy source. The ability to harness wind power on a wider scale was made possible by the development of bigger, more effective wind turbines. As wind farms spread across the country, the need for fossil fuel-based power generation decreased while producing clean electricity.

Another proven renewable energy source, hydroelectricity, saw steady growth in the 20th century. The development of turbine technology and the building of dams made hydroelectric power a dependable and sustainable method of producing electricity. Worldwide, large-scale hydroelectric projects have been put in place, taking advantage of rivers' potential for energy production and adding to the diversification of energy sources. Furthermore, biomass—a renewable energy source with enormous potential—is made from organic materials such as agricultural waste, wood, and crops grown specifically for energy. The use of biomass for heating and electricity generation has gained popularity, providing an alternative to combustion based on fossil fuels and lowering greenhouse gas emissions.

Sustainable development, environmental sustainability, energy security, economic opportunities, and long-term cost savings are among the primary benefits and applications of renewable energy. Renewable energy sources, in contrast to fossil fuels like natural gas and coal, are typically naturally replenished and do not require millions of years for formation. Moreover, these green sources frequently circumvent mining or drilling activities that can have detrimental effects on ecosystems. (TWI-GLOBAL, 2023) Although this source of energy has been used for a long time, it has recently come back into favor as a result of growing worries about climate change and the significance of obtaining energy independence.

Currently, a variety of technologies are available in the well-established and growing field of renewable energy. According to projections, their share of the power mix will increase by 10 percentage points to 38% by 2027. It should be highlighted, nevertheless, that fossil fuels continue to be the main source of energy. Together, coal, oil, and natural gas made up over 78% of the world's primary energy consumption in 2019. (IEA, n.d.)

Given that so many countries and businesses are actively investing in the research and development of cutting-edge technology while pledging to use renewable energy sources, the future of renewable energy appears bright. By 2027, the International Energy Agency (IEA) predicts that the consumption of renewable energy for heating would have increased by about one third. (IEA, n.d.) By 2027, it is anticipated that this advancement would increase current renewable energy use in heating from 11% to 14%. As an illustrative instance, Figure 1 depicts a picture of a common type of wind turbine and a solar panel.

Figure 1. Wind turbines and solar panels (DGB Group, 2023)

1.1. Problem statement

With the development of wind and solar farms across the globe, renewable energy is being embraced in an effort to responsibly meet mankind's energy needs. But for a variety of reasons, these projects pose major difficulties and need a lot of labor. The main challenge is the enormous size and complexity of the jobs involved requiring the transportation of heavy-duty machines used for installation and operation. Contractors largely rely on the dependability of their equipment to assure safety and accuracy because wind turbines, solar panels, and the specialized machinery needed for their installation are demanding construction projects. (YAK MAT, n.d.) Furthermore, getting to and from the project areas is difficult due to the remoteness of these places.

Constructions involving solar and wind farms differ from other typical manufacturing operations in that they frequently call for the delivery and transportation of very large, heavy components that are unable to be manufactured and produced on-site, such as transformers, wind turbine blades, tower parts, etc., and that are too large and heavy to be transported using conventional methods because they surpass their size and weight restrictions. Figure 2 illustrates the size of a single turbine blade. Furthermore, every significant structural project is also connected to a variety of pollutants, not the least from traffic on the work site. For instance, it may take up to 13 heavy load trips to deliver a wind turbine made of steel tubes. Additionally, the number is substantially higher for hybrid tower substitutes. These are multiplied by the numerous additional truck movements needed for the foundations and the earthworks. (Wpd, 2020)

Additionally, the idea of solar roads calls for the employment of solar panels that have been specifically made to endure automotive activity. The intention is to use existing asphalt roads less frequently. (Kohak P.G et al, 2019) These panels have the capacity to collect solar energy, which houses and businesses can use. The reliance on fossil fuels for the creation of power can be lessened by the use of solar road panels, which subsequently results in a reduction in greenhouse gas emissions. This invention not only makes it possible to build an intelligent roadway infrastructure, but it also creates a decentralized, self-healing power grid, obviating the need for fossil fuels.

Figure 2. Transportation of a blade of a wind turbine (photo credit: Utility dive, 2022)

1.2. Scope and Objectives

This thesis's main goal is to examine the methods for the structural design of the final road thickness of unpaved roads for solar and wind farms utilizing a number manuals that have been chosen. The following is the list of these manuals.

- AASHTO: American Association of State Highway and Transportation Officials
- AUSTROADS: Association of Australian and New Zealand Road Transport and Traffic Authorities
- IRC: Indian Road Congress
- TRH 20: Technical Recommendations for Highways: The Structural Design, Construction, and Maintenance of Unpaved Roads (South Africa).

Furthermore, the inclusion of mechanistic-empirical approaches was part of the comprehensive strategy employed in previous case studies of wind and solar farms for the design of unpaved roads.

It also points out the impact and contribution that each specific parameter for each manual has on the end result of the design. Additionally, it contains sensitivity analyses on the variables that are thought to be significant in determining the ultimate design thickness.

1.3. Thesis Organization

The thesis's general organization is outlined in the bullet points below.

- The history of unpaved roads is covered in Chapter 2 in more detail, including its description, main types, typical cross-section, materials used in construction, and related distresses.
- The focus of chapter three is on repairing and maintaining unpaved roads.
- The fourth chapter discusses the structural design of unpaved roads using various manuals.
- Distress Impact on Design: Critical Analysis is covered in Chapter 5.
- Chapter 6 covers the various calculations related to pavement design, including sensitivity analysis and comparative analysis, with a specific focus on past case studies.

Chapter 2

2. Literature Review

2.1. History of Unpaved roads

Across many countries, unpaved roads (roads with no surfacing like bituminous or cement concrete) comprise a considerable proportion of the total road length. Today, in many places in the world, the unpaved road length is a lot more compared to the paved road length. (IRC, 2008). The majority of the roads in rural areas consist of low-volume roads. A road that is in a good location, well planned and designed, well-constructed & maintained is crucial for the development of the community and smooth flow of goods among the different communities. It also plays an important role in resource management activities. (G. Keller, 2003).

The ancient and contemporary worlds have added clay to sandy roads to give them solidity, or sand and gravel to clay surfaces to protect them from rutting and becoming sticky or inaccessible in rainy weather. Both of these methods were inspired by nature. The sand and gravel particles bear on one other and resist traffic loads while the clay serves as a binder. (A. T Visser & W R Hudson, 1983)

Low-volume roads frequently have progressed, and advanced and engineering & scientific methods were just an addendum. Whenever engineers and designer are engaged and participating in low-volume roads construction they use the best available information. In addition, they also expand their experience and advanced technologies from higher-standard roads to the low-volume road cases. Committees and teams of working groups were established to bridge the technology gap to interchange practices between the two extremes. (Coghlan, n.d)

As a renewable answer to our energy needs, wind and solar farms are sprouting up in numerous nations. The amount and complexity of the job that has to be done comes first. Contractors must rely on the stability and safety of the access roads, which are often low-volume roadways to reach the site and guarantee their equipment is delivered in the right manner due to the particular nature of windmills and solar panels and the machines required to construct them.

The bituminous and Portland cement concrete surfacing of the paved road networks have significantly increased in recent decades. However, unpaved roads—those with an earth, sand-clay, or gravel surface—make up a large portion of the network in most nations. (A. T Visser & W R Hudson, 1983)

This thesis primarily discusses various structural design methodologies and design considerations for potential distresses and damages these types of roads may experience over the course of their design lives.

2.2. Definition Low volume Unpaved roads

A low-volume road is one that has low design speeds (usually less than 80 km/h) and relatively low utilization (average daily traffic of less than 400 cars per day). It is a transportation system that is primarily designed to gather or manage resources from rural and remote places. (G. Keller, 2003) Unpaved roads play a crucial role in the socioeconomic growth of many nations as access to remote regions and the exploitation of natural resources both largely depend on transportation systems. (J.-P. Bilodeau, 2017)

Additionally, they provide a network of linkages for raw materials from mines and forests to mills, as well as from houses and farms to marketplaces. Access to fundamental facilities including healthcare, education, civic services, and outdoor recreation areas is also increased by low-volume roadways. They also serve as the main connection between remote locations and the highway transportation network. It's vital to remember that these unique systems are made to handle axle loads that could be extremely high yet modest traffic volumes. (G. Keller, 2003)

Local drivers are accustomed to recognizing and anticipating design imperfections, therefore low volume roads also handle local traffic that uses the same portion of the road. Roadways with very low traffic numbers may have fairly strict design requirements than those with larger traffic levels. (AASHTO, 2001)

Low-traffic highways may be flexible two-lane asphalt roadways that may handle up to 2,000 vehicles per day. The top limit is set at 400 vehicles per day, according to a broadly recognized classification of low-volume highways. Some people also distinguish between farm-to-market rural low-volume routes and urban low-volume roads. (Coghlan, n.d).

Unpaved roads are ones that have not been bound with binders like asphalt or oil and are intended to last as unsealed or exposed surfaces made of natural resources or crushed aggregate. (LawInsider, n.d.)

Based on the function and purpose it serves a low-volume road may be paved or unpaved. Figure 3 summarizes the classification of roads as a function of traffic volume and weight of typical vehicle.

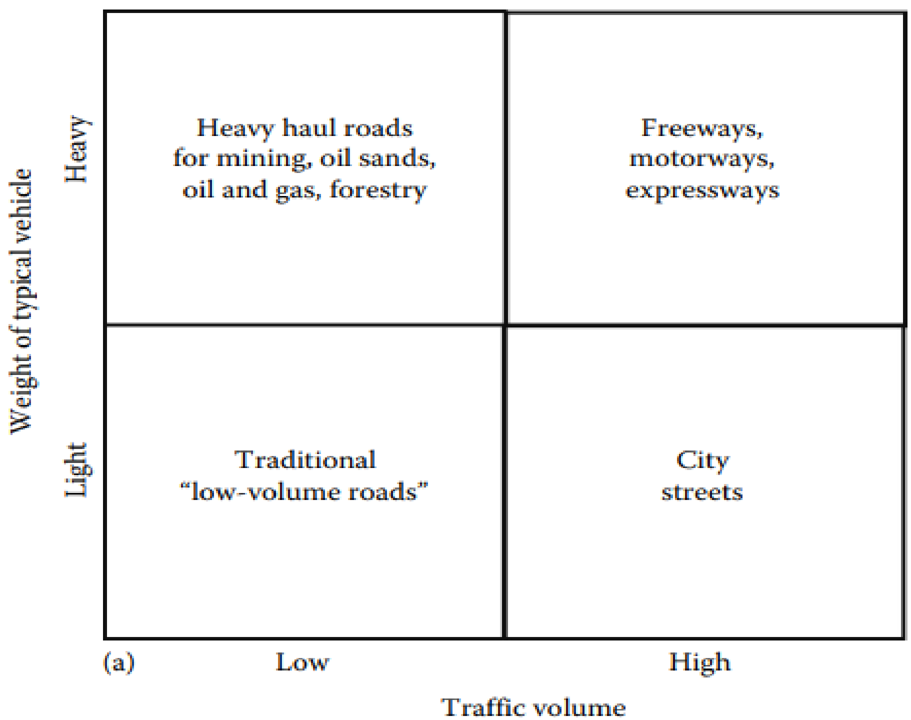

Figure 3. Typical roads of each design environment (Robert A. Douglas, 2016)

2.3. Major types of unpaved roads

Unpaved roads are divided into many types based on the type of surfacing material used, with dirt (track), gravel, and laterite (Murram) roads being a few examples. (Sultana & Weber, 2016)

2.3.1. Dirt Roads

One sort of unpaved road is a dirt road or track, which is constructed using the subgrade and other locally sourced natural materials. They are practical and suited for light vehicles, animals, and humans. (Dr. Netterberg, 2014).

Most often, these roadways are not graded adequately to provide the camber needed for rainfall to extract itself from the surface and drain away, and side ditches might not even be present. As a result, following rain, this may result in waterlogging and/or erosion, making it difficult for vehicles to pass. These routes are also known as dry-weather roads in nations like Finland, Australia, and New Zealand. These roads can also be described as earth roads. (Sultana & Weber, 2016)

It is obvious that dirt roads take the distinctive and unique qualities of the local soils and geology. They could have a bare soil surface, which could be exceedingly muddy and slick when in touch with water, and exceptionally hard in dry conditions. They could also be of a sandy, stony, or rocky character. They are common and well-liked in rural areas of several nations, where they are also periodically employed. (Sultana & Weber, 2016) A typical dirt road looks like as shown in Figure 4 .

Figure 4. Dirt road (photo credit.www.rougton.com, 2014)

2.3.2. Gravel Roads

Unbound granular roadways with a running surface made of small rocks, gravel, or other acceptable and suitable aggregate road surfacing material are known as gravel roads. They may also contain a dust suppressant. These types of unpaved roads are crucial for a number of reasons, including cost, the necessity for a good road network, and traffic volume. (ODOT Manual, 2011).

Gravel roads typically provide the least amount of service to vehicles. However, the level of traffic in many isolated and rural regions is so low that surfacing and maintaining a road is not financially viable. Gravel roads are frequently used as a means of transporting agricultural products in and out of farmlands, removing wood from forests, or gaining access to remote locations like camping grounds, streams, and lakes. (Engineer, 2021)

A gravel road's state and quality can be affected by a variety of elements, such as the gravel's quality, the cross section of the road, drainage, the expertise of the motor grader operator, and the organization of the maintenance and improvement plan. When compared to paved roads, gravel roads may require more frequent and continual grading and dust control procedures, requiring more maintenance. (LTAP Webinar, 2018).

A typical well maintained gravel road is shown in Figure 5. The figure shows a gravel road with a good cross-sectional shape throughout and enough crown that descends directly to the edge of the shoulder.

Figure 5. Example of a gravel road in a good shape (FHA, 2015)

Figure 6 illustrates a road that is located in an area with more than 60 inches of precipitation on average per year. The road functions well because it has an adequate crown on the surface and ditches along the edge of the road to drain the water. (FHA, 2015)

Figure 6. Example of a gravel road located in wet area (FHA, 2015)

In contrast Figure 7 demonstrates a gravel road that is in a desert area, in comparison. It demonstrates that despite receiving less than 10 inches of precipitation on average annually, the area is clearly in terrible condition. The fundamental problem in this situation is the poor cross section, which has no crown on the surface and no ditches to collect and drain water from the surface and away from the road. (FHA, 2015)

Figure 7. Example of a gravel road in a desert area (FHA, 2015)

When compacted with enough and suitable moisture, an ideal gravel surface will form a firm crust since it is made of an even blend of stones, sand, and clay. The material must have enough moisture to be sufficiently bound together, especially on the protective surface layer, or else it will get extremely dusty, which will cause the crucial binding elements to blow away. Grading roads when there is enough moisture is the key to controlling dust on gravel roads. (FHA, 2000)

A gravel road's layer thickness and depth must be sufficient to evenly distribute and disperse the heaviest vehicle loads applied, preventing further stresses from being applied to the subgrade that will cause obvious rutting. (FHA, 2000)

Gravel roads that are well maintained are the result of a thorough maintenance and improvement strategy, sufficient funds, high-quality supplies, thorough training, and high-quality equipment. A key piece of machinery used to preserve and maintain gravel roads is the motor grader. (LRRB, 2014).

2.3.3. Laterite Roads (Murram Roads)

A heavily weathered natural substance called laterite is created when hydrated oxides of iron or aluminum are present. This concentration may take the form of a chemical precipitate or a solution. It is never the outcome of the typical fundamental processes of sedimentation, metamorphism, volcanism, or plutonism; rather, it is always the consequence of secondary physio-chemical processes. (CHARMAN, 1988)

Due to the high iron oxide content, laterite is thought to have developed in warm, humid tropical regions. Its usual color is red and its texture ranges from hard gravel to softer earth that is smudged with small stones. (CHARMAN, 1988)

The majority of laterite or lateritic soils do not adhere to or meet the conventional requirements for road layers. One of the difficulties arises when materials are required for low traffic roadways. However, historical experiences with numerous lateritic materials have demonstrated and established that they can be used successfully with criteria that go beyond conventional standards. (P. Paige-Green, 2019)

Laterites' physical and chemical characteristics vary greatly, which can occasionally make it challenging to use them in road construction. Some types are capable of self-stabilizing in vertical cuts due to their relatively high strengths. Nogami and Villibor (1991) note that until the early 1970s, when trial testing sections were carried out with tidy soils, lateritic soils with fine grains were only used as subbase (or stabilized with cement for base). In Sao Paulo State, such lateritic soils have been routinely used as bases for low to medium traffic since the 1980s. They were frequently used and mixed with gap-graded crushed stone for roads that were intended to handle more heavy traffic. (CHARMAN, 1988)

The incredible performance of several lateritized roads in Zimbabwe has been attributed in large part to the stiffness, roughness, and shape of the particles, which are more than capable of improving the quality through "self-cementation" (van der Merwe, 1971). Curing is crucial, in addition to compaction at the ideal moisture level. Normal lateritic gravel drying times range from one day for low plasticity lateritic gravel to four to five days for lateritic sands (Cocks and Hamory, 1988). Additionally, it is advised that mixing operations should be conducted on the road (with a motor grader) to minimize irregularity. Many of the non-traditional gravels require enough compaction at the right moisture content to ensure a sturdy upper surface. (Dr. Netterberg, 2014)

The in situ Resilient Modulus (M_r) values of laterites have been shown to be high (5 GPa for road samples and greater than 700 MPa for laboratory samples). (NOGAMI, 1991)

To ensure the laterite maintains its rigidity and stiffness, it is crucial to achieve a high level of compaction, even when it is exposed to moisture. This compaction process should be carried out at a moisture content that is close to the ideal level. The surfacing of the base should be appropriately contoured to facilitate drainage and be both durable and well-compacted. This process typically takes around one to two months. It is worth mentioning that the subgrade should be suitably compacted to a depth of at least 1 meter. Furthermore, it has been noted that crushing the material to a dry state, below the optimal moisture content, typically results in an insufficient surface that lacks the required strength.

The performance of laterites in service, despite great flexibility and naturally low CBR values, is one of their distinguishing features. The unusually poor permeability of unsaturated compacted laterites is one of the contributing reasons. Unsaturated permeabilities (2.5×10^{-7} to 4.5×10^{-8} cm/s) are typically 3 to 4 times lower than saturated permeabilities. (Dr. Netterberg, 2014)

(Dr. Netterberg, 2014) also states that in order to meet the specifications' threshold requirements, the proper quality management and control techniques and procedures must be in place. If the drying temperatures are not taken into consideration, problems and issues with the calculation of Atterberg limits and moisture contents after compaction may arise.

Unconventional lateritic materials can perform admirably when employed in road construction, even as base course, according to indisputable evidence. It is vital to utilize these local resources as much as possible when building unpaved roads, especially low volume highways. This necessitates the development of adequate specifications for their range and the standardization of their testing procedures. (Dr. Netterberg, 2014)

It is also important to highlight that the majority of designers view the requirement for a high degree of compaction as necessary. To achieve this, it is essential to have a well-graded aggregate and the typical Fuller type of particle size distributions that are predicted in most standards' specifications, according to (Dr. Netterberg, 2014). This, however, tends to run counter to the demand for streamlined standards and material testing. It is obvious that grading affects the layer's compatibility and surface polish as well. It is also clear that choosing a material and creating specifications are made simpler when strength in situ is used in place of the diverse range of other material needs.

Additionally, (Dr. Netterberg, 2014) proposes that the following key areas be optimized when using laterites in the construction of roads in tropical and subtropical areas:

- A standard set of test techniques that might combine several currently in use methodologies, modify these methodologies, or even come up with entirely new methodologies.
- Comparative laboratory testing of materials is required to identify the most effective test protocols and sample preparation techniques.
- Self-stabilization should be researched with the goal of causing it in the real world and being able to predict it precisely.

Figure 8 depicts a low traffic murram road connecting Lamu and Garissa in Kenya.

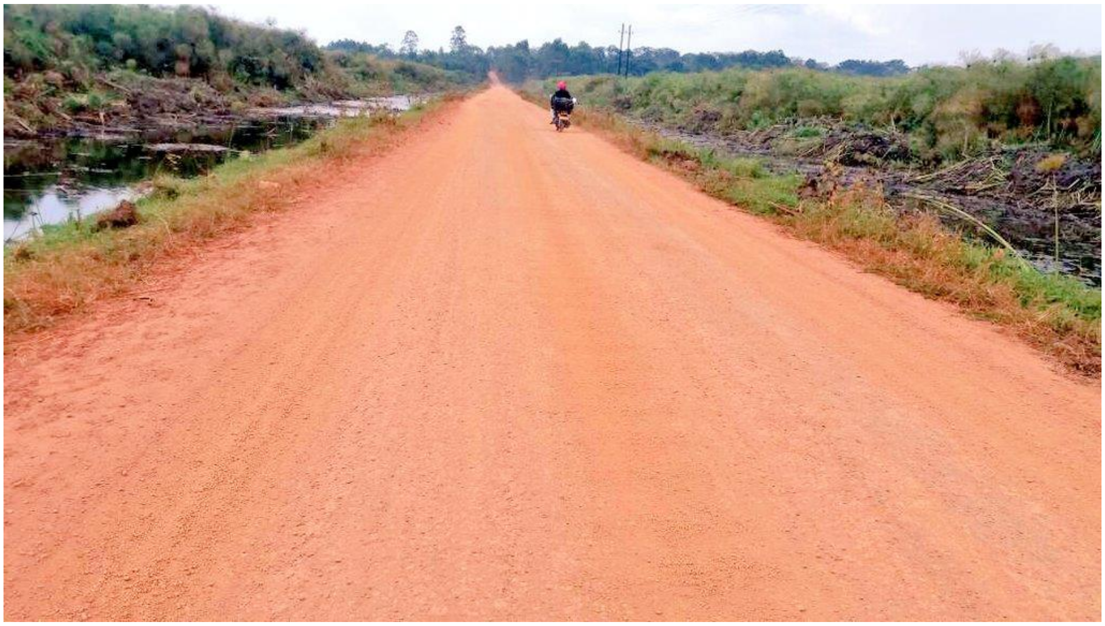

Figure 8. A murram road in Kenya (Photo credit: A. Anyango)

2.4. Typical Cross section of Unpaved gravel roads

Although the conventional design of an unpaved road varies substantially from one manual to another, it normally consists of a gravel wearing course, granular base, and granular subbase (optional) put over the native soil (AASHTO 1993). (FHA, 2015) also adds that the basic cross section and shape of a gravel road are basic and generally typical regardless of the region or terrain, and if they are not designed and built properly in accordance with standards, they will not function well even with very little traffic.

Everyone involved in gravel road maintenance, according to (FHA, 2015), must understand the exact shape of the entire area within the road's right-of-way, the total amount of land acquired for the construction of the route.

Figure 9. shows a typical cross section of a gravel road.

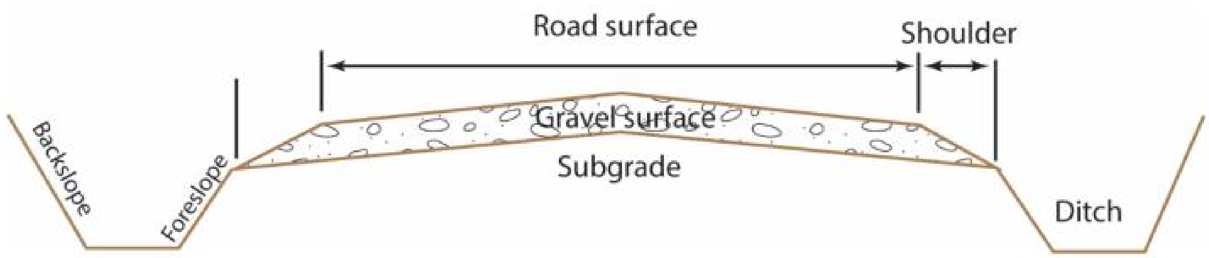

Figure 9. Typical Cross-section of a gravel road (TBR, 2021)

Three essential components make up a standard gravel road: a crowned driving surface, a shoulder, and a ditch. The margin between the gravel road's shoulder and ditch is rather little, as is shown from Figure 9. This is especially true in areas where rights-of-way are constrained or extremely limited. (TBR, 2021)

2.4.1. Crown

The crown is the area of the roadway's shape where the center of the surface is higher than its outer edges to allow water to drain from that area to curbs or ditches. (FHA, 2015) This cross slope, which is typically stated as a percentage, is 4%; to produce the roadway crown for drainage, each side slopes in a different direction from the center of the road.

In order to meet various topography conditions and traffic patterns, surface drainage is utilized to redirect water away from the road in thin, non-erosive sheets that flow in the chosen direction. (PennState, 2019) asserts that when standing water is allowed to infiltrate the road through retention in puddles or potholes, the road's surface and base erode. Damage and material loss are the outcomes of erosion, which is caused by running water that is allowed to concentrate on the road, such as in car tracks.

The two main types of road surface templets are centerline crown and in- and out-slope road surfaces, according to (PennState, 2019). As a watershed, a highpoint close to the middle of the road's centerline distributes water to both sides of the route. A great illustration is Figure 10. The situation is seen in Figure 11 with the water flowing toward both sides of the road.

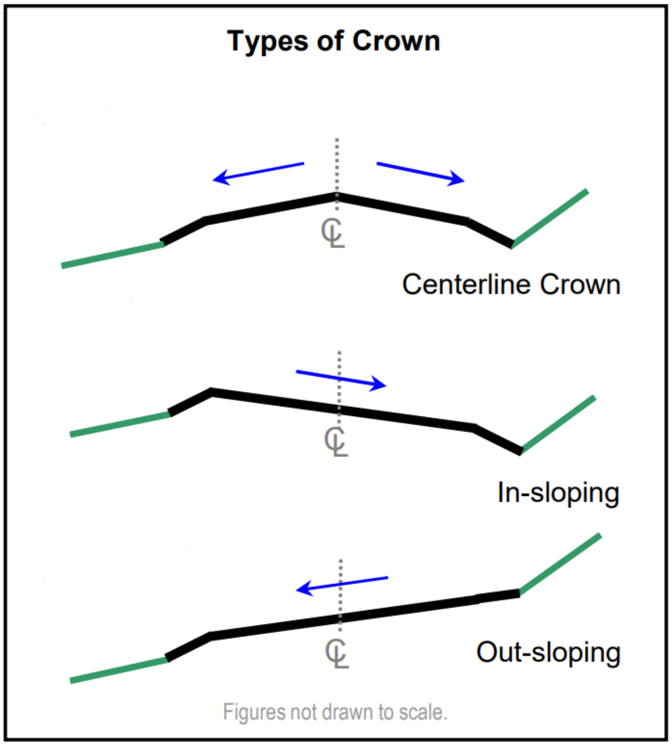

Figure 10. Different schemes of crown. (PennState, 2019)

Additionally, Figure 10 shows an in-sloping crown, a design that channels water away from the cut-bank or up-slope side as well as the entire width of the road, usually applied to slope side hills for safety. A sort of in-sloping known as super-elevation of curves (banked curves) drains the road surface while simultaneously supporting traffic. Figure 10 illustrates a different layout with an outward sloping crown surface. Out sloping road surfaces divert water throughout their entire width away from the fill bank or downslope side. Road ditches on either side of the road can be eliminated with this surface configuration.

Figure 11. Centerline crown (PennState, 2019)

This layout closely resembles natural drainage patterns and permits just a small amount of overland sheet flow to cross the roadway Figure 12. On low traffic roadways, out-sloping is advantageous when side slopes are mild and there is little chance of winter icing. (PennState, 2019)

Figure 12. Out-sloping crown (PennState, 2019)

Unpaved roads obviously need a more robust crown than paved surfaces do because they are unsealed. Pavement scatters water more quickly than an unpaved surface and prevents water intrusion. The cross-slope of a paved road is typically 2%, according to (FHA, 2015). A common problem is making dirt or gravel road appear to be paved. An unpaved road needs a side slope that is two to three times as steep to prevent erosion and displacement. On unpaved surfaces, a cross-slope of between 4% and 6% is appropriate. Due to the steeper cross-slope, less water has a chance to concentrate and wash the road surface or to infiltrate and undermine the road base. With longer intervals between maintenance grading operations and less loss of purchased road material, the road becomes smoother.

Since external factors like traffic and mother nature will wash out and drive out the crown, grading involves periodic and continuing maintenance. It makes sense that some roads will require grading more regularly than others given the variation of topography and soil types employed in each road's construction. (PennState, 2019)

2.4.2. Shoulder

As seen in Figure 9, the shoulder is the area that slopes away from the driving surface's edge immediately. The following duties that a road's shoulders must fulfill are listed in (ERA Part D, 2011)

- Without adequate shoulders, the road will slant and deteriorate. In order to keep the running highway in place, it acts as edge support.
- Allowing broad vehicles to pass one another without harming the shoulder.
- Provide a safe area for temporarily stopped or damaged vehicles.
- Make it possible for vulnerable road users, including pedestrians, bikers, and bicyclists, to travel safely.
- Permit water to evaporate from the pavement's layers.
- Limit the amount of water that can seep through the pavement from surface dripping (often done by extending a seal over the shoulder).

All of the aforementioned actions require the shoulder's specific form. The road border and shoulder should initially converge at the same altitude. Therefore, the shoulder shouldn't start any higher or lower than the edge of the road. Gravel shoulders that properly meet the border of the road and direct water into the ditch are shown in Figure 13 as an example. (ERA Part D, 2011)

Road shoulders should be positioned at a side slope that is equal to or slightly steeper than the travel lane. The height of the shoulders needs to be carefully planned and constructed. If the shoulder is only slightly raised above the level of the road surface, the issue will only worsen from there. (PennState, 2019)

Additionally, in accordance with (ERA Part D, 2011), if the shoulders are built of gravel and the carriageway is paved, the cross-fall of the shoulder must be 1.5–2.0% steeper than that of the latter. Shoulder widths are reduced in steep terrain and along escarpments to lower the costly expenditure of earthworks. The shoulder will typically be a key component in the overall cross-section design in such terrain, which will also include major drainage and erosion control measures.

Figure 13. Good example of gravel shoulder (FHA, 2015)

2.4.3. Ditches

A common saying in the business is that drainage, drainage, and drainage are the three most important ideas to understand while building and maintaining roads. Simply described, ditches are elements of the roadside drainage system. (FHA, 2015)

Depending on how important the infrastructure is, a large-scale project requiring trucks, loaders, excavators, and other equipment may be required. However, during a dry season, a maintenance worker with little more than a grader may be able to restore ditch drainage. The two types of surface drainage systems featured in this are open ditches (rectangular, U-shaped, trapezoidal, and semi-circular), and trench drains with gravel filling. (ERA Part D, 2011)

Most often rectangular, trench drains are. In order to move water as efficiently as possible, open ditches should collect runoff from the catchment region. Splashing, turbulence, and ditch wall erosion could result from abrupt changes in flow direction. (ERA Part D, 2011)

For the road to continue to last as long as it was intended to, water must be drained from the road surface and into a ditch. The roadside ditch is the most crucial and typical drainage component required on a rural road. Every effort must be made to keep at least a small ditch in good condition. The ditch needs to be cleaned if it becomes blocked by eroded dirt or debris. This can occasionally be a large project needing trucks, loaders, excavators, and other machinery. However, during a prolonged dry season, a maintenance worker who just has a motor grader at their disposal can perform simple tasks to restore ditch drainage. (Anon., 2021)

(FHA, 2015) asserts that water must drain from the road surface into a ditch in order for the road to continue to function as intended. Every effort must be made to keep at least a small ditch in good condition. The ditch needs to be cleaned if it becomes blocked by eroded dirt or debris. This can occasionally be a large project needing trucks, loaders, excavators, and other machinery. However, during a dry spell, a maintenance worker who just has a motor grader at their disposal can perform simple tasks to restore ditch drainage. A neatly reshaped flat bottom ditch constructed with a motor grader is shown in Figure 14. Following a ditch reshaping operation, seeding, erosion control, and sediment management are typical.

Figure 14. Nicely reshaped ditch (FHA, 2015)

Even in arid areas, proper drainage is important. Even the finest efforts to repair or maintain roads will produce unsatisfactory results if drainage is inadequate.

Culverts and bridges are listed as drainage structures in (FHA, 2000) that allow water to flow naturally under roads so that it can continue its route. Eroded soil and debris can quickly block small culverts and box structures. Road maintenance includes cleaning and inspecting them at regular intervals to ensure that drainage is not hindered. A perforated pipe, to transport water out of the roadbed, can be inserted when the road is built over saturated soils or through a body of water. It is crucial to have culverts placed at the proper elevation for water to flow underneath. They need to be maintained on a regular basis to keep them in good shape.

2.5. Surface and base materials for gravel Road

Surface material for unpaved roads is often acquired from stockpiles that have been created for other applications if there is a lack of easily accessible locally excellent material. The gravel, for instance, may have been created to act as the foundation or cushion surface for a paved road. Surface gravel and base (cushion) material differ primarily in two ways: larger top-sized stones and very little clay or tiny particles are common characteristics of gravel used as a base material.

This is necessary for base gravels to have the strength and efficient drainage requirements. If used as surface gravel, this material won't produce a crust to hold the material together. It will get quite difficult to keep up. Given that it frequently contains a lot of sand-sized particles, this substance is highly drainable. This is a beneficial property in fill material because it enables water to quickly travel through it and disperse from building foundations and parking lots. However, if this substance is used to overlay a gravel road, it will remain flimsy and unstable. A sufficient amount of fine material and gravel with a plastic or "binding" property are necessary for a successful gravel road. (FHA, 2015)

Gravel is a mixture of three kinds or types of material: stone, sand, and fines. This will be expanded upon in the following section. Without an appropriate blend of these three sizes, the gravel will function badly. (FHA, 2000)

(FHA, 2015) underlines the importance of using the best materials available. Although a higher-grade surface gravel may cost more, it is usually well worth the extra cost. Quality can only be determined by skilled field sampling and testing in a materials lab.

It is evident that the materials to be used heavily depend on the aggregate sources that are close by and other relevant aspects. On a mountain road, a high-quality surface gravel is regarded as a nice blend of stone, sand, and fine-sized particles. The gravel won't work well if it doesn't have a good mixture of these three sizes. Unfortunately, the maintenance worker will frequently get the responsibility for gravel that isn't operating well. Surface gravel and base (cushion) material are distinguished separately by (FHA, 2015). Gravel used as a base material typically contains larger top-sized stones and only a very tiny amount of clay or fine particles. This is essential for the base gravels' required strength and good drainage properties. This substance won't produce a crust to hold the substance together if utilized as surface gravel. It will become quite challenging to maintain. (FHA, 2015)

Adding fresh, clean, virgin fine gravel is one frequent approach for enhancing surface gravel, according to (FHA, 2000). A portion of stone is required on good surface gravel to provide strength and support loads, especially in rainy weather. To fill the spaces between the stones and provide stability, it also needs a certain proportion of sand-sized particles. A good gravel road also requires gravel with an adequate amount of fine material and a plastic or "binding" quality. When the weather is dry, part of the fine material in surface gravel will be lost due to traffic in the form of dust that rises to the surface and blows away. This can be made up for by requiring a larger proportion of particles in the new gravel. But no gravel surface will function as well as pavement! (FHA, 2000)

In compliance with (FHA, 2000)specifications, recycled asphalt is also utilized to construct gravel road surfaces, with the bituminous component of the original pavement acting as the binder. It is impossible to maintain a blade on recycled asphalt that has been set down on a road in hot weather because it may resemble pavement and take on the qualities of pavement. However, the pavement will be brittle because the majority of recovered asphalt is oxidized. It will be challenging to maintain and prone to pothole development. The material should only be used on roads with a firm subgrade, and it should be buried at least three inches deep, according to (FHA, 2015)

Figure 15 provides a superb illustration of surface gravel that has withstood the winter in the northern high plains.

Figure 15. A very good surface gravel. (FHA, 2015)

It is crucial to test the pebbles used for gravel road paving because they might vary greatly depending on several different circumstances. Gradation and flexibility, or cohesive characteristic, of the fine fraction of the material should be the main points of interest in this situation. Additional testing can give information on factors that affect the performance of surface gravel, including hardness or soundness, gradation, the percentage of fractured stone, and plasticity index. In order to determine the aggregate gradation, sieve analysis is used. Each specification should be examined because most countries, as well as their separate states and regions, have their own requirements for aggregate grading. (Engineer, 2021)

2.6. Distresses in Unpaved roads and their Maintenance

Unpaved roads become less functional over time as a result of traffic and environmental variables. The volume of traffic, particularly the wheel load and its repetitions, determines how much distress there is. The performance of drainage structures is impacted by external elements such rainfall, which causes erosion of shoulders and slopes, water intrusion into the pavement structure, and subgrade. Frost action will occur if snow-related moisture seeps into the subgrade and pavement structure. Compared to unpaved roads, a gravel road is far more susceptible to environmental effects. (Anon., 2021)

Dust, ravelling (the generation of loose material), corrugations (wash boarding), potholes, rutting, and foundation problems are typical issues that may impact unpaved roads, according to many manuals including (ARRB, 2020), (AASHTO, 1993), (IRC, 2008), and (TRH 20, 1990).

Before devising cost-effective remediation options, pavement faults must be recognized and identified. The defects are broadly categorized as surface and structural in (IRC, 2008). The surface flaws, as their name suggests, are limited to the top layers of the pavement, and largely affect the comfort and safety of other road users as well as the ride quality. Poor compaction, improper grading, adverse weather, poor materials, inadequate drainage, disregard for normal and preventative maintenance procedures, or a combination of these, are some of the causes. Dust, corrugations, potholes, rutting, and ravelling are some of the more typical surface flaws.

(IRC, 2008) adds that grading the surface may be used to rectify surface flaws. Gravel roads with structural flaws, however, would need a more thorough examination and care, including reconstruction.

Investigating the origins of structural faults is necessary since they are typically identified by significant area settlements, heaving, or rutting. Lack of drainage, inadequate compaction, and the use of the wrong or insufficient material to support the axle loads are some of the possible causes. (ARRB, 2020)

(ARRB, 2020) defines structural problems as broad area settling, heaving, or rutting, and states that the causes must be investigated. Lack of drainage, inadequate compaction, and the use of the wrong or insufficient material to support the axle loads are some of the possible causes. As a structural defect, heaving, settling, and subgrade intrusion are all listed. The following are some potential maintenance and solution options.

- Place subsurface drains
- Make sure the side drains and surface drainage function properly.
- Establish the necessary pavement thickness to accommodate traffic loads

2.6.1. Dust

Distress description

Dust is defined as the fine particles (finer than 0.425 mm) that is emitted from the road surface beneath moving cars and the turbulence that is brought about by moving vehicles. The absence of particles, protracted dry conditions, and heavy winter sanding are the key factors contributing to the development of dust. Its development depends on both the surfacing material's characteristic and the transit speed. (Légère, 2015)

Stated by (Austroads, 2009) the first indication that an unsealed road surface is wearing is the loss of fine particles. It shows up as dust production and aggregate exposure, which produces a rough, noisy, and coarse surface.

Significance

Dust severely reduces visibility, which can make following and overtaking dangerous. Road users experience discomfort as a result of excessive dust.

From a health perspective, certain substances, such asbestos and silica dust, which may be present in some wearing course gravels, might definitely be viewed as undesirable. Both in dry and moist environments, dust can greatly speed up the rate of wear on a vehicle's moving parts by acting as a grinding paste. It is also undisputed that too much dust frequently contributes to air pollution. In terms of economy, the wearing course gravel's characteristics change as a result of the loss of wearing course material in the form of dust. As the particles are removed, materials that were initially sufficiently elastic have been seen to develop corrugations. (TRH 20, 1990)

Damage Mechanism

Although material composition is the primary contributor to dustiness, other elements such as vehicle volume, vehicle speeds, the amount of moisture in the road, how loose the material is, maintenance frequency, and wind also have an impact. Dust is a result of loosening of pavement materials, which results in the loss of tiny particles (less than 0.425 mm), as well as disruption of the wearing course brought on by traffic and environmental factors. (Légère, 2015, ARRB, 2020)

Figure 16 depicts a road that has been harmed by dust.

Figure 16. A road that is distressed with dust. (Légère, 2015)

Loss of fines causes the surface's permeability to rise, which accelerates the requirement for resurfacing and causes early pavement deterioration. Loss of particles also exposes a surface with a rougher texture, which results in more irregularities and thus greater levels of operating expenses for vehicles. Additionally, it affects difficulties with traffic safety. (Austroads, 2009)

For dust reduction and stability, (FHA, 2000) suggests using chlorides, resins, natural clays, asphalts, soybean oil, and other industrial binders. A nice example of application equipment is shown in Figure 17. The truck includes a computerized application system and a pressurized spray bar that precisely meters the liquid chlorine.

Figure 17. A truck applying liquid chloride on unpaved road (FHA, 2000)

It is crucial to remember that any dust-suppression method used on an unsealed road surface is temporary and only serves as a component of a larger road management plan, which may call for repeated applications of the palliative (water or water mixed with an additive). (Austroads, 2009)

Utilizing specialized detection equipment created by the US Army Research and Development Center, dust generation may be quantified (Rushing 2006). Figure 18 depicts the apparatus.

Figure 18. Mobile and static dust monitoring apparatus (Rushing 2006)

2.6.2. Ravelling

Distress description

The phenomenon known as ravelling—the formation of loose gravel under traffic—poses serious safety and financial challenges. Although loose gravel can be dispersed along the whole width of the road, windrows between wheel tracks are where it is most frequently found. Safety risks, car and windscreen damage, higher fuel consumption, and insufficient lateral drainage are the issues that result. (ARRB, 2020)

Significance

On unsealed roads, loose material is a serious issue. Windrows of loose items on the road are a major contributor to single-vehicle accidents. Vehicles may finally overturn if these windrows interfere with their ability to maintain directional stability; the interference increases with vehicle speed. In accordance with the Committee of State Road Authorities (CSRA) report from 1990, it is crucial that these windrows not be allowed to rise over 75 mm.

Gravel loss is the primary cause of gravel roads' high whole-life costs and frequent unreliability, especially as traffic volumes rise, according to (ERA Part B, 2011).

Damage Mechanism

Water-bound pavements stay put during rainy weather. However, if left to dry out, the pavements may begin to crumble, emitting dust and producing an excessive amount of loose aggregate as the surface material ravels. By adding new material that has been graded properly and mixing it with the current surface material, the surface can be recovered. After the new and old materials have been combined, the newly created surface needs to be watered and compacted to form a crust. (ARRB, 2020)

One method of lowering long-term expenses is to reduce gravel loss by using higher-quality gravels or altering the qualities of lower-quality materials. Additionally, according to (TRH 20, 1990), fine material can frequently be mixed with gravel to improve cohesiveness and a high level of moist compaction can also be employed to reduce raveling. Figure 19 provides an illustration of how loose material on the road might lower skid resistance by acting like pebbles on the road surface.

Figure 19. A road distressed with ravelling (Légère, 2015)

According to (Légère, 2015), possible solutions for this issue include employing a maximum aggregate size of 25mm, adequate compaction, and aiming for a fines content of 10-15% in the wearing coarse aggregate.

In a number of investigations on unsealed roads, the rate of loose surface material deterioration can be measured. In this simple test, the loose material inside a square meter of pavement is removed, and it is then weighed (Figure 20). It is possible to rate surfaces as indicated in Figure 20 due to the material loss over time. (Austroads, 2009)

Figure 20. Measurement of loose material on pavement surface (Austroads, 2009)

2.6.3. Corrugation (washboarding)

Distress Description

One of the most prominent defects on unpaved roads is corrugation, which results in excessive roughness and poor vehicle directional stability. Tyre movement combined with the mass and speed of the vehicle causes corrugations to occur through material displacement. (AARB 2000, TRH 20 1990).

(ARRB, 2020) distinguishes corrugations as either loose or fixed. Loose corrugations are formed by parallel crests of free, fine-sandy material that are perpendicular to the direction of motion. The firm, fine-sandy crests of fixed corrugations, on the other hand, are compacted and parallel. With the force of the wheel restoring touch with the ground, the depressions are compressed. Fixed corrugations must be cut with the grader before the material is distributed again, although loose corrugations can be easily removed by blading.

Significance

Corrugation primarily affects the cost of operating a vehicle, the quality of the ride and the risk to safety. Some car parts, such as shock absorbers and bolts, might be damaged by the roadways' abrasiveness. It is advisable to halt sometimes if traveling a long distance on truly bone-jarring corrugation. It would be wise to do this every 45 to 60 minutes or earlier to allow the shock absorbers a chance to relax. Tiny bubbles will eventually form inside the shock absorbers with continued use in these challenging conditions because of the shock absorbers' continual heating from vibration. (4WheelingAustralia, 2012)

Bolts and nuts will also become looser after extended driving. In addition, anything metal screwed into plastic will certainly come loose before anything else, and plastic aftermarket hinges and locks are prone to breaking. (4WheelingAustralia, 2012: ARRB 2020)

Damage mechanism

The drawings in Figure 21 can be used to explain how corrugations arise. The theory behind the explanation is that a road irregularity or potentially even worn suspension parts cause the wheel bounce. The action causes non-cohesive material to kick back, and as the wheel regains contact with the road, the wearing course is compressed and redistributed. Normally, at natural points of gear change, braking, or acceleration, corrugations may develop where there is loose surface material. (Figure 25)

Although there has been much discussion on the origin of washboarding over the years, the "forced oscillation idea" Figure 21 is now widely accepted as the main mechanism. Recent South African study has revealed evidence to support this (Paige-Green, 1989a). The notion is based on the idea that a road irregularity causes the wheel to bounce (or possibly even worn suspension components such as shock-absorbers). The action causes non-cohesive material to kick back, and as the wheel regains contact with the road, the wearing course is compressed and redistributed. (TRH 20, 1990)

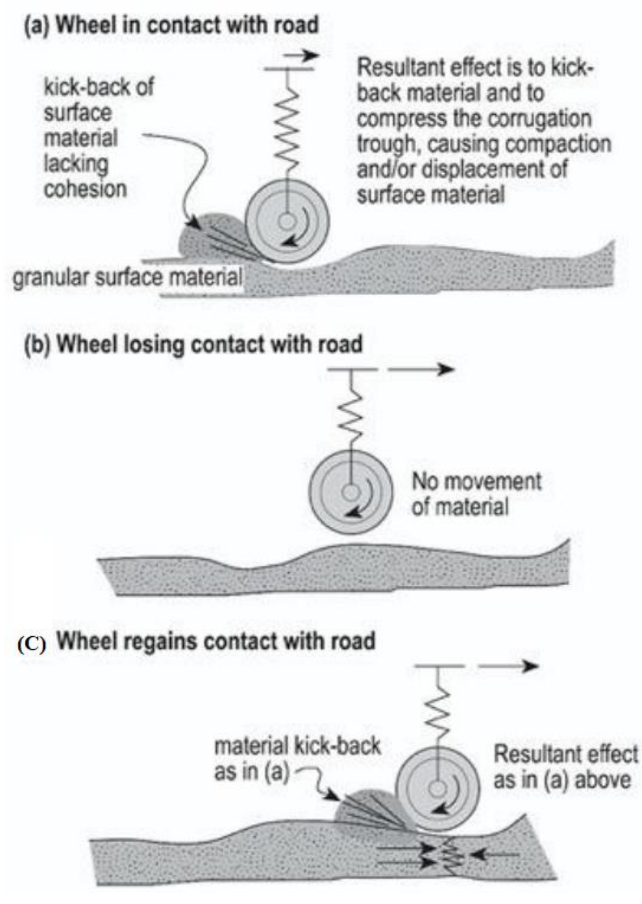

Figure 21. The "Forced oscillation theory" for the formation of corrugation. (After Heath and Robinson, 1980)

The material on the surface organizes itself into parallel ridges that are perpendicular to the flow of traffic. The range of the spacing (wavelength) is 500 mm to 1 m, and the depth is up to 150 mm. The wavelength and depth of the corrugations are shown in Figure 22. Any surface imperfection has the potential to trigger the process, which then progresses at a rate determined by the volume of traffic, areas of acceleration and deceleration, suspension systems, and tyre pressure. (ARRB, 2020)

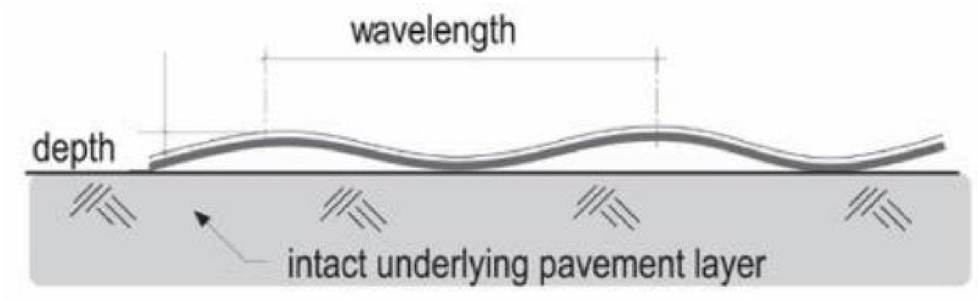

Figure 22 Corrugation formation in dry climates (OEDC 1987)

According to CSRA (1990), the modal speed affects the corrugations' wavelength. Numerous South African observations show that the wavelength of corrugations in centimeters is almost equivalent to the average speed of the cars in kilometers per hour (i.e., a speed of 70 km/h will produce a corrugation wavelength of 70 cm).

Corrugations can occur in granular materials with particle sizes more than 5 mm, poor plasticity, little fines, or materials that have lost fines as a result of traffic action. Only the material that produces the ridges is impacted in dry weather; the underlying material is unaffected. Cutting to the depth of the corrugation and then redistributing the materials constitutes maintenance or remedial work. (ARRB, 2020) The essential corrective action to get rid of the flaw is shown in Figure 23.

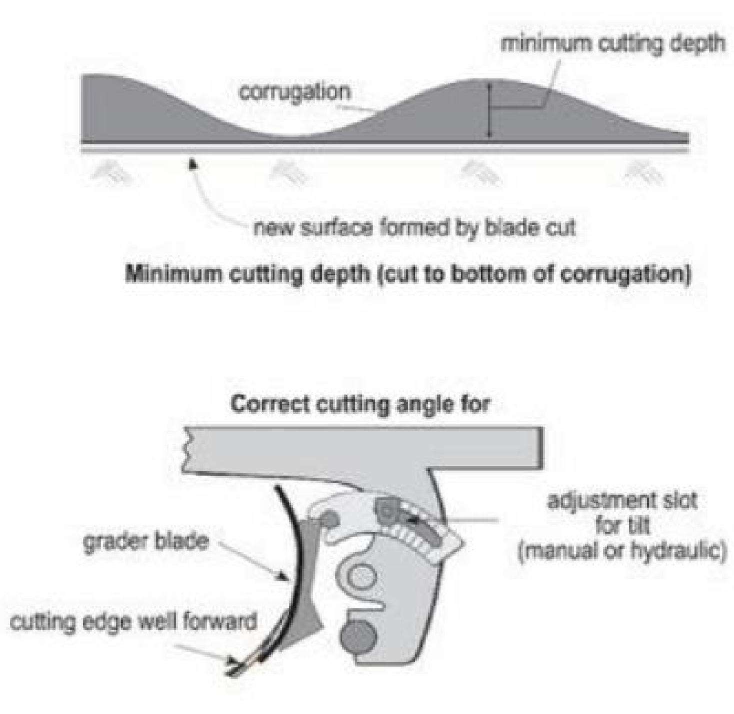

Figure 23. Cutting corrugations (Ferry 1986)

The season affects the development of corrugations and corrective measures in wet regions. Corrugations form during the dry season in a manner similar to that found in dry climates, and the same corrective activity is required to address the flaw. However, during the rainy season, water seeping through the surface may transport surface deformations into subgrades and lower pavement layers, leading to structural flaws. As a result, deformations in lower pavement layers and subgrades may be in sync or out of sync with the corrugation. Potholes may emerge at the corrugation's trough where the deformation and corrugation are out of phase. Figure 24 shows deformations that are both in step with and out of step with the corrugations.

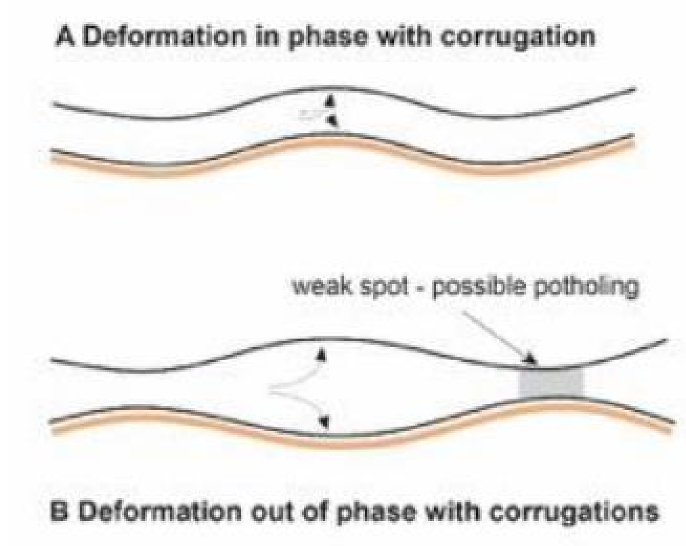

Figure 24. Corrugation formations in wet climates (OECD 1987)

Corrugations in the surface of the gravel are shown in Figure 25.

Figure 25. Corrugations in gravel surface. (ARRB, 2020)

The following are maintenance possibilities, according to (ARRB, 2020)

- Cutting to corrugation depth and redistributing materials
- Sufficient compaction at the ideal moisture level.

- Proper attire and choice of course materials.
- Better grading procedures.
- Cheap "drags" (only a short-term solution and not very effective). Figure 26
- Import and combine components of higher quality.
- Sealing off areas with persistent corrugations.
- Improved road alignment

Figure 26. Low-cost drags (ARRB, 2020)

By carefully incorporating chosen clay binder, especially where dry weather corrugations emerge, longer-term solutions may be identified. For specific difficulty regions, it could be required to import higher-quality crushed aggregate when it is available. In crucial spots, such as on bridge approaches, at livestock grids, or at steep grades and low-radius horizontal bends, short portions of seal may also offer the solution.

2.6.4. Potholes

Distress Description

Potholes are defects brought on by water ponding as a result of insufficient crossfall, insufficient compaction, and excessive pavement moisture deterioration. They are described as "bowl-shaped depressions in the pavement surface" with sharp edges and vertical sides towards the top of the hole. (ASTM Designation 6433). Aside from the technical description, we are aware of them as holes in the road's surface that are frequently an inconvenience when driving. They can develop on any type of road surface, including asphalt pavement and unpaved dirt roads. Additionally, potholes are not just seen in certain climates. Although they are frequently connected to cold settings, they are also common in warm climates. (Légère, 2015)

Significance

If not repaired in a timely manner, potholes can get larger and play a significant effect in the ride quality or roughness of unsealed roads. They can also cause severe damage to automobiles. Both the depth and diameter of a pothole affect how it affects moving vehicles. The most dangerous potholes for motor vehicles have a diameter of 250–1500 mm and a depth of more than 50–75 mm. (Austroads, 2009)

Since the bottom of the tire is what hits the road when a tire rolls over a pothole in the road, most of the effort applied to the suspension is directed upward. As a result, when a wheel strikes a pothole, it falls in and must roll out of the hole. The impact as the wheel exits the hole, rather than the wheel itself, nearly never causes harm. The size of the tire, the length and depth of the pothole, and the speed of the car all affect how much damage is done. The impact is more severe the smaller the wheel diameter, particularly if the hole's depth is equal to the wheel's diameter. (FHA, 2015; ARRB, 2020)

Longer potholes are more dangerous because at almost any speed, the tire is likely to contact bottom before departing, causing damage to components like the vehicle chassis or platform. (Légère, 2015)

Damage Mechanism

Flatter gradients and crossfalls are particularly prone to potholing, as are alignment changes from "left to right" and crossings where water can be present on the surface, especially in wheel paths. The occurrence of potholes is uncommon on gravel roads with proper crossfall and superelevation. Potholes form when the surface material is stripped away and water seeps into the ground. Wheel motion removes suspended solids from the pavement's surface, and as water seeps through the pavement, the action continues, creating a hole in the pavement (Figure 27). (Austroads, 2009)

Figure 27. Potholes on flat crossfall (ARRB, 2020)

Restoring the surface's crossfall and form will fix potholes and stop water from pooling in the flat areas. If the road portion is regulated by longitudinal drainage, such as in low lying areas or at curve transition points, it may be challenging in some locations to accomplish the requisite crossfall. (Légère, 2015; ARRB, 2020)

Patching won't be enough to fix the issue if the road doesn't have enough crossfall. Minor potholes can occasionally be repaired using a grader equipped with tipped blades. In areas with considerable potholing, the surface will need to be scarified, remixed, and reshaped. To replace material lost to traffic, erosion, or wind-driven debris, new material must be introduced and blended with the already-existing material. Potholes are challenging to fix; relatively few are effectively filled by hand behind the grader or by routine grader maintenance. The only method to fix them successfully is to enlarge the hole, overfill it with moist gravel that resembles the substance of the current pavement, and compact it in layers, if necessary. (ARRB, 2020)

(ARRB, 2020) also advises utilizing material with the same qualities as the pavement to ensure equal wear of the road surface and minimize contamination of the pavement components, even though some road authorities have reported success using stabilized material to fill potholes.

2.6.5. Rutting

Distress Description

As demonstrated in Figure 28, ruts are longitudinal deformations or depressions in wheel pathways brought on by the passage of vehicles. Rutting is a surface defect as well as a structural one. They may develop as a result of the wearing course being compacted, the subgrade being deformed, or the wearing course losing gravel. (ARRB, 2020; TRH 20, 1990)

Figure 28.Rutting in Wheel paths (Source: ARRB Group)

Dry season rutting occurs when loose material is moved sideways while traffic is using the same wheel paths. It occurs in non-cohesive materials like sands or gravel that have low fines content. Materials that are sensitive to water, however, exhibit wet season rutting. Deformations develop when water penetrates the pavement through capillary action from the subgrade or from the surface. (ARRB, 2020)

Significance

Ruts can be an issue because they like to cling onto rainwater, which makes the surface softer and more prone to deformation when subjected to traffic. Ruts may be dangerous because of the accumulation of loose material between them. (TRH 20, 1990)

Rutting causes difficulties for users by increasing fuel consumption and the possibility of skidding (on ice or water). Ruts promote water to sink into the pavement rather than drain off the surface, which causes the pavement to deteriorate quickly and causes problems for the owner as well. This type of water infiltration into the pavement runs the risk of accumulating in a subgrade rut that is "buried," as well as lowering the granular layers' capacity to support loads. Less noticeably, as rutting is frequently uneven along the length of the road, it causes discomfort for users. Additionally, there is increased friction against the side of the tire, which increases fuel consumption and tire wear. (A. Dawson & P Kolisoja, 2006)

Damage mechanism

Contributory mechanisms exist for both structural and surface rutting, and in actuality it is frequently caused by a mix of modes. They may develop as a result of the wearing course being compacted, the subgrade being deformed, or the wearing course losing gravel. Rutting often has little impact on the overall performance of an unpaved road in a given area. (TRH 20, 1990)

Rutting can be brought on by the basecourse, subgrade, or surface material failing as a result of too much water penetrating the pavement, a shallow subgrade, or a shallow pavement depth. (ARRB, 2020)

Surface Rutting

Local shear near the wheel may happen in granular materials that are weaker. This causes heave to occur just next to the wheel path (Figure 29). This rutting is mostly a result of the aggregate's insufficient shear strength being located relatively close to the pavement surface.

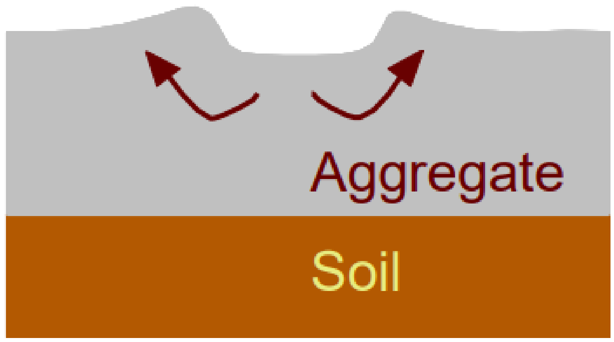

Figure 29 Deformation within granular layers of the pavement, near the surface (A. Dawson & P Kolisoja, 2006)

A Scottish forest road in Figure 30 displays this kind of damage. The typical shoulder heave that is visible in the images can be seen. (A. Dawson & P Kolisoja, 2006)

Figure 30. Close up caption of a rut in Scottish forest (Photo: courtesy W Tyrrell) (A. Dawson & P Kolisoja, 2006)

In regions where there is seasonal frost, this kind of rutting is regularly seen. When an aggregate of poor quality temporarily loses its ability to carry loads due to spring thawing and excess moisture, it is likely to be the primary cause of the accumulation of rutting in many circumstances. (A. Dawson & P Kolisoja, 2006)

The only way to prevent this rutting is to either improve the aggregate or lessen the stresses that the tyres put on the surface. This type of rutting will not be affected by subgrade remediation. By compaction (within limitations), stabilization, the application of a geosynthetic reinforcement, or by changing the environmental factors that influence its behavior, such as drainage, the granular material may be improved. If none of these methods work, it could be necessary to replace the aggregate. Instead, tire pressure can be decreased. (Légère, 2015; A. Dawson & P Kolisoja, 2006)

Structural Rutting

The vertical resilient strain that occurs at the top of the subgrade soil, which is determined by the resilient modulus, the thickness, and the Poisson ratio of the layers in a pavement system, is often linked to the rutting mechanism in subgrade soils. A higher strain amplitude (poor structural capacity of the pavement, higher loads, etc.) is linked to a rapid failure. The strain amplitude is connected to the amount of load repetition before reaching a failure situation (lower load repetitions). The literature suggests parameters of 25 to 75 mm when defining the failure as a specific rut depth. (J.-P. Bilodeau, 2017)

Shear deformation, insufficient pavement depth, and a poor subgrade all contribute to ruts as a structural problem. Figure 31 displays an idealized depiction of the subgrade deforming as one or more granular layers physically deflect on it (i.e. without any thinning). As it is the displacement of the top layer material that produces this, the surface deformation pattern is that of a wide rut with a minor heave away from the wheel. (A. Dawson & P Kolisoja, 2006)

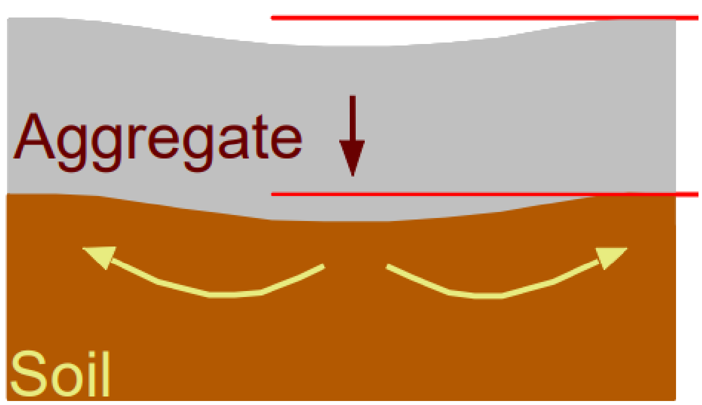

Figure 31. Shear deformation within the subgrade (A. Dawson & P Kolisoja, 2006)

Figure 32 exhibits an extreme case of this kind of failure. In this instance, the surface rut has been regularly filled, yet afterward, aggregate rutting at the subgrade surface has persisted. The arrows in Figure 31 illustrate a fairly sophisticated example of rotational shear within the subgrade where the subgrade has to compress upward between the wheel tracks and in the margin.

Figure 32. Advanced case of Structural rutting with weak subgrade (photo: courtesy W Tyrrell) (A. Dawson & P Kolisoja, 2006)

The Spring thaw issue can result in structural rutting of the subgrade in areas that experience deep seasonal cold. In such circumstances, excessive structural rutting is only visible in the spring when subgrades are temporarily softened by an increase in moisture as a result of thawing. (A. Dawson & P Kolisoja, 2006)

The best way to prevent this kind of rutting is to make the aggregate better or thicker so that the wheel loads are distributed more evenly. The subgrade will then be under less stress. Another strategy is to limit high axle loads, which have the biggest effects on stress at depth (as opposed to tyre stresses). (A. Dawson & P Kolisoja, 2006; Légère, 2015)

Chapter 3

3. Maintenance and rehabilitation of Unpaved Roads

Two fundamental concepts are required for good unpaved road upkeep or rehabilitation: correct motor grader (or other grading device) use, and use of high-quality surface gravel. (FHA, 2000)

Most professionals in the field are aware of how to use a grader to shape a road appropriately, but they may not be as knowledgeable about the type and quantity of unbound material required. Although the underlying issue is frequently a material-related one, it seems that the majority of gravel maintenance and rehabilitation issues are faulted on the grader operator. This is especially accurate when dealing with the corrugation or "washboarding" issue. Even though the grader operator is frequently criticized for the issue, the actual source of the issue might be in the material. (Légère, 2015) (FHA, 2015)

The significant change in the types of vehicles and equipment that use low volume highways is another crucial factor to be mindful of. The size and horsepower of trucks and agricultural machinery are growing. The machinery is getting bigger and bigger. It is generally known how larger, heavier vehicles affect paved roadways. There is unquestionably a need to construct more durable bases and pavements. However, the impact on unpaved roads is equally detrimental and frequently goes unnoticed. To bear heavy weights, one must pay attention to the depth of the material and the subgrade's strength. Another crucial factor is proper drainage. (K. Skorseth, 2012)

In general, unpaved roads require routine blading and unbound material additions as needed, either by "spot graveling" or by completely regraveling portions. But practically any gravel road will gradually start to exhibit problems that call for more than simple repairs. The "berms" or secondary ditches that accumulate along the shoulder line and the transfer of debris from the surface to the shoulder region, sometimes even onto the slope of the grade, are the most frequent issues that arise. Almost all gravel unpaved roads eventually need significant renovation. (Administrative Manual, 2020, FHA, 2000)

The upkeep of gravel roads or unpaved roads in general is broken down into maintenance and repair operations. Repair efforts are targeted at multiyear variables while maintenance is focused on one-year issues. Spring maintenance, summer blading and dust binding, and fall maintenance make up the gravel road maintenance cycle. According to current RWA regulations, five potential maintenance strategies were defined for this system. During the multi-year rehabilitation, wearing course material will be added, and the road's structural integrity will be improved. During the spring or fall maintenance, the wearing course material is introduced. Repairing harmed base points, improving drainage, and updating the road's construction are all included in the structural condition upgrades. (FRA, 1988)

It will be necessary to constantly interpret measurement data and estimate distress models in the future. Continuous research is needed to understand how unpaved roads deteriorate. Continued research is required into the gravel road condition variables, particularly the state of the road's structure. (FRA, 1988; FHA, 2000)

3.1. Reshaping Surface and Shoulder

Usually, with just the motor grader, reshaping of surface and shoulder may be accomplished. The greatest season for this is spring because there is less vegetative growth and there is moisture. By cutting material with a motor grader and redistributing it to the correct shape and crown, the driving surface and the road shoulder can be reshaped. If at all possible, compaction should be done with a roller since this would substantially enhance the final surface. The result will be a surface that is smoother, stronger, denser, and easier to maintain. Figure 33 clearly depicts an illustration of reshaping a shoulder of unpaved roads built in the 1970s. (FHA, 2015; NACE/APWA, 2015)

Figure 33. Reestablishing of shoulder line (Photo credit NACE/APWA 2015)

3.2. Reshaping Entire Cross Section

A significant reshaping is required if there is a combination of any two or more of the following: severe rutting, loss of crown, gravel loss, and deep subsidiary ditches. This calls for much more work. It frequently happens following an unusually heavy shipment on a gravel road. If a big haul happens in bad weather, it will be worse. In spite of vegetative development, major reshaping frequently needs to be done on the entire cross section right away. (FHA, 2015)

Rollers, disks, pulverizers/mixers, and motor graders are frequently required. These aren't always accessible, but they undoubtedly make the task simpler. It becomes crucial for the operator and field supervisor to understand how to rebuild the cross section. Rarely do these projects benefit from extensive planning or technical support. Surveying and staking are seldom ever done. However, it is crucial to reconstruct the cross section uniformly and to pay close attention to reestablishing proper drainage. Good surface gravel should only be replaced once this has been carried out – and carried out correctly. The front dozer fitted with carbide bits is shown in Figure 34 as an efficient instrument for dealing with washboard areas. (FHA, 2000)

Figure 34. A photo of a front dozer equipped with carbide bits maintaining areas affected by washboards. (FHA, 2000)

Chapter 4

4. Structural Design of Unpaved roads using Different Manuals

4.1. Pavement Design

A pavement's primary purpose is to endure traffic loads, safeguard the subgrade, and deliver a comfortable and secure ride for all users of the road. The operating costs of vehicles, especially large vehicles on haulage and freight routes, can be significantly increased by badly maintained roads that lack the necessary pavement strength or thickness. (ARRB, 2020).

Unpaved roads respond to wheel loading in a flexible manner structurally speaking (TAC 1997), with the stress gradually being distributed and attenuated by the stiffer granular layers throughout their thickness down to a level that is permissible for the typically softer subgrade soil (Doré and Zubeck 2009). Unpaved roads provide a variety of advantages, such as fairly straightforward surface management techniques that may be applied to maintain the pavement in its original state. Similar to paved roads, however, a significant quantity of rutting is frequently associated with rutting of the pavement system's softest layer, which is typically the subgrade (Asphalt Institute 1991)

The premise that unpaved road maintenance and design practices are mostly reliant on local knowledge presents a challenge because these practices cannot be directly applied to different locations and material types. (A. T Visser & W R Hudson, 1983)

Unpaved roads' pavement degradation and distress differ from paved roads' near the surface because issues like fatigue and transverse cracking won't happen on a granular surface. On unpaved roads, however, common problems include potholes, rutting, and washboarding (Skorseth and Selim 2000).

Minimizing travel time, lowering vehicle operating costs, and making the best use of resources are all primary goals of pavement design. The conditions in various areas vary depending on a number of elements, such as the climate, the purpose of the road, and the availability of materials, thus design solutions must be customized accordingly. The strength and performance of the pavement material, climatic factors (moisture and temperature), subgrade strength, and support stiffness all affect how well the pavements operate. (Austroads, 2009). The structural approaches of Australia (AUSTROAD), the United States (AASHTO), India (IRC), and South Africa (TRH 20) are discussed in brief in the sections that follow in this chapter.

4.1.1. Australia (AUSTROAD) design Method

In the design of the pavement thickness of unpaved roads, Australian Road Research Board uses two methodologies. When the majority of the following conditions hold true, the first strategy, a nominal single basecourse (for instance, 100 to 150 mm thick, can be considered: (ARRB, 2020)

- A dry climate and effective drainage
- low volumes of traffic
- traffic comprising mainly of light vehicles
- The road is readily repaired and levelled on a regular basis.
- the risk of overloaded vehicles is low
- availability of suitable paving materials

The second method offers a specified pavement thickness that is between 50 and 100 mm less than what would be necessary for a sealed road. The pavement should have a minimum thickness of 100 to 150 mm. When the following circumstances exist, it is more appropriate: (Austroads, 2009)

- wet climatic conditions and/or inadequate drainage
- increased traffic volumes
- A sizable proportion of the traffic is made up of heavy vehicles.
- the road carries through truck traffic
- The road will probably be used when the subgrade moisture level is high
- overloaded vehicles pose a serious risk.

(ARRB, 2020) identifies the following environmental elements as influences on pavement design:

- The area's rainfall and evaporation patterns.
- Permeability of the drains, surrounding surfaces, and subgrade as well as the wearing surface and pavement layers
- The efficiency and vicinity of the drainage (table drains, culverts)
- Ground water movement and water table depth

- Roadside vegetation, especially trees that hang over the road and shade the pavement
- The regional geology, particularly the existence of open jointed or fractured rock formations that frequently include permeable layers that may allow for substantial seepage flow.

Roads built on wide subgrades may continue to move, with seasonal moisture fluctuations causing form loss, damaging nearby trees, and lowering ride quality. The majority of roads have safeguards in place to prevent water from entering the pavement's structure. Water will have less of an impact on pavement performance if there is a suitable crossfall (4-6%), a tightly bound wearing surface, table drains, cross drains, and, if necessary, sub-surface drainage (or moisture barriers). (Austroads, 2009)

According to (Austroads, 2009), subgrade support is the main variable determining the design of an unpaved road's thickness and is frequently out of the control of both the designer and the builder. However, drainage can affect material strength and subgrade support significantly, thus where applicable, drainage improvements should be given top priority.

The subgrade support in the unsealed road design technique described in (ARRB, 2020) is determined by the material's California Bearing Ratio (CBR). Field and/or laboratory tests, as well as experience utilizing suppositional values, can all be used to calculate the CBR. Additionally, the worst possible subgrade moisture conditions should be accounted for.

The number of Equivalent Standard Axles (ESAs) that are anticipated to travel across the pavement during its design life is a popular way to characterize design traffic loading. According to Austroads (2008b), which categorizes traffic loading according to road class, estimates of traffic loading (ESAs) are made. To get better estimates of traffic loading, on-site traffic counting can be done manually through visual logging or automatically using pneumatic counter strips and vibration sensors mounted to stock grids. (Austroads, 2009),

(ARRB, 2020) suggests the following method for determining design traffic for the purpose of designing unsealed roads:

1. Select a design period or life. For unsealed roads, this is typically 10 years; but a longer duration (of at least 20 years) should be thought about when doing a whole-of-life analysis of options, including potential sealing.

2. Calculate the current heavy vehicle traffic or AADT percentage. In the absence of AADT, average daily traffic (ADT) based on the greatest daily traffic annually may be utilized.
3. Calculate the estimated rise in heavy vehicles over the design period as well as any substantial alterations that are expected as a result of development (such as resource development, mineral extraction, etc.)
4. Calculate the ESA/HV factor for the road's heavy vehicle traffic. Because values can vary greatly, it is recommended to use local data.
5. Calculate the ESAs for the total heavy vehicle loading over the design life (Eq. 1).

The design number of ESAs for pavement design purposes can be determined using Eq. 1

$$\boldsymbol{DESA = \left(AADT * DF \frac{\%HV}{100} * LDF * CGF * 365\right) * \left(\frac{ESA}{HV}\right)} \qquad \textit{Eq. 1}$$

Where,

$DESA =$ design number of ESAs

$AADT =$ annual average daily traffic in vehicles per day in the first year

$DF =$ direction factor

$\%\, HF =$ average percentage heavy vehicles

$LDF =$ lane distribution factor

$CGF =$ cumulative growth factor

$$\boldsymbol{CGF = \frac{((1 + 0.01 * R)^{P} - 1)}{0.01 * R} \; for\; R > 0, CGF = P, for\; R = 0} \qquad \textit{Eq 2}$$

$R =$ annual growth rate (%)

$P =$ design period (years)

$\frac{ESA}{HV} =$ average number of ESAs per HV

Designers are urged to use site-specific traffic data to determine the design traffic for a particular road. Table 1 offers a broad guide based on expected traffic volumes and anticipated ESA for various road classifications, while it is only useful if readily available traffic information is lacking or for preliminary planning reasons. (ARRB, 2020)

Table 1. Indicative design traffic volumes for various road classes (based on 10 year design life)

Road Class (1)	ADT(2)	%HV(3)	%HV growth	ESA/HV(4)	Cumulative design traffic (ESA) (5)
Main road	>150	10	0	0.5	$3.7*10^4$
		20		3.9	$5.7*10^5$
		10	2	0.5	$4.0*10^4$
		20		3.9	$6.2*10^5$
Minor road	50-150	10	0	0.5	$1.8*10^4$
		20		3.9	$2.8*10^5$
		10	2	0.5	$2.0*10^4$
		20		3.9	$3.1*10^5$
Access road	10-50	10	0	0.5	$5.5*10^3$
			2		$6.0*10^3$
track	<10	10	0	0.5	$1.8*10^3$
			2		$2.0*10^3$

Because trafficking typically crosses the centerline on unsealed roads, the lane distribution factor is taken by (ARRB, 2020) to be 1.0.

A constructed and graveled road pavement, as per ARRB (2009), often comprises of distinct layers, as seen in Figure 35. The subgrade, base-course, and wearing course are the three layers that should ideally make up a constructed and graveled road. The subgrade serves as the pavement's base. Heavy wheel loads are dispersed to the subgrade by the base-course. The wearing course offers a tough surface that can endure the abrasive effects of tire wear, as well as reduce water infiltration and dust emissions. The loss of base-course material could be prevented by a suitable wearing course on the basecourse. If no wearing course is specified, the base-course must fulfill both functions. Higher fines content and a higher plasticity index are necessary for this since they will better bond the granular material without making it overly slick and soft when wet. The wearing course and base-course are frequently combined using the same materials in several jurisdictions. (ARRB, 2020)

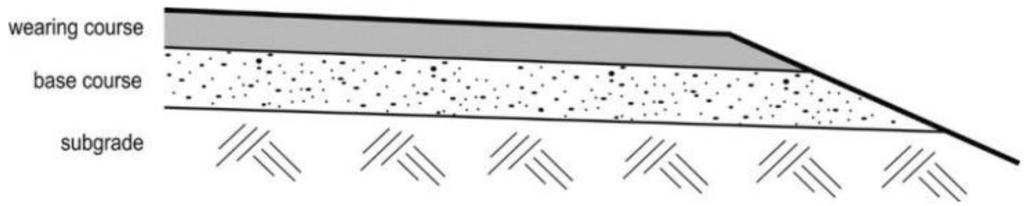

Figure 35. Preferred layers associated with unsealed road (ARRB, 2020)

It is obvious that the materials used for unsealed road pavements are typically chosen based on variables such as availability, material qualities, cost, and environmental considerations. The performance of unsealed roads depends on the choice of the appropriate materials.

The following are listed as the ideal qualities of a wearing course for unsealed roads by (ARRB, 2020)

- Good skid resistance and comfortable riding qualities
- well-graded with a maximum size of 19 mm
- cohesive properties and easy to grade and compact
- resistance to ravelling and scouring
- wet and dry stability
- reduced permeability
- ability to distribute the load.

A well-graded gravel-sand mixture with a sufficient quantity of clayey fines is the optimum unsealed wearing course material, while less ideal wearing course materials include fine-graded silts and silty sands without gravel-sized particles or gravel and sands that are low in fines. (Austroads, 2009)

(ARRB, 2020) also reports the preferred characteristics of a basecourse for unsealed roads as:

- The maximum density concept is used, and a proper particle size distribution is used to ensure adequate strength is achieved through particle interlock.
- For a successful compaction, the stone's shape is crucial. The stones' cubical shape and rough edges work best for mechanical interlocking. In order to avoid cutting vehicle tires, avoid sharp edges.

- Adequate plasticity so that the fine material reduces interlock when wet and provides a cohesive strength to keep the aggregate in place when dry, helping to densify the aggregate.
- Enough aggregate hardness to withstand breakdown during trafficking and compaction.

A soaking CBR value of more than 40% is frequently sufficient for base and subbase materials, according to (Austroads, 2009), and performance evaluation and material availability/cost are given more importance.

Pavement Thickness Design

Sometimes the thickness of the granular pavement is not specified. According to experience, the minimum thickness of granular material usually ranges from 100 to 300 mm. It's crucial to keep in mind that this practice necessitates more regular pavement repairs. On the other hand, determining the thickness of the granular base (and subbase, if present) is necessary for the design of an unsealed road surface. The needed total pavement thickness is determined by the design traffic loading and subgrade support. (ARRB, 2020)

Pavement thickness design curves are given in (ARRB Transport Research, 1998) for residential streets that are sealed and unsealed as well as for country roads with low structural integrity and granular materials. These figures are based on an 80 percent probability threshold, which means there is a 20 percent chance that the pavement will need rehabilitation (i.e., replacing the wearing course for unsealed roads) before it reaches the end of its design life.

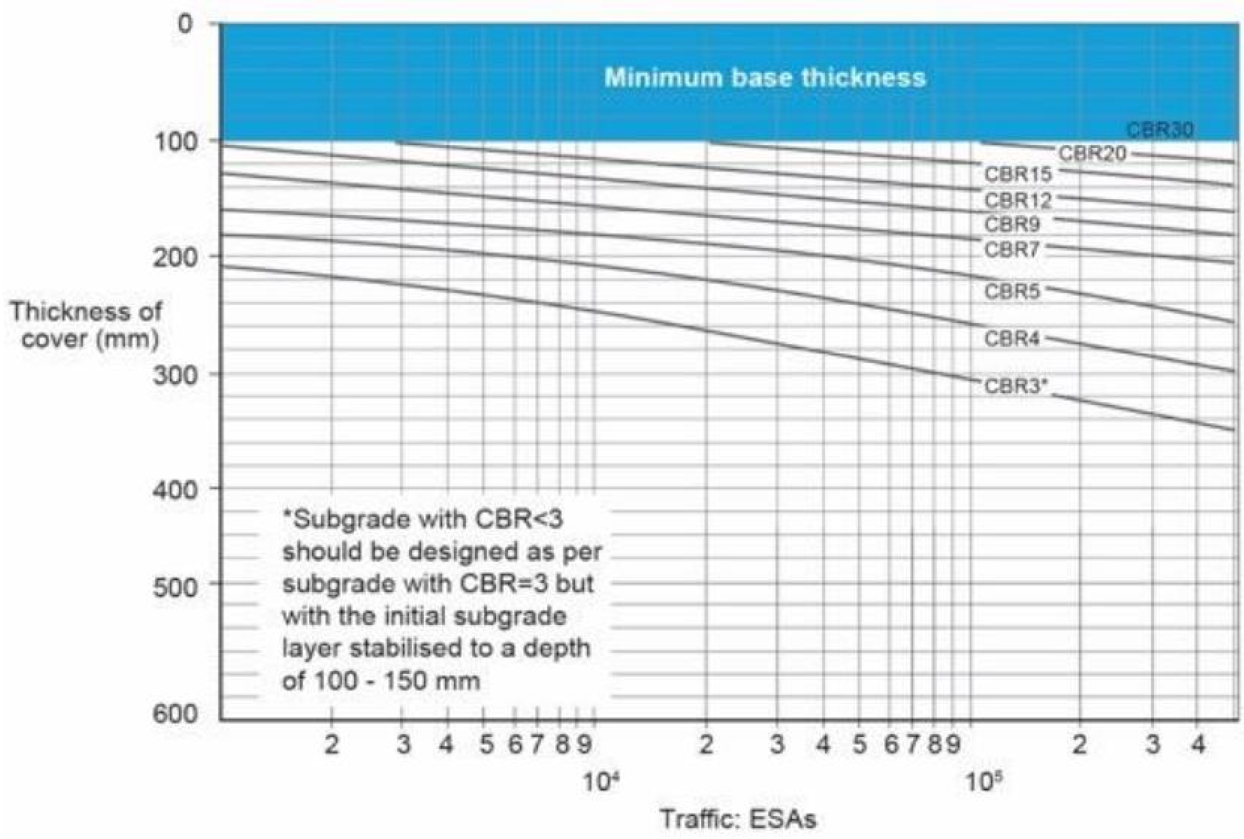

Figure 36. Design chart for granular pavements (80% confidence) (ARRB, 2020)

In order to prevent the subgrade from deforming (rutting) under traffic during its design life, the thickness obtained from Figure 36 serves as a minimum structural thickness. However, it is acknowledged that during the chosen design period the unsealed wearing course's structural thickness will decrease owing to gravel loss. It is understood that regular patrol grading will change the surface's contour notwithstanding the possibility that this loss of thickness could cause surface deformation.

The design procedures are:

- determine the support conditions
- determine the pavement design traffic
- determine the total pavement thickness (i.e. thickness of cover) required using Figure 36

The following are a few of the crucial factors identified by (ARRB, 2020) for designing pavement thickness:

- In cases where a separate wearing course is used, this adds another layer to the overall thickness calculated using Figure 36.
- The minimum depth of the wearing course, which is typically in the range of 75-100 mm, should be chosen to compensate for the potential loss of material during the re-sheeting cycle.
- Where a separate wearing course is not specified, an additional allowance for pavement thickness should be made to account for the loss of gravel over time. It should also be ensured that the minimum gravel depth is not less than 50 mm prior to re-sheeting to limit punching into lower layers and to prevent rapid deterioration.

Pavement Materials

(Austroads, 2009) The characteristics of a pavement material determine its behavior and are influenced by its skeletal structure, the type of stone aggregate, and the fine soil matrix. The following are the main variables that influence how well materials perform in respect to unsealed roads:

- The stability (all pavement layers)
- Wear resistance (wearing course)
- The impermeability (all pavement layers)
- Compaction and workability (all pavement layers).

Table 2 and Table 3, respectively, present typical CBR values and indicated permeability values recommended by (Austroads, 2009) for unsealed road pavement layers.

Table 2. Suggested CBR values for pavement materials for unsealed roads

Pavement Layer	Typical CBR (soaked)
Wearing course (gravel materials)	Minimum 40
Base	Minimum 50
Subbase	Minimum 30

Table 3.Indicative permeability values (100% standard compaction)

Material	Suggested maximum permeability (m/s)
Unsealed wearing course	$1*10^{-4}$
Base and sub-base	$1*10^{-3}$

According to (Austroads, 2009), specifications for unsealed road paving materials are typically more flexible than for sealed roads. However, the fundamental ideas behind desired performance remain the same and are founded on the following three essential components:

- Strength is provided through particle interlock and the maximum density concept in particle size distribution (PSD) (i.e. strength is directly related to density). Additionally, the PSD regulates a soil's permeability, with a focus on the percentage of material that is finer than 0.5 mm.
- Plasticity, where the fine material helps to densify the aggregate by reducing interlock when wet and by offering a cohesive strength to keep the aggregate in place when dry.
- The ability of an aggregate to survive significant breakdown due to trafficking and compaction is known as aggregate hardness. A wearing course must also be strong enough to keep its integrity in challenging circumstances.

Materials with a maximum size greater than 40 mm, according to (Austroads, 2009), should only be used for the base and subbase layers, not the wearing course. However, if the source rock is soft (typical LA abrasion > 50%) or there are too many fines (> 30%), aggregate sizes larger than 40 mm may occasionally be used in the wearing course. Table 4 offers suggestions for appropriate gradings for wearing surfaces for untreated pavements.

Table 4. Typical properties for unsealed road wearing course (source NAASRA, 1980)

Sieve size(mm)	Percent passing for all maximum sizes
55	100
37.5	95-100
26.5	90-100
19	80-100
2.36	35-65
0.425	15-50

0.075	10-40
Plasticity	Less than 500 mm annual rainfall – max. 20 More than 500 mm annual rainfall – max. 12 OR Weighted Plasticity Index (PI x % passing 0.425) Max. 500 for low rainfall Max. 250 for high rainfall
4 day Soaked CBR	Minimum 40%

All maximum particle sizes below 55 mm, such as 40 mm through 20 mm, are accommodated by the PSD envelope depicted in Figure 37 as per (Austroads, 2009). The distribution of sizes below the maximum size (2.36 mm, 0.425 mm, and 0.075 mm), which occurs when the maximum size is less than 20 mm, does not alter. The PSD levels between 26.5 mm and below are adequate for maximum particle sizes greater than 55 mm.

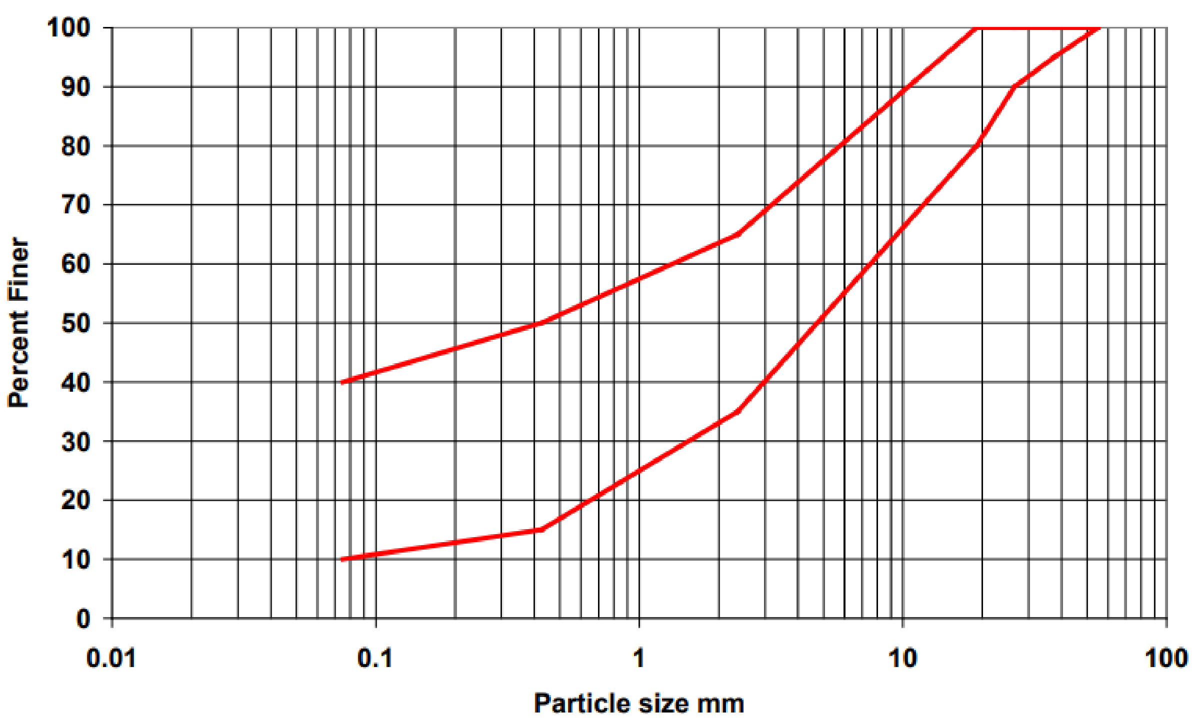

Figure 37. Suggested PSD range for unsealed wearing course (Source based on NAASRA, 1980)

As per (Austroads, 2009), for base and subbase materials, a higher priority is given to material availability and evaluation in terms of performance qualities, as shown in Figure 38, with saturated CBR being greater than 40%.

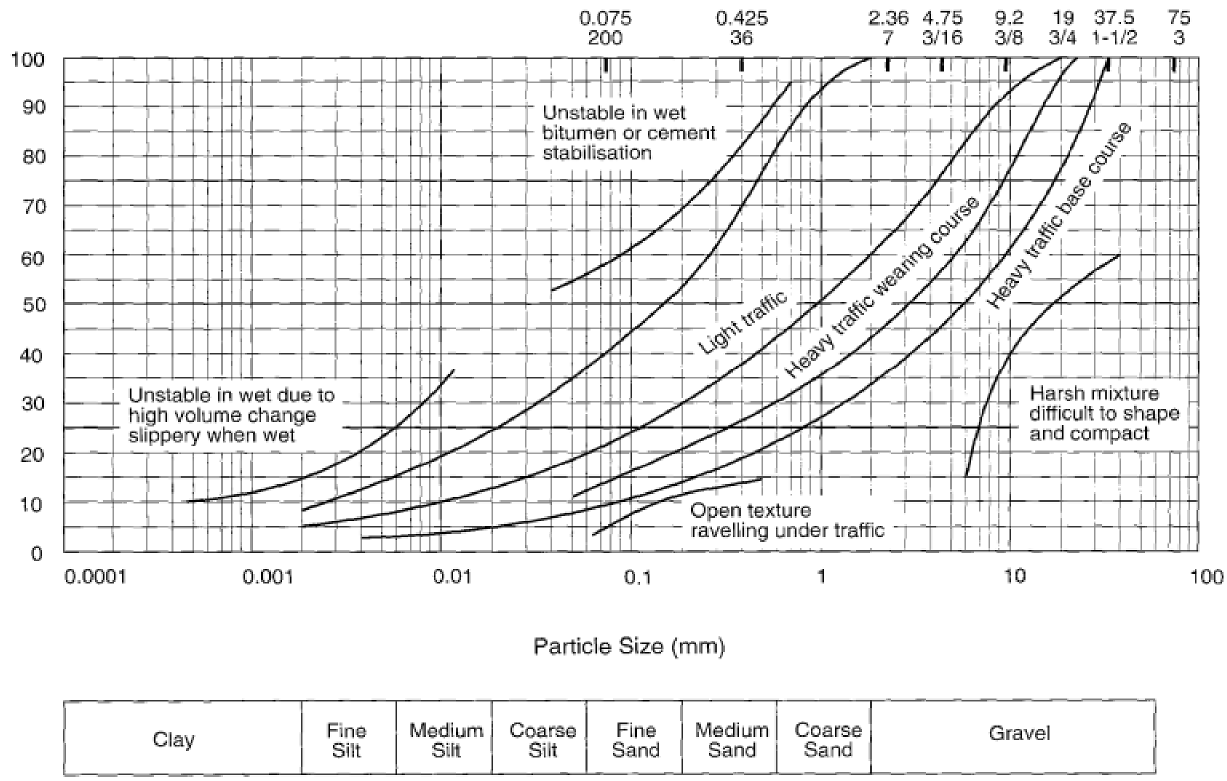

Figure 38. Workability attributes of granular materials. Source: Wooltorton (1947)

The PSD requirements for unsealed road bases and subbases may lie on the finer side of the "light traffic" zone for the performance shown in Figure 38, however slipperiness will be more dependent on plasticity and the proportion of aggregate (coarser than 2.36 mm). (Austroads, 2009)

4.1.2. American Association of State Highway and Transportation Officials (AASHTO)

For the design of low-traffic unpaved roads, AASHTO uses the design chart-based technique and the design catalog. A graphical solution is necessary for the design chart-based process for highways with aggregate surfaces.

Design Chart Procedure

The primary design specifications for aggregate-surfaced roads are listed as follows in (AASHTO, 1993)

- The anticipated traffic, W18 for the time
- The length of the seasons
- Roadbed soil's seasonal resilient moduli
- Elastic moduli, E_{BS} of aggregate base and sub-base layer
- Design serviceability loss, *Δ*PSI
- Rutting permissible in inches in the top layer
- Inches of surface layer aggregate loss, GL

The computational chart in Table 5 and the design nomographs for serviceability Figure 39 and rutting Figure 40 are utilized in conjunction with all of the aforementioned requirement lists.

Table 5. Chart for Computing Total Pavement Damage (both serviceability and based on trial Aggregate Base Thickness (AASHTO, 1993)

TRIAL BASE THICKNESS, D_{BS} (inches) ______				Serviceability Criteria, ΔPSI = ______		Rutting Criteria, RD (inches) = ______	
(1) Season (Roadbed Moisture Condition)	(2) Roadbed Resilient Modulus, M_R (psi)	(3) Base Elastic Modulus, E_{BS} (psi)	(4) Projected 18-kip ESAL Traffic, w_{18}	(5) Allowable 18-kip ESAL Traffic, $(W_{18})_{PSI}$	(6) Seasonal Damage, $\frac{w_{18}}{(W_{18})_{PSI}}$	(7) Allowable 18-kip ESAL Traffic, $(W_{18})_{RUT}$	(8) Seasonal Damage, $\frac{w_{18}}{(W_{18})_{RUT}}$
Winter (Frozen)							
Spring/Thaw (Saturated)							
Spring/Fall (Wet)							
Summer (Dry)							
		Total Traffic =		Total Damage =		Total Damage =	

The processes for the design procedure are outlined below, per (AASHTO, 1993).

- Choose four aggregate base thickness levels, i.e., four different charts to fill. (Table 5)
- In the appropriate columns of Table 5, enter the design serviceability loss, the permissible rutting, the seasonal elastic (resilient) roadbed moduli (M_R), the elastic base material (E_{BS}) (psi), and the seasonal 18-kip EASL traffic.
- Using the serviceability-based nomograph in Figure 39 and entering it in column 7 of Table 5, estimate the allowable 18-kip EASL traffic for each of the four seasons for each of the four tables.
- Using the serviceability-based nomograph in Figure 39, estimate the permitted 18-kip EASL traffic for each of the four seasons and enter it in column (7) of Table 5.
- Divide (Column 4) by (Column 5) and fill (Column 6) of Table 5 to get the seasonal damage values (with respect to traffic) in each of the four tables for the serviceability criterion. The same method is used to determine the seasonal damage brought on by rutting. Fill in Column 8 of Table 5 with (Column 4 divided by Column 7).
- To determine the total damage, add the two seasonal damages.
- By interpolating in Figure 41 for a total damage of 1, it is possible to calculate the average base layer thickness, $\overline{D}_{BS}$, that is needed.
- If the consequences of aggregate loss are minimal, the base layer thickness established in the preceding phase should be used for design. However, if the aggregate loss is considerable, the design thickness is calculated using Eq 3

$$D_{BS} = \overline{D}_{BS} + (0.5 \times GL) \qquad \textbf{\textit{Eq 3}}$$

Where;

GL=total estimated aggregate(gravel) loss in inches over the performance period and is calculated as

$$GL = 0.12 + 0.1223\ (LT) \qquad \textbf{\textit{Eq 4}}$$

LT= number of loaded trucks in thousands

- Using Figure 42, a portion of the aggregate base layer thickness is converted to an equivalent thickness of subbase material as the last stage in the design chart technique for aggregate-surfaced roads.

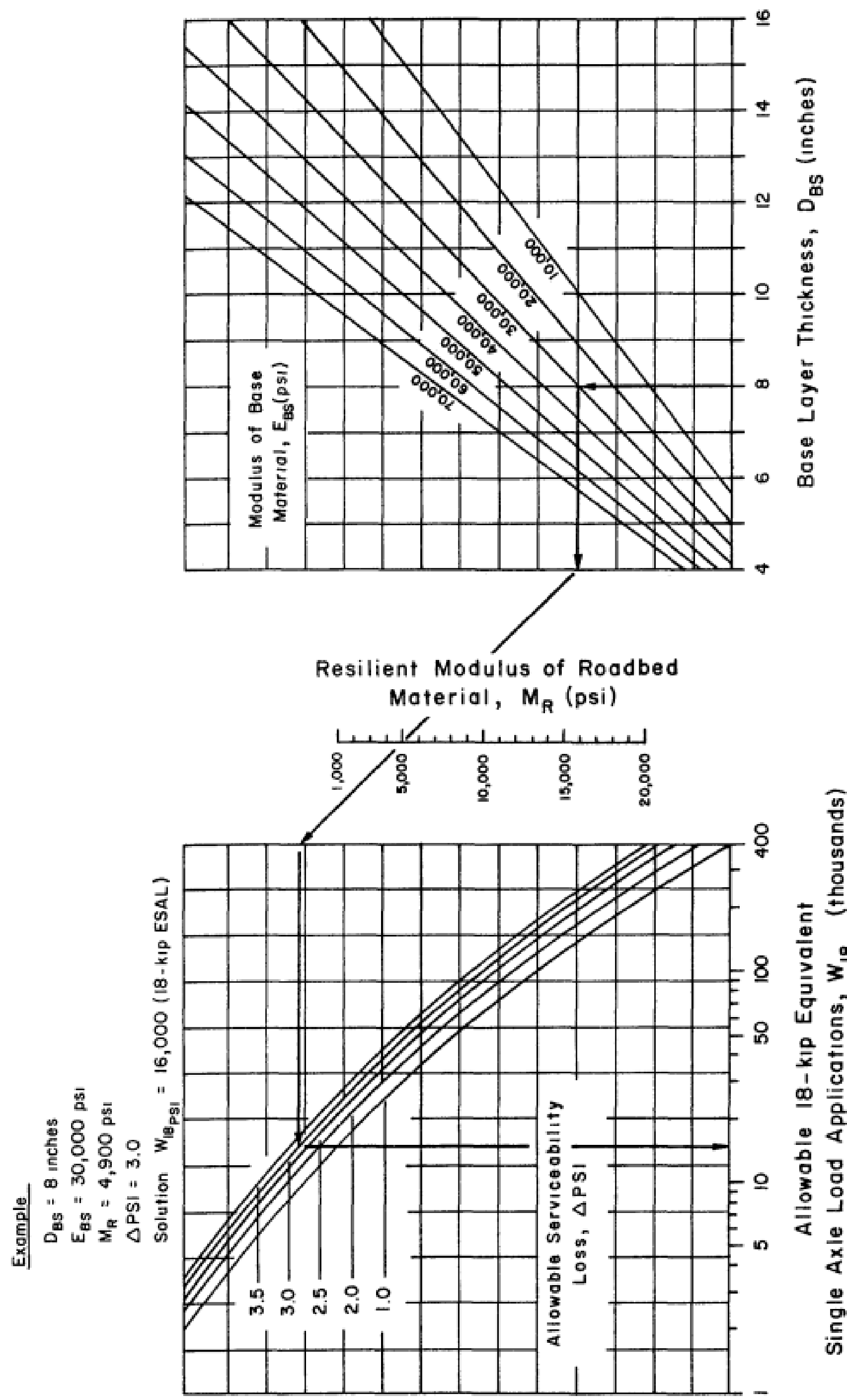

Figure 39. Design chart for aggregate-Surfaced Roads Considering Allowable Serviceability Loss (AASHTO, 1993)

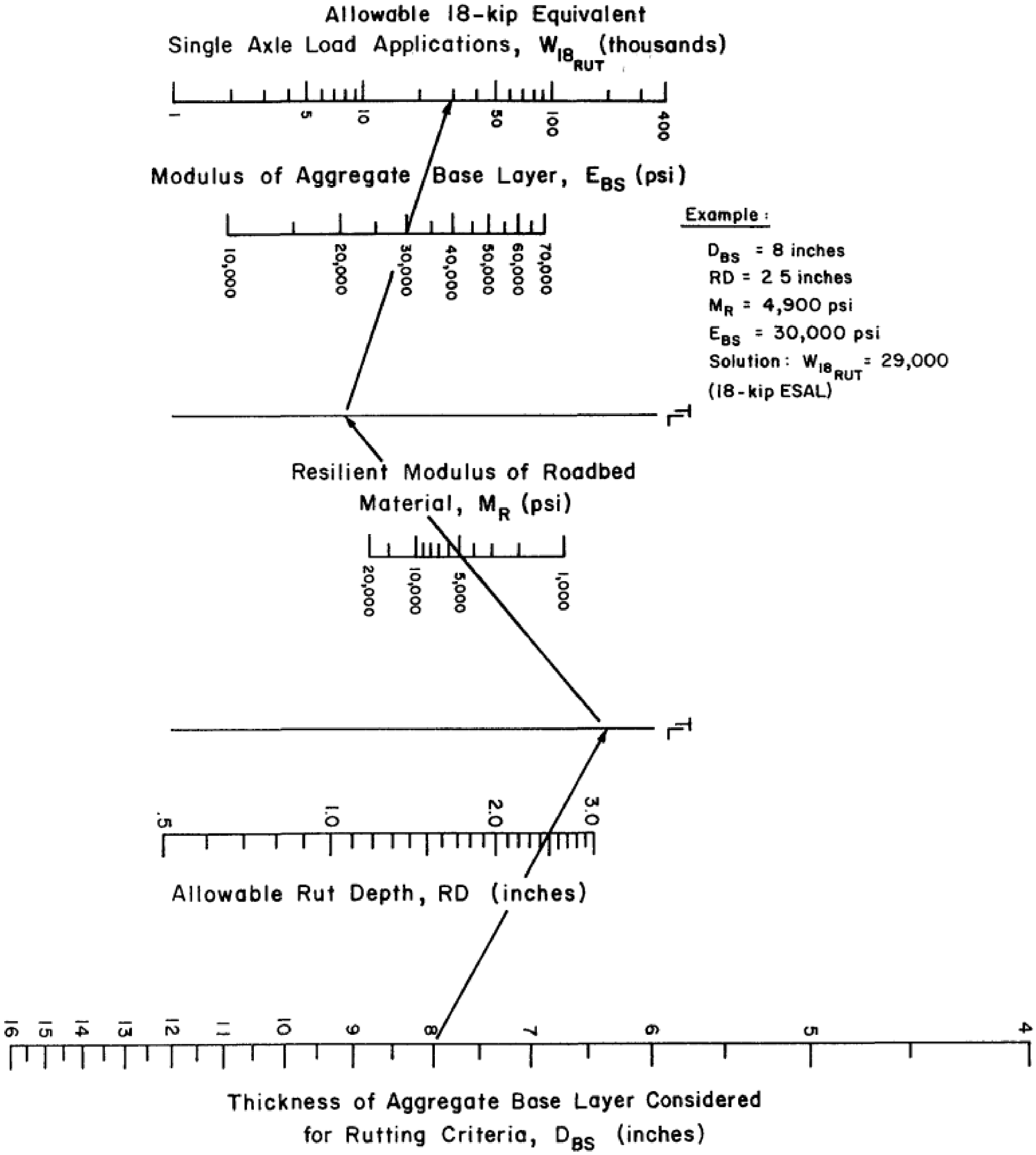

Figure 40. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting (AASHTO, 1993)

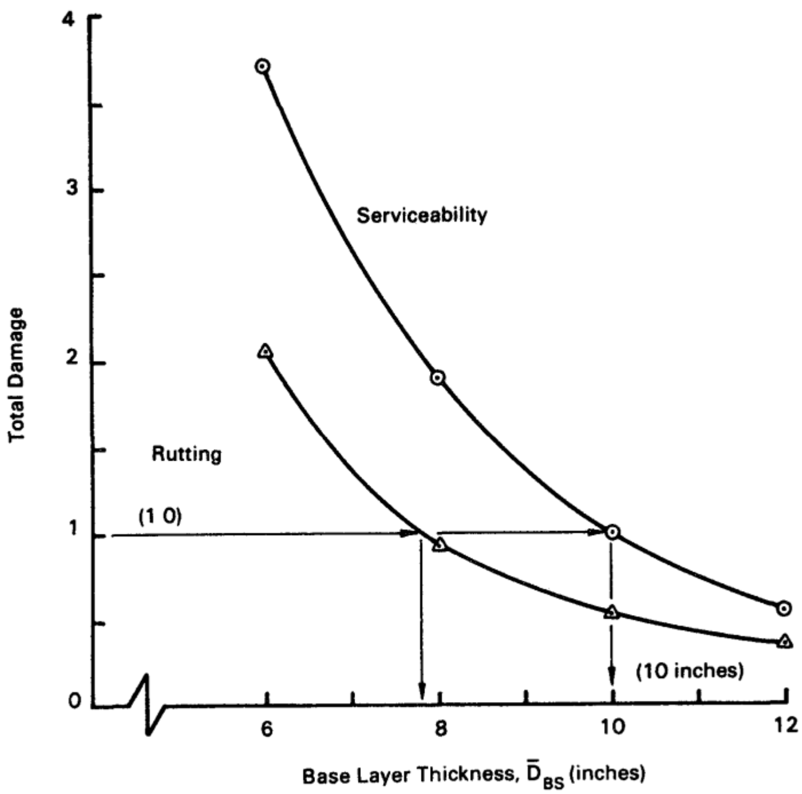

Figure 41. Example Growth of Total Damage Versus Base Layer Thickness for Both Serviceability and Rutting Criteria. (AASHTO, 1993)

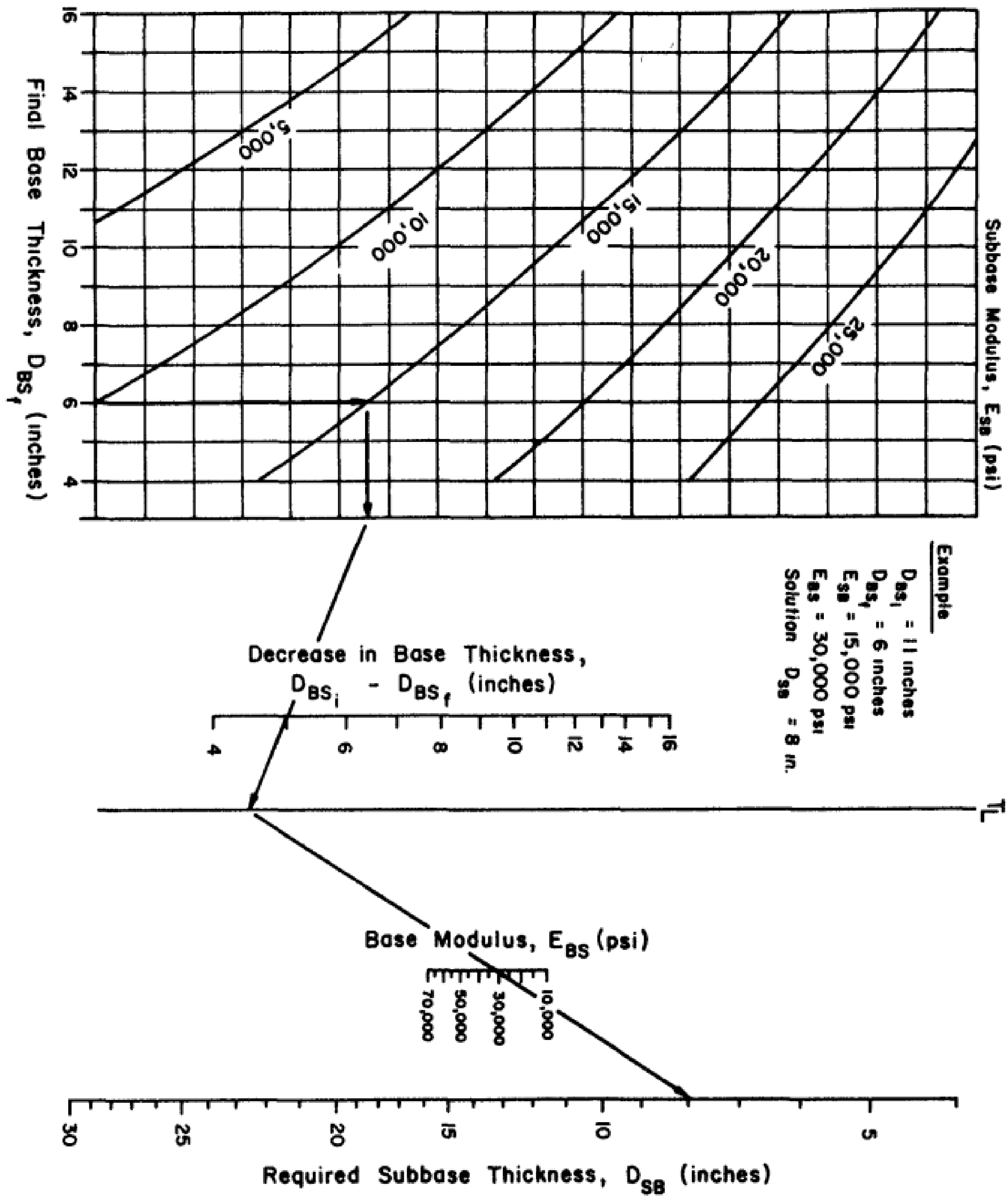

Figure 42. Chart to convert a Portion of the aggregate Base Layer Thickness to an Equivalent Thickness of Subbase (AASHTO, 1993)

Design Catalog

When there is not enough specific information available, this approach is used. When a more intricate design technique is not feasible, Table 6 provides a catalog of aggregate base layer thicknesses that may be employed for the construction of low-volume roads. (AASHTO, 1993)

Six diverse climatic and environmental traits of U.S. geographic regions are depicted in Figure 43.

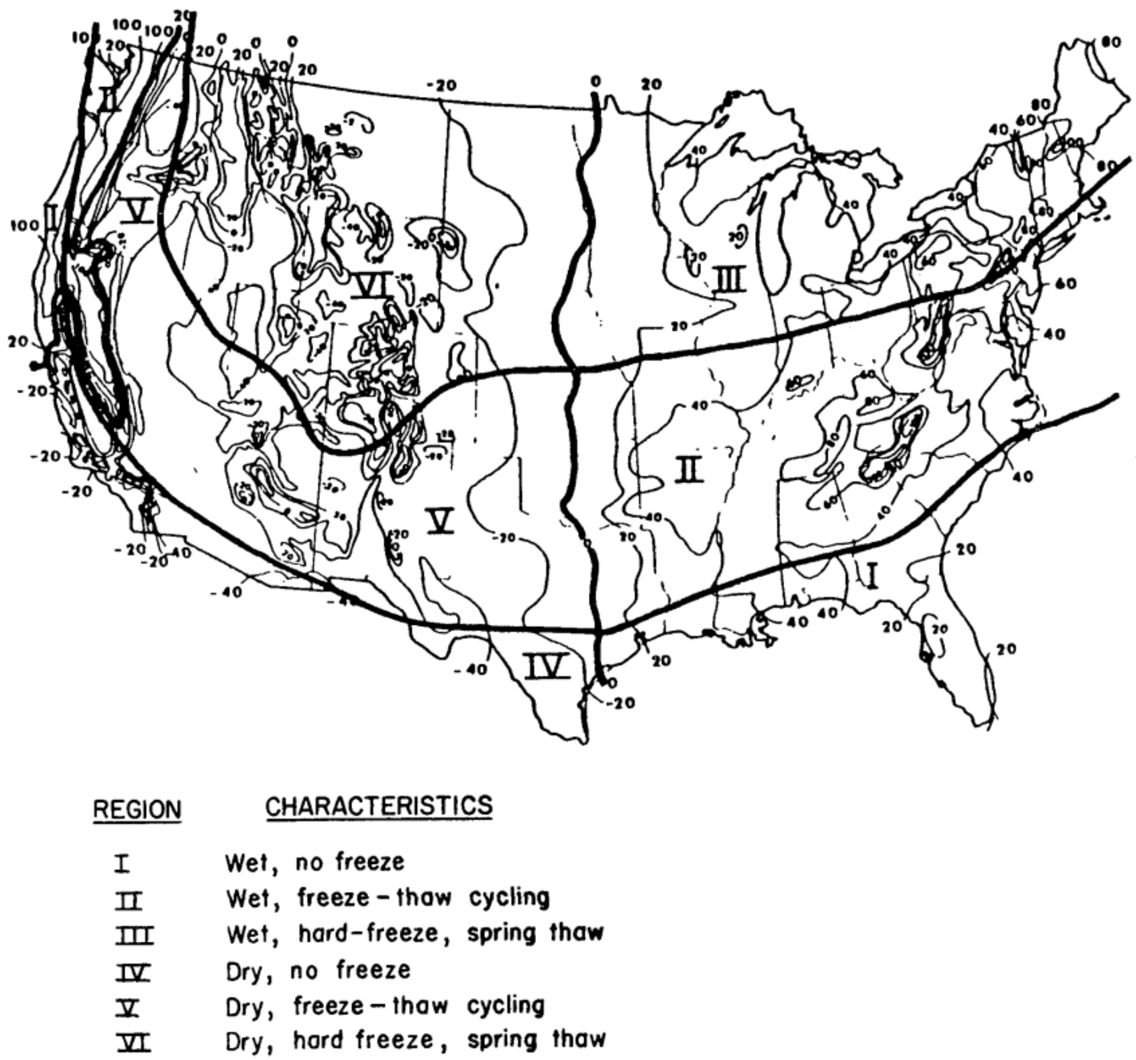

Figure 43. The six Climatic Regions in the United States (AASHTO, 1993)

Table 6. Aggregate Surfaced Road Design Catalog: Recommended Aggregate Base Thickness in inches for the Six U.S. Climatic Regions, Five Relative Qualities of Roadbed Soil and Three Levels of Traffic (AASHTO, 1993)

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very good	High	8*	10	15	7	9	15
	Medium	6	8	11	5	7	11
	Low	4	4	6	4	4	6
Good	High	11	12	17	10	11	17
	Medium	8	9	12	7	9	12
	Low	4	5	7	4	5	7
Fair	High	13	14	17	12	13	17
	Medium	11	11	12	10	10	12
	Low	6	6	7	5	5	7
Poor	High	**	**	**	**	**	**
	Medium	**	**	**	15	15	**
	Low	9	10	9	8	8	9
Very poor	High	**	**	**	**	**	**
	Medium	**	**	**	**	**	**
	Low	11	11	10	8	8	9

*Thickness of aggregate base required (in inches)

**Higher type pavement design recommended

Based on specific ranges of 18-kip EASL applications at traffic levels, the thicknesses presented in Table 6 are calculated.

High 60,000 to 100,000

Medium 30,000 to 60,000

Low 10,000 to 30,000

4.1.3. India (IRC)

In accordance with the Indian Road Congress, the majority of rural roads being designed and built are aimed at linking farms to markets. And it is clear that there are notable fluctuations in traffic volumes depending on whether it is harvesting season or not.

Figure 44 depicts a typical cross-section of an unpaved road as per the (IRC, 2008).

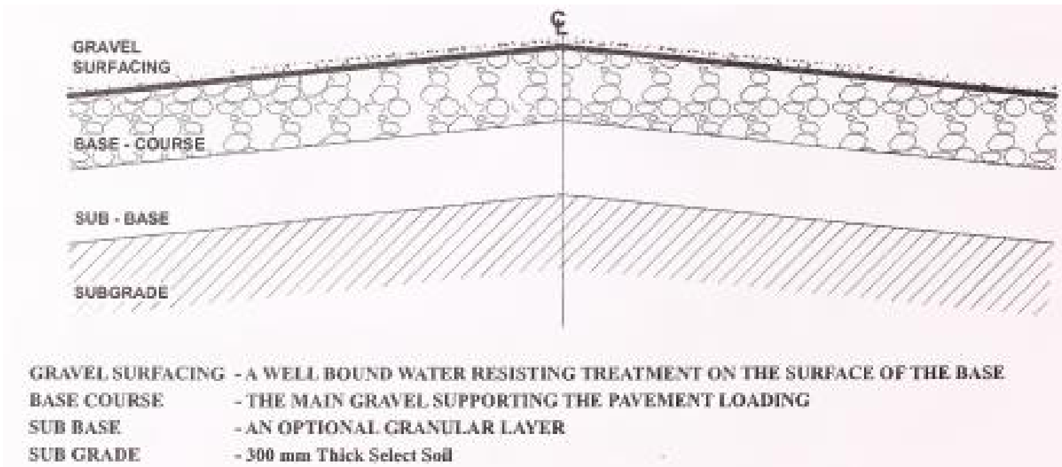

Figure 44. Typical Cross-Section of a Gravel Road Pavement

The following parameters are part of the design process as per (IRC, 2008).

- Traffic as measured by the volume of standard axles (80 kN) to be transported over the design life and
- CBR strength sub- grade.

<u>Pavement Design</u>

The following factors should be taken into account when estimating the thickness of gravel/aggregate-surface roads, per (IRC, 2008):

- The initial serviceability index is taken to be 4.0 shortly before opening the road to traffic, and the terminal serviceability is taken to be 2.0 when rehabilitation will be necessary, with or without the provision of an overlay. This limits the serviceability loss over the design life to 2.0.
- The maximum allowed depth of rutting under a 3 m straight edge is 50mm.

<u>Computation of design traffic</u>

Given that the majority of rural roads function as farm-to-market routes, there are noticeable fluctuations in traffic volumes between the non-harvesting and harvesting seasons. (IRC, 2008). The straightforward formula shown below can be used to calculate the Annual Average Daily Traffic (AADT), properly accounting for seasonal variations: -

$$AADT = T + \frac{1.2(n * T) * t}{365} \quad \textbf{\textit{Eq 5}}$$

Where,

T = Number of vehicles per day during the lean non-harvesting season, including both motorized and non-motorized vehicles but excluding all two-wheelers.

t = is the number of days in a harvesting season.

n = The number of times the daily average number of cars increases during the busiest harvesting season over and above traffic during the slow season.

Local authorities frequently have the T and n desired data for the formula above.

According to (IRC, 2008) in the event of a new road, an approximation of the traffic that will use the route should be made, based on the number of communities and their population served along the road alignment as well as other socio-economic criteria. The projected traffic on the new proposed road should be assessed based on the population served and the agricultural produce that has to be moved. Traffic counts can be performed on an existing road nearby under comparable conditions. Due consideration should be given to the traffic that will be "Diverted" and "Generated" as a result of the construction of the planned road, the land use of the area serviced, the potential for increased traffic, and the design.

<u>Traffic parameters for gravel base thickness design</u>

Only commercial vehicles with a gross loaded weight of three tons or more and their axle loading are accounted for pavement design. (IRC, 2008) These consist of

- Heavy Commercial Vehicles (HCV), such as full-size buses and large trucks.
- Medium-Heavy Commercial Vehicles (MCV), which include pick-up vans, tractor-trailers, light trucks, and minibuses.

The design traffic is calculated using the total number of standard axles of 80 kN that will be transported during the course of the road's design life.

$$N = T * 365 * \frac{[(1 + 0.01r)^n - 1]}{0.01} * L * F \qquad \textbf{\textit{Eq 6}}$$

N = Cumulative number of standard axles for design of gravel base thickness

T = Number of commercial vehicles per day in the year of opening the road

r = Annual growth rate of traffic

L = Lane Distribution Factor = 1 for single lane/ intermediate lane

n = Design Life in years

F = Vehicle Damage Factor

The Vehicle Damage Factor (F), defined by the (IRC, 2008), is the "Equivalent number of standard axles per commercial vehicle." Although the factor "F" is derived from real axle load measurements on the existing roads, the size of the project and the level of traffic on rural roads may not make an axle load survey necessary.

As a result, it is recommended to use the indicative F values for HCV and MCV of 2.5 and 0.33, respectively, when the vehicle is completely loaded, and 0.3 and 0.02, respectively, when it is not fully loaded. Information about commercial vehicles that are loaded and unloaded may not always be available. When actual data is not available, (IRC, 2008) recommends the following VDF values of 2 and 0,3 for HCV and MCV respectively:

(IRC, 2008) states, the potential for generating traffic in a given location determines the rate of traffic growth. If there isn't any trustworthy information available on this subject, a 6% growth rate (r) will be used for design. From the year that actual field traffic surveys were conducted to the year that the route was opened to traffic, the same yearly increase rate of traffic must be accounted for.

A fair estimate of design traffic in terms of cumulative standard axles during the design life of a gravel road shall be obtained as shown in Table 7 in the absence of suitable data for calculating design traffic parameters in respect of commercial vehicles (both motorized and non-motorized) and proportions of HCV and MCV. (IRC, 2008)

Table 7. Reasonable estimate of design traffic in terms of cumulative ESAL (IRC, 2008)

AADT	Cumulative EASL Application for 10- year Design Life
50	10,000
100	50,000
150	75,000
200	100,000

Traffic categories: For the purpose of gravel road surface design, the traffic has been categorized into the following three groups:

Table 8. Traffic Categories

Traffic Category	Cumulative EASL Applications (N)
T_1	10,000-30,000
T_2	30,000-60,000
T_3	60,000-100,000

<u>Sub grade Strength</u>

The subgrade strength needs to be assessed in terms of CBR value for new road pavement design.

The CBR tests should be performed on representative samples of subgrade soil that have been static compacted to 100% Standard Proctor dry density. The samples should also be tested at a moisture content that corresponds to the wettest moisture condition that is likely to exist in the subgrade throughout its service life. The test results from a set of three specimens should be reported as an average. A test value average generated from 6 specimens should be used if there are significant variances in the test values from the set of 3 specimens. (IRC, 2008)

By doing CBR tests on 4-day-soaked samples in the lab, one can ascertain the soaked CBR value of the subgrade. In the absence of a competent testing facility, the typical presumptive values listed in Table 9 can be used.

Table 9. Typical Presumptive Design CBR Values

Description of Sub-grade Soil	Typical Soaked CBR Value (%)
Highly Plastic Clays and Slits	* 2-3
Silty Clays and Sandy Clays	4-5
Clayey Sands and Silty Sands	6-10

*On expansive clays, a straightforward Free Swelling Index test should be conducted.

(IRC, 2008) classifies the sub-grade strength as expressed in Table 10 in order to use the design chart shown in Table 11.

Table 10. Subgrade strength classes

Quality of Sub-grade	Class	Range (CBR %)
Very Poor *	S1	2
Poor	S2	3-4
Fair	S3	5-6
Good	S4	7-9
Very Good	S5	10-15

* If it is proven to be economically viable to replace 300 mm of subgrade with acceptable soil when the CBR of the subgrade soil is less than 2, the pavement should then be planned to use the CBR value of the improved subgrade. As an alternative, a modified soil capping layer with a CBR of at least 10 and a thickness of at least 100 mm should be provided.

Table 11 displays the required gravel base thickness for the three traffic categories of cumulative ESAL repetitions of 10,000–30,000 (T1), 30,000–60,000 (T2), and 60,000–100,000 (T2) for each of the five subgrade strength classes (S1, S2, S3, S4, and S5) (T3). (IRC, 2008)

Table 11. Pavement Design Catalog (IRC, 2008)

CBR% \ Cum. EASL	10000-30000	30000-60000	60000-100000
CBR =2 (very poor)	200 100 (mm)	75 150 100 (mm)	75 100 100 100 (mm)
CBR =3-4 (poor)	200 (mm)	275 (mm)	75 100 150 (mm)
CBR =5-6 (fair)	175 (mm)	250 (mm)	275 (mm)
CBR =7-9 (Good)	150 (mm)	175 (mm)	225 (mm)
CBR =10-15 (very good)	125 (mm)	150 (mm)	175 (mm)

Gravel base
Granular sub-base
Improved sub-grade
Bituminous surface treated
Base of Gravel

Figure 45 illustrates a chart that translates a portion of the aggregate base layer thickness into an equivalent thickness of subbase with a middle CBR value between the base and subgrade.

However, it must be made sure that a gravel base with a thickness of at least 100 mm is constantly present. The minimum soaked CBR of 80 for the gravel base material is frequently thought of as a supplementary requirement, in addition to the grading specifications provided for the gravel base and surfacing. It must be remembered that if either of the two requirements—grading or soaking CBR value—is not met, the gravel foundation material may not be able to survive the design cumulative ESAL treatments over the design life, necessitating regravelling earlier than intended. It should be noted that the necessary regular and periodic maintenance inputs must be provided for the recommended designs to function satisfactorily during the course of the design life. (IRC, 2008)

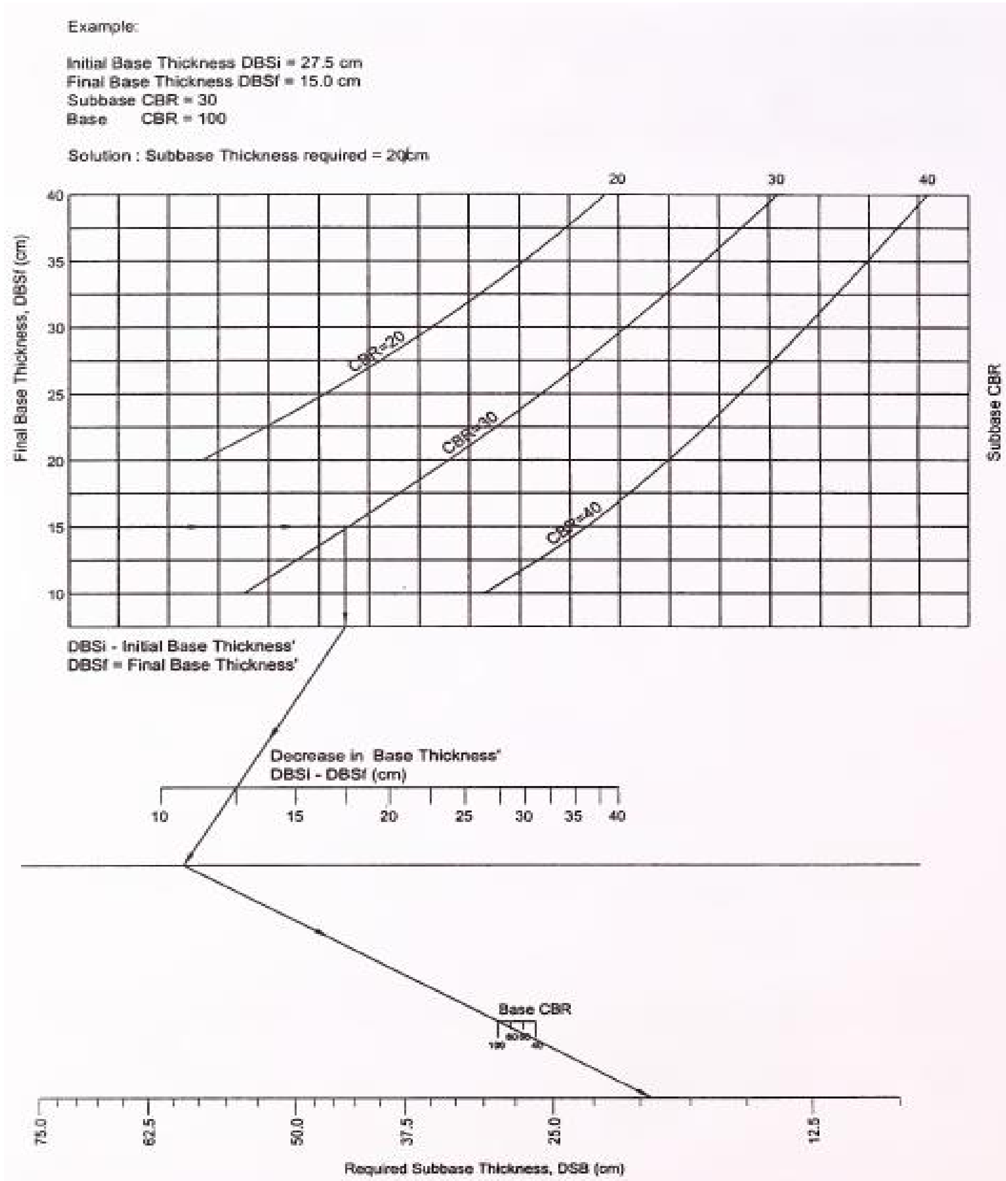

Figure 45. Chart to convert a portion of the Gravel/Soil- Aggregate Base Layer Thickness to Equivalent of Subbase Adapted from AASHTO, 1993)

Pavement Materials

According to the MORD (Ministry of Rural Development) Specifications, Table 12 and Table 13 show the required gradations of gravel/soil aggregate for use in the base and surface courses of a gravel road, respectively. These gradations are advised in the event that the gravel is surface-dressed in bituminous material or chip-sealed. (IRC, 2008)

Table 12. Grading Requirements for Base Course (IRC, 2008)

Sieve Size	Percent by Mass Passing IS Sieve Grading Designation		
	A	B	C
53mm	100		
37.5 mm	97-100	100	
26.5 mm		97-100	100
19 mm	67-81		97-100
9.5 mm		56-70	67-79
4.75 mm	33-47	39-53	47-59
425μm	10-19	12-21	12-21
75 μm	4-8	4-8	4-8

Table 13. Grading Requirements for Surface Course (IRC, 2008)

Sieve Size	Percent by mass Passing Designated Sieve
26.5 mm	100
19 mm	97-100
4.75 mm	41-71
425μm	12-28
75 μm	9-16

For strength and longevity, gravel for base courses should have a very low proportion of fine materials (silt and clay) and a relatively bigger top-sized aggregate. To easily shed off water that falls on the gravel road's surface, surface gravel should have a significantly larger percentage of fines (silt and clay) and a relatively smaller top sized aggregate. The crushing of rounded stones when they are locally accessible is always advised since the fragmented stones adhere to the surface of a gravel road considerably better than rounded stones. (IRC, 2008)

The percentages of gravel, sand, and fines (silt and clay) in grades A, B, and C in Table 14 are as follows, according to (IRC, 2008)

Table 14. Percentage of Gravel, sand and fines (Silt and Clay) in Gradings A, B, C (IRC, 2008)

	Grading A	Grading B	Grading C
Gravel	53-67 %	47-61 %	41-53 %
Sand	25-43 %	31-49 %	39-55 %
Silt and Clay	4-8 %	4-8 %	4-8 %

(IRC, 2008) recommends when a single naturally occurring material falls short of any of the needed gradings, "processing"—the blending of two or more materials—must be used to produce the desired grading.

Table 15. Requirements when the requirements with respect to Table 12 grading is not met

(a) Base Course	
Percent retained on IS 4.75 mm sieve and passing 80 mm in size (Percent Gravel)	: 50 - 70%
Percent retained on IS Sieve 75 micron, and: 25 - 40% passing IS Sieve 4.75 mm (Percent Sand)	25 - 40%
Percent passing IS Sieve 75 micron: Absolute max. 10%) (Percent Slit and Clay) Desirable max.	Absolute max. 10%) Desirable max. 5%)
(b) wearing Course/Surface course	
Percent retained on IS 4.75 mm sieve and passing 80 mm in size (Percent Gravel)	: 50 - 70%
Percent retained on IS Sieve 75 micron, and passing IS Sieve 4.75 mm (Percent Sand)	25 - 40%
Percent passing IS Sieve 75 micron. (Percent Slit and Clay)	: 8 - 15%

4.1.4. South African Design Manual (Draft TRH 20)

According to (TRH 20, 1990), southern Africa currently employs no scientific structural design method for unpaved roads. Models that forecast the rut depth from material qualities, traffic, and surface thickness were developed at the Waterways Experiment Station (Barber et al., 1978), and they are now part of the Maintenance and Design System (MDS) (Visser, 1981). Although Visser (1981) transposed the variables in the models to forecast the design thickness, design thickness models were regrettably not included. For subgrade CBR values greater than 5 (at Proctor compaction) with 150 mm of surfacing material (Proctor CBR greater than 30), the transposed model produced a more realistic cover thickness than the models developed earlier, showing that approximately 10 000 truck repetitions would be needed to produce a rut of 75 mm.

For typical rural roads in southern Africa, the 75 mm-deep rut failure criterion is deemed excessive. Even though the rut would typically be eliminated by routine grader maintenance, a sizable percentage of it would take the form of subgrade deformation, which would result in a loss of wearing course material. (TRH 20, 1990)

A structural design process for application in southern African environments has been established based on this idea and thorough local observations and measurements. Therefore, the suggested design thickness (T in mm) for imported gravel wearing courses is:

$$T = t + \left(1 + \frac{C_t}{100}\right) x (GL_p x L_d) \qquad \textbf{\textit{Eq 7}}$$

Where,

t = Required thickness for subgrade protection at a minimum (mm)

C_t = compaction induced by traffic (%)

GL_p = estimated annual gravel loss (mm)

L_d = design life of the road or frequency of regraveling (years)

(TRH 20, 1990) suggests that, in general, the minimum thickness needed for subgrade protection can be left out of the formula above, especially for subgrade materials with a field CBR higher than 5%. The Dynamic Cone Penetrometer (DCP) makes it simple to determine the value for the in situ CBR (Kleyn, 1984). For most subgrade soils, a DCP penetration of more than 32 mm per blow suggests that the CBR is 5% or less. Due to the deep-water tables, Southern African subgrades mainly consist of sandy materials that are unaffected by moisture.

It is advised to employ a nominal 50 mm of wearing course material (t in the equation) for subgrade protection for subgrades with field CBR values (i.e. at estimated in situ density and moisture content) of less than 5% unless further research suggests otherwise. Regular grader maintenance on unpaved roads typically corrects any subgrade deformation on the surface. Naturally, it is advised that a formation with a minimum thickness of 300 mm be built out of material with a satisfactory CBR strength (=5%), which also acts as subgrade protection. (TRH 20, 1990)

Geotextiles and Geogrids have been successfully used to reinforce unpaved roads across extremely weak subgrades as a result of recent breakthroughs in the field (Giroud and Noiray, 1981; Giroud et al, 1984; Hausmann, 1987). These are not further covered in this study, although they can be thought about in unique situations when very weak subgrades occur infrequently (CBR always less than roughly 3%). Due to traffic compaction, wearing courses that have been compacted with a nominal number of grid-roller passes may lose up to 30% of the designed thickness in a short amount of time (Paige-Green, 1989). In order to prevent pavement thickness loss due to traffic compaction, it is crucial to maintain enough compaction or account for it in the thickness design. (TRH 20, 1990)

(TRH 20, 1990) uses the following approximate estimates of the potential traffic compaction:

Compaction during construction	Moisture content during construction	Potential loss of gravel thickness
3 passes of a grid roller	About OMC (5%)	10%
3 passes of a grid roller	Dry OMC (>5%)	20%
3 passes of a pneumatic tyred roller	About OMC (5%)	5%

With a high degree of confidence, the following factors can be used to forecast the annual gravel loss (AGL stated in mm) to within 11 mm per year:

$$AGL = 3.65[ADT(0.059 + 0.0027N - 0.0006\,P26) - 0.367N - 0.0014\,PF + 0.0474\,P26] \qquad \textbf{\textit{Eq 8}}$$

Where,

ADT = average daily traffic

N = Weinert N-value

P26 = Percentage passing the 26.5mm sieve*

PF = product of plastic limit and percent passing 0.075mm sieve*

* grading analysis carried out according to the TMH 1 (Technical Methods for Highways)

<u>Weinert N-value</u>

Weinert [1980] asserts that environmental factors frequently have a bigger impact on a road's lifespan and functionality than is generally recognized. The formulation of Weinert's climatic N-value, which is defined by Eq 9, resulted from observed variances in the performance of weathered dolerite used in road construction in various regions of South Africa: (11DBMC, 2008)

$$N = \frac{12E_J}{P_a} \qquad \textbf{\textit{Eq 9}}$$

where P_a is the total yearly precipitation and E_J is the calculated evaporation from a shallow free water surface in January (the warmest month).

As per weathering properties of the naturally occurring materials used to create roads, southern Africa is regarded to be almost unique in the world. The widespread occurrence of late palacozoic (Karoo) and younger basic crystalline rocks and argillaceous rocks in Southern Africa is well known (Weinert, 1980).

It must be considered that these fundamental rocks, as seen in Figure 46, include minerals of the Smectite group, notably Montmorillonite, as a result of decomposition when combined with warm temperatures and seasonal rain associated with Weinert n-values of less than n=5. (Jordaan et al, 2017)

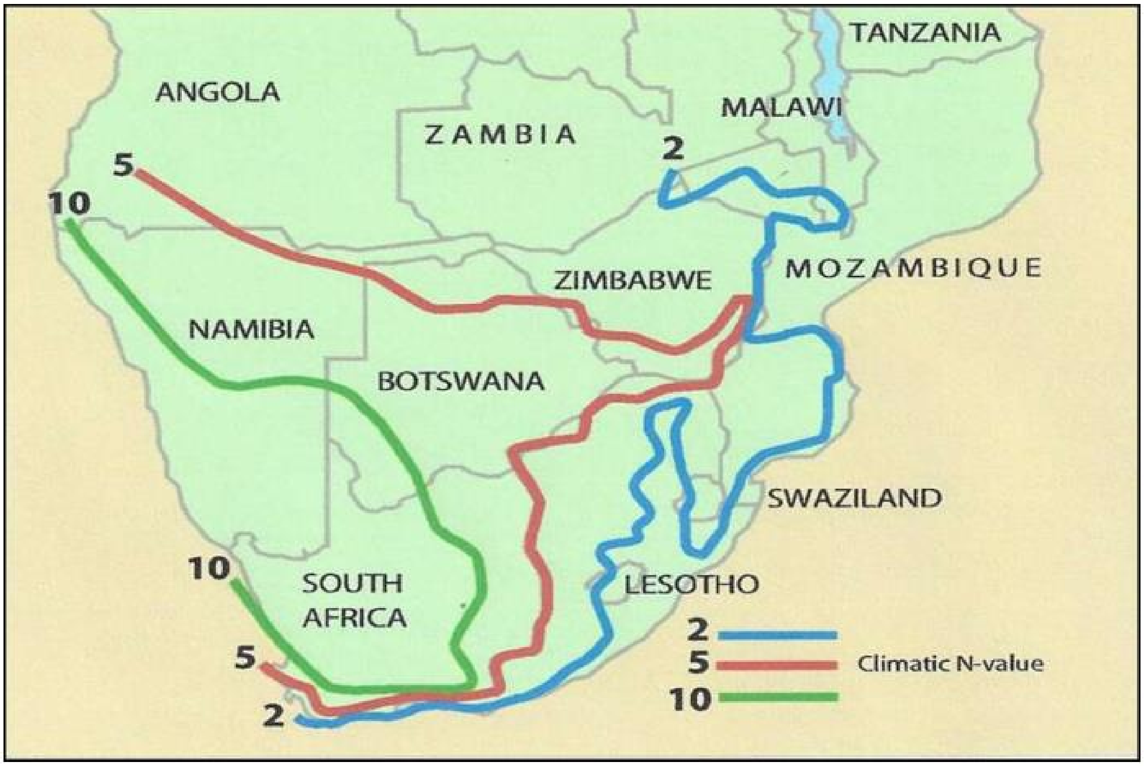

Figure 46. Weinert N-values for southern Africa (Weinert, 1980)

<u>Pavement Materials</u>

Regarding the use of materials for the South African unpaved road wearing courses Performance-related requirements have been created for the southern African environment (Paige-Green, 1989a). These are based on earlier studies on the subject as well as the sampling, testing, and monitoring of the performance of 110 sections of unpaved road in southern Africa over a period of more than three years (Paige-Green, 1989a). During the trial, it was discovered that the durability of the material was not significant for unpaved roads. However, the 5-cycle wet-dry test should be used to examine mud rocks because they may be susceptible to rapid disintegration in some places (Venter, 1989). Indicators of extremely soft or hard material that may break down under traffic or that will not break down under a grid-roller, respectively, may be found using other tests, such as the Los Angeles Abrasion. (TRH 20, 1990)

According to (TRH 20, 1990), the following specifications for materials for unpaved rural roads are recommended (Table 16)

Table 16. Recommended material specifications for unpaved rural roads

Maximum size	37.5 mm
Oversize index (Io)[a]	=5 percent
Shrinkage product (Sp)[b]	100-365 (max of 240 preferable)
Grading coefficient (Gc)[c]	16 – 34
CBR: = 15 at = 95 per cent Mod AASHO compaction and OMC[d]	

[a] IO = Oversize Index (per cent retained on 37,5 mm sieve)

[b] SP = Linear shrinkage x per cent passing 0,425 mm sieve

[c] GC =(Per cent passing 26,5 mm – per cent passing 2,0 mm) x per cent passing 4,75 mm/100

[d] tested immediately after compaction.

4.1.5. Mechanistic-Empirical approach

The amount of information regarding the design of unpaved roads is relatively scarce in comparison to paved roads. Unpaved roads, characterized by low traffic volume (less than 400 heavy vehicles per day), primarily rely on experiential knowledge and empirical design methodologies for their design. However, the drawback of using empirical methods is their effectiveness is confined to the specific environment and conditions in which they were originally devised. When applied in different settings, these methods can result in excessive design work and wastage of materials. (Mickaël Le Vern et al, 2016)

Mechanistic methods or procedures are commonly referred to as the techniques used to translate analytical calculations of pavement reaction into performance indicators, particularly pertaining to physical distress. (AASHTO, 1993) One of the essential components of a pavement mechanistic-empirical design process is the empirical transfer function, which describes relationships between elastic strain and the quantity of load repetitions. The design engineer can calculate the strains occurring in the structure when subjected to a particular load configuration using a trial pavement construction and known or assumed material mechanical parameters. The designer can thus determine the associated number of loads before failure for a particular damage mechanism using an empirical transfer function. (J.-P. Bilodeau, 2017)

Mechanistic-empirical design approaches are being developed by researchers who believe they will model pavements more accurately than traditional empirical equations because they are based on well-established theory. Successfully applying mechanistic approaches can have three main advantages: increased design reliability; the capacity to anticipate particular types of distress; and the capacity to extrapolate from scarce field and laboratory data. (AASHTO, 1993)

Currently, the Forest Service employs a number of aggregate surfacing design techniques. The goal is to establish the criteria that should allow for the selection of the ideal granular layer thicknesses in relation to the subgrade soil and the number of load applications that the road will support over the course of its lifetime. (Margot T, n.d) By definition, these methodologies offer suitable designs in the environment in which they were created, but they lose validity when applied to different environments. Mechanistic-empirical (ME) approaches, which enable the fusion of physics with empirical findings, are therefore currently advised. (J.-P. Bilodeau, 2017)

<u>USFS Surfacing Design and Management System (SDMS)</u>

The performance model for rutting and the aggregate loss model for roads with aggregate surfaces were derived using data from the Corps of Engineers, as documented by V.C. Barber in 1978. It is mentioned that all the data points from the Corps' 1978 report were utilized in formulating the equation for manual design. D.R. Luhr in 1984 provided the most comprehensive documentation of the 5-year (1978-1983) development process for SDMS, indicating its successful operation by 1978. Furthermore, there is an implicit suggestion of a certain level of skepticism regarding the reliability of the publicly available Corps regression equations, as mentioned by (Margot T, n.d) The rutting formula is:

$$W_{18R} = 0.1044 * RUT^{2,575} * \log_{10} THICK^{5.155} * \left(\frac{E_1}{1,800}\right)^{3,434} * \left(\frac{E_2}{1,800}\right)^{1,048} \qquad \textbf{\textit{Eq 10}}$$

Where,

W_{18R}= no. of applications of 18-kip equivalent single axle loads (ESALS).

RUT = rut depth (in.)

THICK = thickness of aggregate surface (in.).

E_1 = modulus of aggregate surface (psi), and

E_2= modulus of roadbed (psi).

At the Low-Volume Road Conference that took place in Ames, Iowa in 1979, a collective decision was reached to integrate the algorithm into the design procedure as a manual approach. As a result, endeavors were undertaken to computerize the entire design process, which reached completion in 1983. During the subsequent five-year period, significant progress in computer technology and revisions to established design methodologies, notably exemplified by the 1986 AASHTO guide, prompted the Forest Service to conduct a reevaluation of the prevailing circumstances. In 1988, the Surfacing Design Evaluation Report was meticulously prepared for internal utilization and discourse, wherein it put forth a series of recommendations for consideration. (Margot T, n.d)

- ✓ Embrace the updated AASHTO design manual from 1986 and its accompanying software (DNPS86/PC) for roads with bituminous surfaces.
- ✓ Create a comprehensive design manual for roads with aggregate surfaces and roads without any surfacing, utilizing available technology.
- ✓ Integrate the notion of multiple user levels into the design procedure, wherein these levels indicate varying degrees of operational complexity and design variability.

USFS Region 8 Analysis Road Materials System (ARMS):

ARMS is a method for designing road surfaces that was developed in Region 8 of the Forest Service. It was created to address the need for an aggregate management system that could be applied across the entire region, considering the presence of numerous isolated, low-cost, and short road segments. The design process of ARMS relies on existing geological data, such as state geological maps, to determine soil properties used in surface design equations. Given the extensive oil and gas exploration in Region 8, detailed geological maps are readily available.

These maps classify rock formations based on similar petrographic characteristics, which are then correlated with the engineering properties of soils. Thus, the maps serve as a geotechnical database for the implementation of ARMS. Furthermore, regression equations derived from laboratory data correlation are utilized alongside traditional sampling and testing techniques.

The thickness design for pavement is primarily derived from the equations provided in the SDMS manual. This design approach incorporates two key criteria, namely rut depth and serviceability loss. The design equation specifically focusing on rut depth is employed to determine the appropriate thickness requirement for the pavement. (Eq 11) (Margot T, n.d)

$$W_{18RUT} = 64.51 * (t_p)^{-1.4665} * [3\ (TSI)^{-0.5}]^{2.575} * \log t^{5.155} * \left(\frac{E_B}{1800}\right)^{3,434} * \left(\frac{E_{SG}}{1800}\right)^{1,048} \quad \textbf{Eq 11}$$

Where,

W_{18RUT} = no. of applications of 18-kip ESALs.

t_p = tire pressure (psi),

TSI = terminal serviceability index.

t = thickness of aggregate surface (in.).

E_B = resilient modulus of aggregate surface (psi). and

E_{SG} = resilient modulus of subgrade (psi).

Error! Reference source not found. exhibits striking similarity to Eq 10 found in the SDMS manual for rutting. Notably, the last three terms in both equations are identical. However, Eq 10's rut depth component has been substituted with an alternative expression that incorporates the terminal serviceability index instead. Region 8 has devised Table 17, which establishes a relationship between rut depth and the Traffic Service Level (TSL). By consulting Table 17**,** the relevant TSI and its associated permissible rut depth can be determined and subsequently inputted into **Error! Reference source not found.**.

Table 17. Serviceability Index and Rut Depth**)** (Margot T, n.d)

TSL	PSI	TSI	MRD
A	4.7	2.0	2.1
B	4.2	2.5	2.4
C	3.5	0.5	4.2
D	3.5	0.25	6.0

Where,

TSL = Traffic Surface Levels

PSI = Present Serviceability Index

TSI = Terminal Serviceability Index

MRD = Maximum allowable rut depth

Chapter 5

5. Distress Impact on Design: Critical Analysis

This chapter explains how the various types of distresses mentioned are accounted for when determining the thickness of unpaved roads in the structural design process. Some are factored in as direct contributions because they have a significant impact on the design, such as gravel loss and rutting, which both significantly contribute to the deformation of the road.

5.1. Dust

One of the earliest indications of the loss of fine particles and the deterioration of an unsealed road surface is dust. It shows up as dust production and aggregate exposure, which produces a rough, noisy, and coarse surface. Dust is a result of loosening of pavement materials, which results in the loss of tiny particles (less than 0.425 mm), as well as disruption of the wearing course brought on by traffic and environmental factors. (ARRB, 2020) Its entry into the design processes of the various manuals is briefly covered in the sub-section that follows.

The Australian approach does not directly reflect on the distress dust during the design phase, but it undoubtedly does so when choosing the appropriate wearing course materials. The best unsealed wearing course material to choose is a well-graded gravel-sand mixture with a suitable amount of clayey fines, whereas less ideal wearing course materials include fine-graded silts and silty sands lacking gravel-sized particles or gravel and sands that are low in fines. Adequate compaction during the construction phase as well is crucial because dust is created by materials that lack strength or cohesiveness. Therefore, at the construction stage, this distress is considered as well. (Austroads, 2009)

Like the (Austroads, 2009), the (IRC, 2008), (AASHTO, 1993), and (TRH 20, 1990) manuals, it works with the surface distress, dust during the construction stage to prevent or decrease its occurrence.

5.2. Ravelling

One of the major problems is the ravelling, which occurs when loose gravel accumulates beneath traffic and poses a threat to both safety and the economy.

In the design and construction phases, the (Austroads, 2009) manual integrates ravelling into account through the selection of the right wearing course material and increased compaction. The maintenance procedure also reflects this distress in the following ways: importing and replacing lost fines, compaction to the optimum moisture content, grading loose materials to the side and respreading when enough moisture and materials are available, and routine smoothing operations.

(AASHTO, 1993) uses Eq 4 to predict and account for the direct loss of gravel caused by traffic and erosion in the structural design. The equation was created using limited information on areas with more than 50% truck traffic. The total loss calculated (in inches) using Eq 4 depends on the number of loaded trucks, expressed in thousands.

Gravel loss isn't specifically accounted for in the structural design by the (IRC, 2008) manual, although its effects may be mitigated by employing the right materials. (IRC, 2008)

Ravelling enters the structural design in South African Design manual (TRH 20, 1990) through Eq 8 and calculates average gravel loss as a function of ADT, Weinert N-value, percentage passing 26.5mm sieve and product of plastic limit, and percentage passing 0.075mm sieve. Weinert N-values are significantly influenced by precipitation and evaporation.

5.3. Corrugation

Unpaved roads include one of the most unsettling flaws, corrugations, which can either be "fixed" or "loose" and cause extreme roughness and poor vehicle directional stability. While fixed corrugations require cutting or perhaps thinning with the grader before the material is distributed again, loose corrugations can simply be eliminated by blading. (TRH 20, 1990)

How this distress is incorporated in the various design manuals covered in the previous chapter is briefly explained in the paragraphs that follow.

Corrugation enters the design of each manual covered in this thesis's earlier chapters through best practices and maintenance procedures like: Adequate compaction at an ideal moisture content, appropriate wearing course material selection, improved grading practices, low-cost "drags" (only a temporary fix and not very effective) and sealing of sections where corrugations persist. (ARRB, 2020: IRC, 2008: TRH 20, 1990)

5.4. <u>Potholes</u>

In the structural design procedures, pothole is not explicitly accounted for in the manuals for AUSTROADS, AASHTO, IRC, and (TRH 20, 1990). However, ponding of water is taken into account by raising crossfalls to 4-6% to allow adequate surface drainage, improved compaction, appropriate material selection, and finally, if they occur, they are maintained by patching.

5.5. <u>Rutting</u>

Although the (ARRB, 2020) does not directly consider rutting when determining the thickness of the upper layer, it is accounted for when choosing improved material properties and during the construction process in the following ways: improved compaction of pavement materials, providing crossfall (4-6%) to reduce water penetrating pavement, improved drainage, and routine blading, stabilization, or modification of basecourse materials.

According to (AASHTO, 1993), one of the design criteria used to read from the rutting-based nomograph Figure 40 and quantify the seasonal damage it causes is the rutting criteria. Using this knowledge, an interpolation of the overall damage (caused by serviceability loss and rutting) is then performed to determine the necessary base layer thickness.

The rutting criteria and serviceability loss are not directly used by (IRC, 2008) to base the structural design of unpaved roads. The computation of traffic design and sub-grade strength, which are the direct factors to establish the design thickness, are not directly affected, but it does place a limit on the amount of rutting that may be done beneath a three-meter straight edge, which cannot be greater than 50 mm.

Rutting is typically seen by (TRH 20, 1990) under local conditions as insignificant in terms of the overall performance of unpaved roads. The firm, free-draining, sand subgrade that covers most of southern Africa and the deep-water tables are likely probable reasons for this. However, the rutting brought on by the ravelling of low-cohesion material under traffic movement is still present throughout southern Africa. Adequate compaction is regarded as one of the best construction procedures in the THR to reduce rutting.

Chapter 6

6. Pavement design for wind and solar farm case studies

The primary objective of this chapter is to provide an in-depth exploration of two case studies, a wind farm and a solar farm. Both case studies entail the meticulous calculation of design traffic, employing the methodologies elucidated in the preceding chapters. Moreover, a comprehensive sensitivity analysis is conducted on pivotal variables to discern their importance and assess the extent of their impact on the ultimate outcome. Furthermore, the chapter encompasses a meticulous computation of the final design thickness, which is a critical aspect of the overall road design process. Additionally, an insightful comparison is made between the different manuals utilized and the renowned AASHTO guidelines. The purpose of this comparison is to determine the level of agreement or divergence in the results obtained when employing the same dataset for analysis, thus shedding light on the reliability and consistency of these methodologies.

6.1. Case Study 1

This case study pertains to a solar farm situated in Spain, which experiences a continental climate with certain influences from its location. The region is characterized by dry summers, cold winters, and limited rainfall, lending it a distinct Mediterranean character. The summers are relatively brief, marked by high temperatures and predominantly clear skies, while the winters are protracted, cold, and accompanied by breezy conditions and intermittent cloud cover. Overall, the area remains arid throughout the year. Temperature fluctuations during different seasons typically range from -1 °C to 31 °C, with rare instances of dropping below -5 °C or exceeding 34 °C.

Based on the geotechnical investigation conducted in the project area, it has been determined that the soil composition consists of sand, clay, and silt. Additionally, the climate of the area is characterized as arid or semi-arid, with limited rainfall. Consequently, the California Bearing Ratio (CBR) of the subgrade is determined to be 10%. As a result, the resilient modulus of the subgrade is computed using the following relationship: (W. Spencer Guthrie, 2015)

$$M_r = 2555(CBR)^{0.64}$$

Where,

M_r= resilient modulus (psi)

CBR = California bearing ratio (%)

$$M_r = 2555(10)^{0.64} = 11\ 152{,}98$$

As a result, the outcome is determined to be 11 152,98 psi.

The primary objective of constructing the access road to the substation is to fulfill two important functions: enabling smooth construction operations and ensuring continuous accessibility for maintenance activities during the substation's operational lifespan. These roadways have been intentionally designed with a width of 4.5 meters to accommodate the necessary traffic and ensure efficient movement throughout the area.

The design period of the project spans a total of 30 years, with a designated duration of 1 year allocated for construction purposes. The remaining 29 years are specifically designated for the ongoing maintenance and operation of the project.

<u>Construction Stage:</u>

During the construction period, the access road experiences a daily traffic volume of 4 trucks, 3 semi-trailers, 6 vans, and 25 light vehicles, operating 6 days a week. To accurately estimate the total traffic flow at the substation, it is necessary to multiply the number of vehicles by two, taking into account both incoming and outgoing traffic. It is important to note that this traffic estimate is considered to be on the conservative side, as there will be many days where heavy vehicle traffic is non-existent.

It is worth emphasizing that heavy vehicles have a significant impact on the design of road pavement thickness. They can cause damage that is between 5000 and 10000 times greater than the damage caused by regular passenger vehicles. As a result, the design considerations for the road pavement must prioritize the influence of heavy vehicles due to the magnitude of potential damage they can inflict.

<u>Operation and maintenance stage:</u>

During the operation and maintenance phase, a consistently low volume of heavy vehicle traffic is anticipated, with no annual fluctuations. It has been taken into account that maintenance staff at the substation will use Type A vehicles for their transportation needs. A weekly flow of 2 semi-trailers, 15 vans, and 20 light vehicles has been considered.

The presence of heavy vehicles will only occur in cases of major repairs or maintenance operations, equipment and material supply, oil transportation, water tank delivery, etc. Additionally, for the calculation of the equivalent number of axles, it is assumed that a transformer replacement will take place during the lifespan of the substation, requiring the inclusion of 2 special vehicles in the calculation.

Considering the entry and exit of vehicles at the substation, the traffic load estimation necessitates doubling the vehicle traffic. Since the width of the substation's roads is 4.5m, a 50% traffic distribution across two lanes is not assumed. Therefore, the design will be carried out for 100% of the calculated traffic.

6.1.1. Calculation of number of Equivalent Axes (ESALS)

This particular section of the study focuses on the calculation of Equivalent Single Axle Load (ESAL) to accurately predict the design traffic, utilizing different manuals as references. The aim is to determine the anticipated traffic load and its impact on the road design. By employing various manuals, the study seeks to identify any variations or disparities in the results obtained.

In addition to the calculation of ESAL, this section provides a comprehensive overview of the detailed procedures followed to arrive at the final outcome. The step-by-step approach is outlined, ensuring transparency and clarity in the methodology employed. Furthermore, it delves into the various parameters considered during the calculation process. These parameters encompass a range of factors, such as axle load, traffic volume, vehicle classification, and road type, among others.

By presenting the detailed procedures and comprehensive consideration of various parameters, this section aims to provide a comprehensive and robust framework for accurately estimating the design traffic. Moreover, by comparing the results obtained from different manuals, this chapter sheds light on potential variations and discrepancies that may arise, thus enabling a better understanding of the strengths and limitations of each approach.

AUSTROADS

By employing Eq. 1 and considering the diverse load factors applicable to the various heavy vehicles, the resulting DESA value is obtained. For a comprehensive calculation, please refer to Appendix A.

$$DESA = 25\,857$$

AASHTO

In the process of converting traffic data into Equivalent Single Axle Load (ESALs), it is necessary to employ specific conversion factors that are tailored to each type of vehicle in use. These corresponding factors can be found in Appendix A, providing a comprehensive reference for accurate conversions. Furthermore, a crucial element in this conversion process is the application of the Load Equivalent Factor (LEF), which serves to establish the correlation between diverse combinations of standard single axle loads and an 80 kN (18,000 lbs) load, facilitating precise calculations.

By employing the various factors as outlined in Appendix A, the number of equivalent axles becomes:

$$W_{18} = [50 * 0{,}0004 + 12 * 0{,}0852 + 6 * 2{,}317 + 8 * 2{,}582] * \left(\frac{6}{7}\right) * 365 * 1$$

$$+[40 * 0{,}0004 + 30 * 0{,}0852 + 4 * 2{,}317] * \left(\frac{1}{7}\right) * 365 * 29 + 2 * 18{,}482 = \mathbf{29\,078}$$

IRC

In contrast to the approaches followed by AASHTO and AUSTROADS, the Indian manuals take into account both the laden and unladen conditions of traffic, incorporating distinct multiplying factors for each. As a result, the cumulative number of standard axles for design (N) is determined to be *23 231*, utilizing the formula outlined in Eq 6. Notably, in the specific case study under consideration, the annual growth rate is zero.

SA

Irrespective of the specific vehicles employed, the South African Method lacks a dedicated approach for calculating traffic. Instead, it relies on the Annual Average Daily Traffic (AADT) to estimate the annual gravel loss. And the AADT calculated from the number of vehicles employed is 76. The total number is calculated by adding together the daily count of vehicles used during both the construction and maintenance phases.

Observations

Table 18 below provides a concise overview of the calculated multiplying factors used to derive the standard axle count, taking into account the specific data associated with the employed vehicles. The table clearly illustrates the side-by-side comparison of the methodologies employed, elucidating the factors that contribute to the differences observed in the obtained results.

Table 18. Summary of the LEF for the different manuals utilized in the case study

Load factors				
Manual / *Vehicles*	AUSTROADS	AASHTO	IRC	
trucks	2.5075	2.582	2,5	Laden HCV
			0,3	Unladen HCV
Semi- trailers	1.1030	2.317	2,5	Laden HCV
			0,3	Unladen HCV
vans	-	0,0852	0,33	Laden MCV
			0,02	unladen MCV
light vehicles	-	0,0004	-	-
Special vehicle (for transformers)	17,49	18,482	-	-

The following key points outline the variations and commonalities observed in the calculation methodologies:

1. *Consideration of laden and unladen circumstances (IRC):* The IRC method takes into account both the laden and unladen conditions of vehicles, ensuring a comprehensive assessment of their impact on the design.

2. *Wider range for vehicle categorization (IRC):* IRC provides a broader spectrum of categories to classify vehicles, allowing for a more detailed analysis of their influence on the design.
3. *Deficiency in considering special vehicles (IRC)*: However, the IRC method falls short in adequately addressing the unique characteristics of special vehicles, which may require specific considerations in the calculation.
4. *Provision of factors for smaller vehicles in the calculation (AASHTO):* AASHTO methodology incorporates specific factors to account for smaller vehicles, recognizing their distinct contribution to the overall design requirements.
5. *Different factors for Semi-trailers (AASHTO & AUSTROADS):* Both AASHTO and AUSTROADS methodologies incorporate specific factors tailored to semi-trailers, acknowledging their unique characteristics and influence on the calculation process.
6. All the manuals exhibit similarities in their treatment of trucks and their influence on the design process.

In summary, although the IRC method takes into account both loaded and unloaded conditions and provides a broader classification of vehicles, it does not include provisions for special vehicles. In contrast, AASHTO and AUSTROADS consider smaller vehicles and incorporate specific factors for semi-trailers, thus ensuring a more comprehensive assessment of their influence.

6.1.2. Pavement Design

AUSTRODS

As explicitly outlined in section 4.1.1 and Eq. 1, this particular methodology primarily considers the strength of the sub-grade and the calculated DESA to ascertain the final thickness required for the structural design, ensuring the protection of the sub-grade (as depicted in Figure 36). Furthermore, to account for potential material loss, an extra thickness of 90 mm is added to the result obtained from the design chart presented in the manual. Additionally, an extra 20 mm is incorporated to accommodate construction tolerance.

Consequently, the calculation procedure yields the following final results:

- Thickness for sub-grade Protection: 140mm
- Construction Tolerance: 20 mm
- Wearing course: 90 mm
- Final thickness:250 mm

Among the overall final thickness, it should be noted that only 160mm is allocated for structural purposes. All the intricate steps and procedures involved in the calculation can be found in Appendix A.

AASHTO

Unlike the other manuals, AASHTO employs the strength property of the base material as an input parameter in the determination of the final design thickness. Additionally, AASHTO takes into account the allowable rutting depth, utilizing nomographs (Figure 40) to assess the associated damage. The calculation of aggregate loss is also incorporated using Eq 4, where it is dependent on the number of loaded trucks in thousands. The following bullet list outlines the specific parameters directly utilized in the design, which are the available data in the case study.

- Sub-grade modulus: 15 000 psi
- Base-layer modulus: 30 000 psi
- Design Serviceability Loss: 2.2
- Allowable rutting: 2 inches

After conducting a series of calculations and employing trial thicknesses ranging from 5 to 8 inches, an interpolation process was employed to assess the damages associated with rutting and design serviceability loss. As a result, a meticulous analysis led to the determination of a thickness of 7 inches. By incorporating a thickness that accounts for gravel loss, the final determined thickness measures 7.4 inches. For a comprehensive and detailed calculation, refer to Appendix A.

After completing the aforementioned calculations, an additional optional step involves converting a portion of the calculated base thickness into sub-base, as depicted in Figure 42. Upon performing this step, the following results are obtained:

Base layer thickness: 4 inches (101.6 mm)

Sub-base layer thickness: 7 inches (177.8 mm)

Total thickness: 11 inches (279.4 mm)

For a comprehensive and detailed calculation, refer to Appendix A.

IRC

Like AUSTROADS, the IRC methodology primarily relies on two key parameters, the sub-grade strength and design traffic, to utilize the design chart (Table 11). By leveraging this chart, a final thickness is determined to ensure adequate protection of the sub-grade. Consequently, the final value obtained from the calculation's quantities to 125 mm.

SA

As stated in section 4.1.4, this methodology encompasses several parameters, and the following bullet points outline these variables along with their corresponding values derived from the available data in the case study.

- ADT: 76
- Weinert N-value: 5
- P26 = 85 Percentage passing the 26.5mm sieve*
- PF = 9.56
- C_t = 10 %
- GL_p = 6.96 mm

Where:

ADT= Average daily traffic

P26 = Percentage passing the 26.5mm sieve*

PF = product of plastic limit and percent passing 0.075mm sieve*

C_t = compaction induced by traffic (%)

GL_p = estimated annual gravel loss (mm)

As a result, the final outcome becomes 230 mm.

A comprehensive and detailed calculation is found in Appendix A.

6.1.3. Sensitivity Analysis

This section is specifically dedicated to the examination of various parameters to assess the sensitivity of each design manual towards variations in these factors. A comprehensive sensitivity analysis is conducted on the parameters of sub-grade and traffic, as these variables play a significant role in almost all the design manuals discussed in this study.

Table 19 is generated through a series of calculations that involve manipulating the subgrade strength and traffic variables to determine the final thickness. In relation to the traffic parameter, a deliberate increase of 10% in the number of heavy vehicles was considered specifically during the construction phase of case study 1. Furthermore, a 5% increase in heavy vehicle volume was taken into account for the maintenance and operation phases. To analyze the sensitivity of the results, the heavy vehicle percentage was decreased by 4% during the construction phase and by 2.5% during the operation phase. These variations in traffic conditions allow for a comprehensive assessment of their impact on the final thickness determination.

Table 19. Sensitivity Analysis summary for Traffic and sub-grade strength

Manuals	Results from Case Study		Traffic ++ (↑10 %HV; ↑5%HV		Traffic-- (↓4 %HV; ↓ 2,5 %HV		Sub-grade CBR (↑↑*50 %) = 15 %*		Sub-grade CBR *(↓↓50 %) = 5 %*	
AUSTROADS	*DESA =25 857*	***250 mm***	*DESA =48 342*	***265 mm (↑ 6 %)***	*DESA =18 002*	***248 mm (↓ 0,8 %)***	*DESA =25 857*	***200 mm (↓ 20 %)***	*DESA =25 857*	***310 mm (↑ 24 %)***
AASHTO	*W_{18} =29 079*	***188 mm***	*W_{18}= 54 225*	***254 mm (↑↑ 35 %)***	*W_{18} = 18 899*	***160 mm (↓↓14,9 %)***	*W_{18} =29 079*	***177,8 mm (↓ 5,4 %)***	*W_{18} =29 079*	***234 mm (↑↑ 24,5 %)***
IRC	*N =23 231*	***125 mm***	*N =38 488*	***150 mm (↑↑ 20 %)***	*N= 17 026*	***125 mm*** No change	*N =23 231*	***125 mm*** No change	*N=23 231*	***175 mm (↑↑ 40 %)***
SA	*ADT =76*	***230 mm***	ADT =102;78	***259 mm (↑↑ 12,6 %)***	ADT =68;72	***220 mm (↓ 4,3 %)***	*ADT =76*	***230 mm*** No change	*ADT =76*	***230 mm*** No change

Where

DESA: Design number of Equivalent Standard Axles (AUSTROADS)

W_{18} = Number of Equivalent Standard Axels (AASHTO)

N= Cumulative number of standard axles for design of gravel base thickness (IRC)

ADT- Average daily traffic (S.A)

Similarly, to the traffic parameter, the subgrade strength is also subjected to variations to evaluate its influence on the final thickness calculation. In this case, the subgrade strength was intentionally increased by 50% and subsequently decreased by 50%. By examining the variations in the subgrade strength, a comprehensive understanding of its role in the determination of the final thickness can be obtained.

From the results of the calculations, several important observations can be made:

AUSTROADS: Changes in sub-grade strength have a more significant impact compared to changes in traffic. The final design thickness is notably affected by variations in the sub-grade strength parameter.

AASHTO: Both changes in traffic and sub-grade strength have a substantial effect on the final design thickness. This indicates that both parameters play a crucial role in the AASHTO methodology.

IRC: Due to the wide range of traffic considered, changes at the lower end of the traffic spectrum may not be readily noticeable. However, overall, both traffic and sub-grade strength parameters do have an impact on the final design thickness in the IRC methodology.

SA (South African Manual): Sub-grade modifications have no effect on the final design thickness unless the sub-grade strength is below a CBR value of 5%. On the other hand, traffic significantly influences the final result, as the annual gravel loss is calculated based on the traffic data. The type and size of vehicles used do not have a direct impact on the aggregate layer thickness.

By analyzing these major points, it becomes evident that different design methodologies place varying degrees of importance on the sub-grade strength and traffic parameters, ultimately impacting the final design thickness determination.

Moreover, further sensitivity analyses were conducted, specifically targeting parameters that are unique to particular manuals. These parameters encompassed the allowable rutting in the AASHTO manual, the impact of traffic-induced compaction, and the significance of the Weinert N-value as stipulated in the South African Manual. The purpose of these analyses was to assess the sensitivity of the design outcomes to variations in these specific parameters, providing valuable insights into their influence on the overall results. A concise summary of these findings is presented in Table 20, encapsulating the key observations and trends resulting from the sensitivity analyses performed on these manual-specific parameters.

Table 20. Sensitivity Analysis Summary on other parameters

AASHTO		***South Africa***				
Case Study-Design thickness	Allowable rutting= 1 in	Case Study-Design thickness	$C_t = 5\%$	$C_t = 20\%$	Weinert N-value =2	Weinert N-value =10
188 mm	↑ ***27,10 %***	***230 mm***	↓ ***4,55 %***	↑ ***9,09 %***	↑ ***12,72 %***	↓ ***21,21 %***

In the AASHTO manual, the sensitivity analysis revealed a significant variation of 27.1% in the final results when considering the maximum and minimum ranges of allowable rutting. This indicates that the allowable rutting parameter has a substantial impact on the overall outcome of the design process according to the AASHTO methodology.

On the other hand, the South African (S.A) manual exhibited a maximum variation of 9.09% in the final results when considering different values (ranging from 5% to 20%) for C_t, which represents traffic-induced compaction. Additionally, the sensitivity analysis conducted on the Weinert N-value, a key parameter in the S.A. manual, showed a total variation of 33.93% when extreme values of N were considered. This suggests that changes in Weinert N-values significantly affect the design outcomes according to the S.A. methodology.

The observed variations in the results emphasize the sensitivity of the AASHTO and S.A. methods to specific parameters, specifically the allowable rutting and Weinert N-value, respectively. These findings highlight the importance of carefully considering and controlling these parameters during the design process to ensure accurate and reliable results in accordance with the respective methodologies.

6.1.4. Comparative Analysis of Design Manuals with Reference to AASHTO

To ensure a thorough comparison of the different manuals, a detailed analysis was carried out by aligning each manual with AASHTO and designing according to the AASHTO method while considering the minimum material requirements specified by the other manuals. This approach was employed to address the discrepancies in material specifications for the base layer outlined in each manual.

Following the completion of the calculation process, Table 21 was produced to summarize the results. It is important to note that the design traffic used to evaluate the design thickness was consistent across all cases, specifically the design traffic W18, which is 29 078 ESALs. Furthermore, the comparison was focused solely on the structural layers, excluding the consideration of wearing courses. In order to maintain consistency, the basic inputs of subgrade strength (CBR = 10%) and design traffic were held constant throughout the comparison.

Table 21. Summary of Comparative Analysis -Case Study 1

AUSTROADS→ AASHTO		IRC→ AASHTO		SA→ AASHTO	
Result using (AUSTROADS)	AASHTO method (min. requirements of AUSTROADS)	Result using (IRC)	AASHTO method (min. requirements of IRC)	Result using (SA)	AASHTO method (min. requirements of SA)
Design Traffic= W_{18}= 29 079					
*160 mm	180 mm (Base course)	125 mm	137 mm (Base course)	230 mm	305 mm (Base course)
Δ 12.5 %		Δ 9.6 %		Δ 32.6 %	

* Structural layer

Based on the results presented in Table 20, several observations can be made:

1. The Indian manual yields result that are most similar to those obtained using the AASHTO method. This similarity can be attributed to the fact that both manuals place

significant emphasis on two key factors: sub-grade strength and traffic. This observation is consistent with the findings of the sensitivity analysis conducted, which highlighted the importance of these parameters.

2. In contrast, the South African (SA) manual prioritizes the Weinert N-value for determining sub-grade protection. This emphasis is justified by the unique conditions in South Africa, where the road materials are susceptible to decomposition, both chemically and physically, when exposed to warm weather and moisture. Therefore, the SA manual takes into account these specific challenges in its design approach.
3. It is noteworthy that the thickness values obtained using the AASHTO method tend to be higher compared to the other manuals. This indicates that AASHTO may adopt a more conservative approach in its design, considering seasonal variations in sub-grade strength. This conservative nature of the AASHTO method contributes to its robustness in accounting for potential fluctuations in sub-grade conditions.

Overall, these observations highlight the differences in emphasis and design philosophies among the manuals, with each manual prioritizing certain factors based on their specific regional considerations and objectives.

6.1.5. Comparative Analysis of Design Manuals with Mechanistic-Empirical approaches

A comprehensive comparison was conducted to assess the differences between the manuals and the Mechanistic-Empirical (ME) methods. Two specific ME methods were considered: the (SDMS) using Eq 10, and the USFS Region 8 (ARMS) using **Error! Reference source not found.**. The results of this comparison are presented in Table 22 and Table 23, respectively.

Table 22 provides a side-by-side comparison of the manual methods and the SDMS ME method. It allows for a detailed analysis of the variations in design thickness and other parameters obtained from the different approaches. This comparison enables a better understanding of how the manual methods differ from the SDMS ME method and provides insights into the factors influencing the design outcomes.

In the SDMS approach, a crucial factor considered is distress rutting, which influences the quantities used in Eq 10. Specifically, the quantities used in place of rut-depth are determined by the allowable rutting depth specified in each manual. Additionally, for the base layer modulus, the minimum requirements from each manual are utilized.

It is worth noting that the sub-grade strength, CBR of 10%, remains consistent across all the manuals and is based on the available data from the case study 1. Furthermore, it is important to emphasize that the thickness comparisons conducted were specifically focused on the structural layers of the pavement.

By employing the SDMS method and incorporating the relevant specifications and parameters from each manual, a comprehensive assessment is made, highlighting the similarities and differences in design thickness and other key factors among the manuals. This comparative analysis provides valuable insights into the variations and implications of utilizing different design methods, enabling engineers to make informed decisions and select the most suitable approach for their specific project requirements.

Table 22. Comparative Analysis between the Different Manuals and the SDMS Method (Case Study 1)

	Allowable Rutting		Suggested Base layer Modulus (E_1)		Base thickness using Manual (mm)	Base thickness -Rutting equation (mm)
	mm	inch	CBR	psi		
AUSTROADS	75	2.96	50	31 241	160	123
AASHTO	50.8	2	46.9	30 000	203.2	181
IRC	50	1.97	80	42 205	125	123
SA	75	2.95	15	14 457	230	211

Based on the findings in Table 22, it is clear that there is a variation in the results which can reach up to 20%. This variation can be attributed to the utilization of a wide range of material properties for the base course in each manual. Among all the manuals examined, the IRC manuals demonstrate similar results to the SDMS methods, while the AUSTROADS manual shows the largest deviation in the results.

Table 23. Comparative Analysis between the different manuals and the ARMS method (Case Study 1)

	Allowable Rutting		Suggested Base layer Modulus (E_1)		Base thickness using Manual (mm)	Base thickness -ARMS equation (mm)
	mm	inch	CBR	psi		
AUSTROADS	75	2.96	50	31 241	160	209
AASHTO	50.8	2	46.9	30 000	203.2	228
IRC	50	1.97	80	42 205	125	127
SA	75	2.95	15	14 457	230	250

Similarly, Table 23 offers a comparative analysis between the manual methods and the ARMS ME method. This table provides a comprehensive overview of the variations in design thickness and other relevant parameters obtained from each approach. By examining the differences between the manual methods and the ARMS ME method, it becomes possible to gain valuable insights into the impact of the ME approach on the design outcomes.

Among the various methods used, the ARMS method yields results that are closest to those obtained from the IRC method. However, there is a slight difference in the results when compared to the AUSTROADS method.

6.2. Case Study 2

The wind farm case study presented in this section involves turbines with different hub heights, specifically 88.6m and 80m. The project is located in the United States, specifically within Climatic Region VI (as depicted in Figure 43). The client's construction schedule assumes varying traffic distribution throughout the year: 36% in winter, 18% during spring thaw, 18% in spring/fall, and 28% in summer.

The aggregate base material is assumed to have a California Bearing Ratio (CBR) value of 30. The Equivalent Single Axle Load (ESAL) count for the road is estimated to be around 5500.

Furthermore, a road section with an ESAL count of 1300 is evaluated, which represents roads experiencing traffic from a single turbine and operations and maintenance (O&M) building.

For road sections in the project design, a change in serviceability index of 1.5 is utilized, with a terminal serviceability index of 1.5. The allowable rutting depth is set at 3 inches, with the intention of minimizing road maintenance. Given the poor soil conditions, ongoing maintenance of aggregate-surfaced access roads is expected to ensure their safety and serviceability. Designing an aggregate section with sufficient thickness to eliminate the need for ongoing maintenance is impractical and could result in significant cost overruns.

During the project construction phases, it is important to anticipate periodic inspection and repair of ruts, depressions, and soft subgrade to facilitate traffic flow. This may involve placing additional aggregate in ruts and depressions or completely replacing the aggregate section and soft subgrade with a new one.

6.2.1. Pavement Design

Similar to case study-1, the same procedure is followed in this case study to determine the design traffic and subsequently calculate the final thickness. However, in this particular case study, the sub-grade material exhibits variable values depending on the season, as indicated below:

- ✓ Winter: 20,000 psi
- ✓ Spring: 1 500 psi
- ✓ Fall: 3 300 psi
- ✓ Summer: 4 900 psi

The elastic modulus of the base material is specified as CBR of 30%, which is equivalent to 22 529 psi. The variability in sub-grade strength allows for the full and effective utilization of the AASHTO method, while for other methods, an average value is used during the calculation process.

After conducting a comprehensive calculation using each method, the results are summarized in Table 24. It is important to note that the obtained thicknesses correspond to the base material, which must meet the minimum requirements specified in each manual. Detail calculations are presented in Appendix B.

Table 24.Summary of the results of design thickness of case-study 2 for each manual

	AUSTROADS	**AASHTO**	**IRC**	**S. A**
	Thickness in mm			
Wearing course	100			
Base course	185	102	175	214
Sub-base		165		
Total thickness	285	267	175	214

6.2.2. Comparative Analysis of Design Manuals with Reference to AASHTO

In a similar manner as in case study 1, a comparison was conducted among different manuals using AASHTO as the reference standard. The objective was to determine how closely or distantly each manual aligns with the results obtained from the AASHTO method.

To perform this comparison, the minimum requirements for base material strength specified in each manual were considered. A design procedure based on the AASHTO method was then applied, and the outcomes were documented in Table 25. This table presents the results obtained from the comparative analysis. Additionally, it is important to highlight that the traffic used in all the procedures, including those involving other manuals, remains consistent. The available data for the traffic is 5600.

Table 25. Summary of Comparative Analysis -Case Study 2

AUSTROADS→ AASHTO		IRC→ AASHTO		SA→ AASHTO	
Result using (AUSTROADS)	AASHTO method (min. requirements of AUSTROADS)	Result using (IRC)	AASHTO method (min. requirements of IRC)	Result using (SA)	AASHTO method (min. requirements of SA)
Design Traffic= W 18= 5 600					
*185 mm	196 mm (Base course)	175 mm	175 mm (Base course)	214 mm	330 mm (Base course)
Δ 5.94 %		Δ 0 %		Δ 54,2 %	

*structural layer

Based on the findings presented in Table 25, several key observations can be made:

1. The Indian manual produces results that are identical to those obtained through the AASHTO method. This similarity can be attributed to both manuals placing significant emphasis on two crucial factors: sub-grade strength and traffic. These findings align with the outcomes of the sensitivity analysis, which emphasized the importance of these parameters.
2. In contrast, the South African (SA) manual places greater importance on the Weinert N-value for determining sub-grade protection. This emphasis is justified by the unique conditions in South Africa, where road materials are susceptible to chemical and physical decomposition when exposed to warm weather and moisture. As a result, the SA manual takes these specific challenges into account in its design approach.
3. It is worth noting that the thickness values derived from the AASHTO method tend to be higher compared to the other manuals. This suggests that AASHTO may adopt a more conservative approach in its design, considering seasonal variations in sub-grade strength. This conservative nature of the AASHTO method contributes to its robustness in accounting for potential fluctuations in sub-grade conditions.

Taken together, these observations underscore the differences in emphasis and design philosophies among the manuals, with each manual prioritizing certain factors based on their specific regional considerations and objectives.

6.2.3. Comparative Analysis of Design Manuals with Mechanistic-Empirical approaches

Similar to in section 6.1.5, a comprehensive comparison was conducted to assess the differences between the manuals and the two Mechanistic-Empirical (ME) methods mentioned, SDMS and ARMS, using Eq 10 and Eq 11 respectively. The results of this comparison are presented in Table 26 and Table 27, for SDMS and ARMS respectively.

Table 26. Comparative Analysis between the Different Manuals and the SDMS Method (Case Study 2)

	Allowable Rutting		Suggested Base layer Modulus (E_1)		Base thickness using Manual (mm)	Base thickness -Rutting equation (mm)
	mm	inch	CBR	psi		
AUSTROADS	75	2.96	50	31 241	185	88
AASHTO	50.8	2	30	22 529	203.2	166
IRC	50	1.97	80	42 205	175	88
SA	75	2.95	15	14 457	214	202

Table 26 presents a direct comparison between the manual methods and the SDMS ME method, facilitating a comprehensive examination of the disparities in design thickness and other parameters derived from these distinct approaches. This comparative analysis enhances our understanding of the distinctions between the manual methods and the SDMS ME method, offering valuable insights into the factors that influence the design outcomes.

The results presented in Table 26 indicate a noticeable disparity in the findings, with variations reaching up to 50%. These variations can be attributed to the utilization of diverse material properties for the base course in each manual. Among the examined manuals, the IRC manuals demonstrate similar outcomes to the SDMS methods, whereas the AUSTROADS manual exhibits the most significant deviation in the results.

Table 27. Comparative Analysis between the different manuals and the ARMS method (Case Study 2)

	Allowable Rutting		Suggested Base layer Modulus (E_1)		Base thickness using Manual (mm)	Base thickness -ARMS equation (mm)
	mm	inch	CBR	psi		
AUSTROADS	75	2.96	50	31 241	185	242
AASHTO	50.8	2	46.9	30 000	203.2	228
IRC	50	1.97	80	42 205	175	178
SA	75	2.95	15	14 457	214	231

Likewise, Table 27 presents a comprehensive comparative analysis of the manual methods and the ARMS ME method. This table provides a detailed overview of the variations observed in design thickness and other pertinent parameters obtained from each approach. By scrutinizing the disparities between the manual methods and the ARMS ME method, valuable insights can be gained regarding the influence of the ME approach on the design outcomes. Among the different methods employed, the ARMS method demonstrates results that closely align with those obtained from the IRC method. Nevertheless, there exists a slight variation in the outcomes when compared to the AUSTROADS method.

Chapter 7

7. Conclusion and recommendations

This section provides a concise yet comprehensive overview of the thesis paper, outlining the methodologies, procedures, and pathways followed to obtain the results. The research encompassed a series of carefully executed design calculations, thorough sensitivity analyses, and a detailed comparative examination of each manual and the Mechanistic-Empirical (ME) approaches.

To contextualize the study, an extensive literature review was conducted, delving into the historical background of unpaved roads. This review encompassed various aspects, including the classification of unpaved roads into main categories, the construction and maintenance practices employed, as well as an exploration of the primary distresses commonly associated with these roads.

Moreover, the section encapsulates a comprehensive examination of all the elements considered in the research, coupled with their practical implementation. This involved drawing insights from past case studies, specifically focusing on the application of the methodologies within the context of wind and solar farms. These case studies served as valuable practical examples, enabling the assessment of the effectiveness and applicability of the methodologies employed throughout the thesis.

By summarizing the procedures undertaken and incorporating the relevant literature and practical implementation, this section provides a condensed yet informative glimpse into the broader framework and approach adopted in the thesis.

7.1. Conclusions

The subsequent bullet points outline the key concepts and methodologies that were employed in the execution of this thesis, highlighting their significance and contribution to the overall results:

- A comprehensive literature review was conducted to explore the historical evolution of unpaved roads, encompassing an in-depth analysis of their prominent categories, construction practices, maintenance approaches, and the primary distresses commonly associated with such roadways. This review delved into the rich historical context of unpaved roads, shedding light on their development, challenges, and advancements over time.

- In addition to the extensive coverage of unpaved roads, a concise yet informative overview was provided regarding the specific context of solar and wind farms. This included a brief examination of the unique considerations and requirements associated with the construction, operation, and maintenance of roads within these renewable energy facilities. By incorporating this additional focus on solar and wind farms, the thesis offers valuable insights into the practical implementation and challenges faced within these specialized environments.
- The thesis undertook a comprehensive exploration and analysis of the design methodologies employed for unpaved roads, drawing from the expertise and guidelines provided in renowned manuals such as AASHTO, AUSTROADS, IRC, and S.A manuals. These manuals served as invaluable resources, offering detailed frameworks and procedures for designing and assessing unpaved road systems.
- In addition to referencing these established manuals, the study integrated two empirical mechanistic approaches into the analysis. These approaches provided a more rigorous and scientifically grounded perspective by considering the underlying structural behavior and performance of the unpaved road systems. By incorporating these mechanistic approaches, the study aimed to enhance the accuracy and reliability of the design process.
- The thesis places significant emphasis on the utilization of the aforementioned manuals as a fundamental framework for conducting a thorough and comprehensive analysis of previous case studies pertaining to wind and solar farms. This analysis involves a meticulous examination of these case studies, with a specific objective of contrasting and comparing the approaches employed within them. The primary goal is to identify any variations or discrepancies that may arise in the design methodologies, procedures, and considerations specific to the context of wind and solar farms. By conducting this critical analysis, the research aims to shed light on the diverse perspectives and approaches utilized in these case studies, ultimately contributing to a deeper understanding of the

design considerations and challenges associated with road construction within the context of renewable energy facilities.

- Furthermore, a comprehensive sensitivity analysis was conducted, scrutinizing the crucial variables to assess their significance and the magnitude of their influence on the final outcome. This analysis aimed to ascertain the sensitivity of the results to changes in these key variables, allowing for a more comprehensive understanding of their impact on the overall outcome. Through this meticulous examination, the thesis sought to identify the variables that carry substantial weight in the design process, enabling a more informed decision-making process and ensuring the robustness and reliability of the final outcomes.

7.2. Recommendations

Based on the findings from the calculations and the subsequent sensitivity analysis, it becomes evident that variations in subgrade strength, traffic patterns, and other crucial parameters have a significant impact on determining the final thickness. These observed variations highlight the substantial role played by these factors in the overall determination of the pavement thickness. The sensitivity analysis provides valuable insights into how changes in these key parameters can directly influence the structural integrity and performance of the pavement. Consequently, these results emphasize the importance of accurately assessing and considering these influential factors during the design and construction processes to ensure the desired outcomes in terms of pavement thickness and long-term durability.

Moving forward, future studies can delve into conducting an extensive analysis of the influence of additional parameters that were not considered in this thesis. These parameters could include a thorough examination of the precise relationship and correlation between environmental factors such as temperature and precipitation on the determination of the structural thickness. It is noteworthy that the manual mentioned in this thesis, apart from the South African method, did not extensively explore these aspects. Therefore, conducting further research to explore the impact of these unexplored parameters would contribute to a more comprehensive understanding of their effects on pavement design and performance. By incorporating these factors into the analysis, researchers can enhance the accuracy and reliability of pavement design methodologies, leading to more robust and resilient infrastructure systems.

This aspect holds significant importance considering the inherent nature of unpaved roads, which are directly exposed to the surrounding environment. Unlike paved roads, these unpaved surfaces are more vulnerable to the effects of environmental impacts and variations. Factors such as temperature fluctuations, precipitation levels, and other climate-related variables can greatly influence the condition and performance of unpaved roads. Understanding the precise relationship between these environmental parameters and their impact on unpaved roads is crucial for developing effective design and maintenance strategies. By conducting further studies to explore these relationships, we can enhance our ability to mitigate the negative effects of environmental factors on unpaved roads and ensure their long-term durability and functionality.

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Appendix A

Case Study -1

Calculation of design traffic

Vehicles employed for construction and Operation & Maintenance stage

Construction Stage (6 days/week)	
Vehicle types	Number
trucks	4
Semi- trailers	3
vans	6
light vehicles	25

Operation and maintenance stage (1 day/week)	
Vehicle types	Number
Semi- trailers	2
vans	15
light vehicles	20
Special vehicle (for replacement of transformers)	1

Calculation of Design Traffic Using the different Manuals

AUSTROADS

EASL Calculation			
Construction Stage (6 days/week)		*Operation and maintenance stage (1 day/week)*	
Vehicle types	Load factor	*Vehicle types*	Load factor
trucks	2.508	Semi- trailers	1.1030
Semi- trailers	1.103	vans	0
vans	0,000	light vehicles	0
light vehicles	0,000	Special vehicle (for replacement of transformers)	17.49
Sum	3.611		

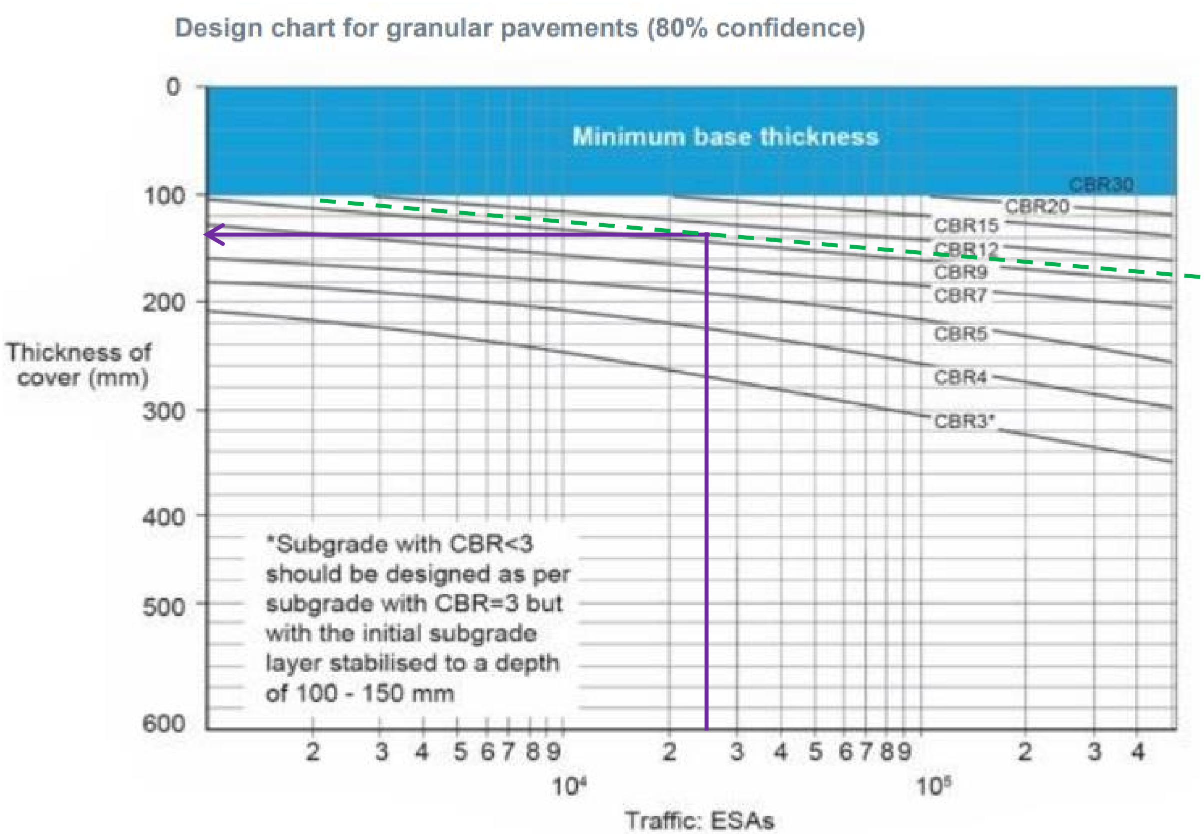
AASHTO

EASL Calculation					
Construction Stage (6 days/week)			*Operation and maintenance stage (1 day/week)*		
Vehicle types	*Number	Load factor	*Vehicle types*	*Number	Load factor
trucks	8	2,582	Semi- trailers	4	2,317
Semi- trailers	6	2,317	vans	30	0,0852
vans	12	0,0852	light vehicles	40	0,0004
light vehicles	50	0,0004	Special vehicle (for replacement of transformers)	2	18,482

*Multiplied by 2 to account entering & leaving the sub-station

Calculation of Design Thickness for the different Manuals

AUSTROADS



AASHTO

Trial thickness 1

Trial Base thickness, D_{BS} (inches)=			5	$\Delta PSI =$	2,2	RD (inches) =	2
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
All seasons	11152,98	30000	29078,57	33000	0,88	12000	2,42

Trial thickness 2

Trial Base thickness, D_{BS} (inches)=			6	$\Delta PSI =$	2,2	RD (inches) =	2
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
All seasons	11152,98	30000	29078,57	40000	0,73	19000	1,53

Trial thickness 3

Trial Base thickness, D_{BS} (inches)=			7	$\Delta PSI =$	2,2	RD (inches) =	2
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
All seasons	11152,98	30000	29078,57	53000	0,55	29000	1,00

Trial thickness 4

Trial Base thickness, D_{BS} (inches)=			8	$\Delta PSI =$	2,2	RD (inches) =	2
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
All seasons	11152,98	30000	29078,57	69000	0,42	40000	0,73

Nomograph for serviceability

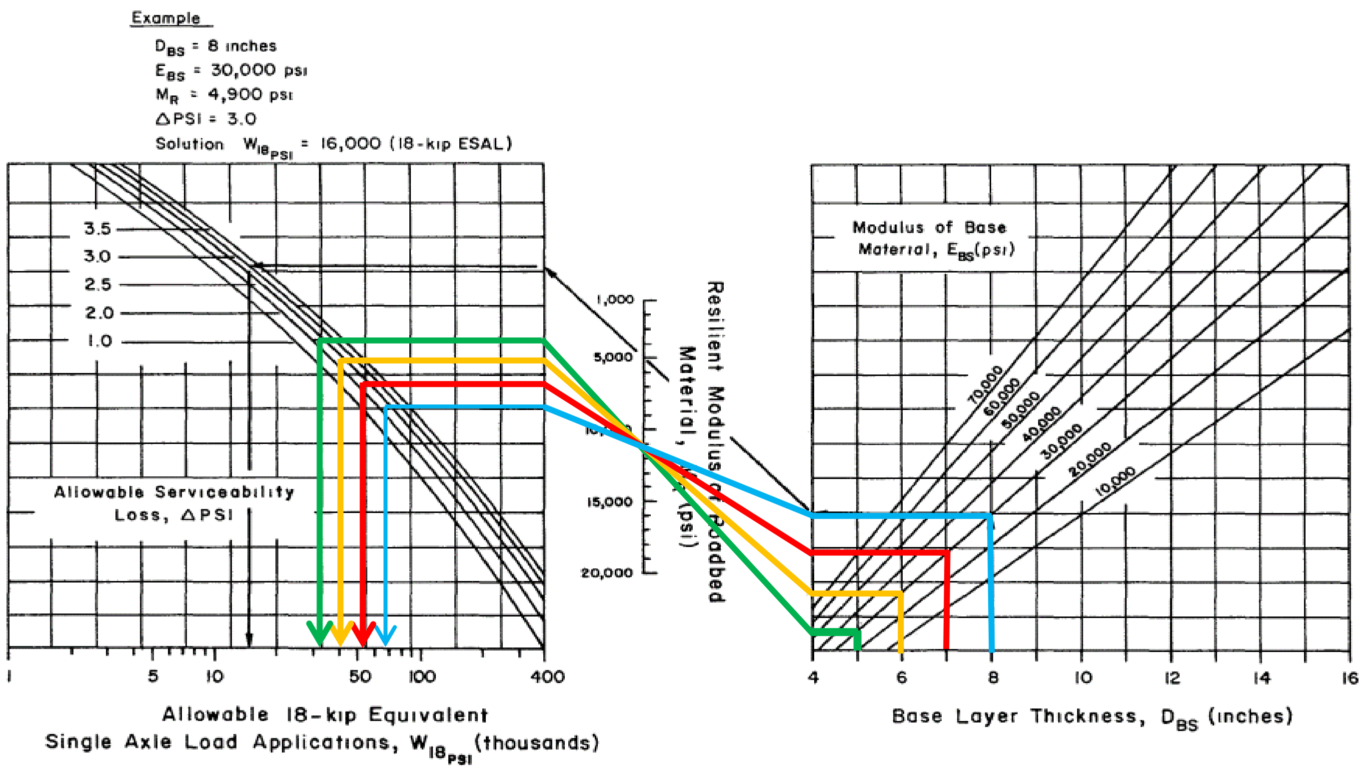


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

Nomographs for rutting

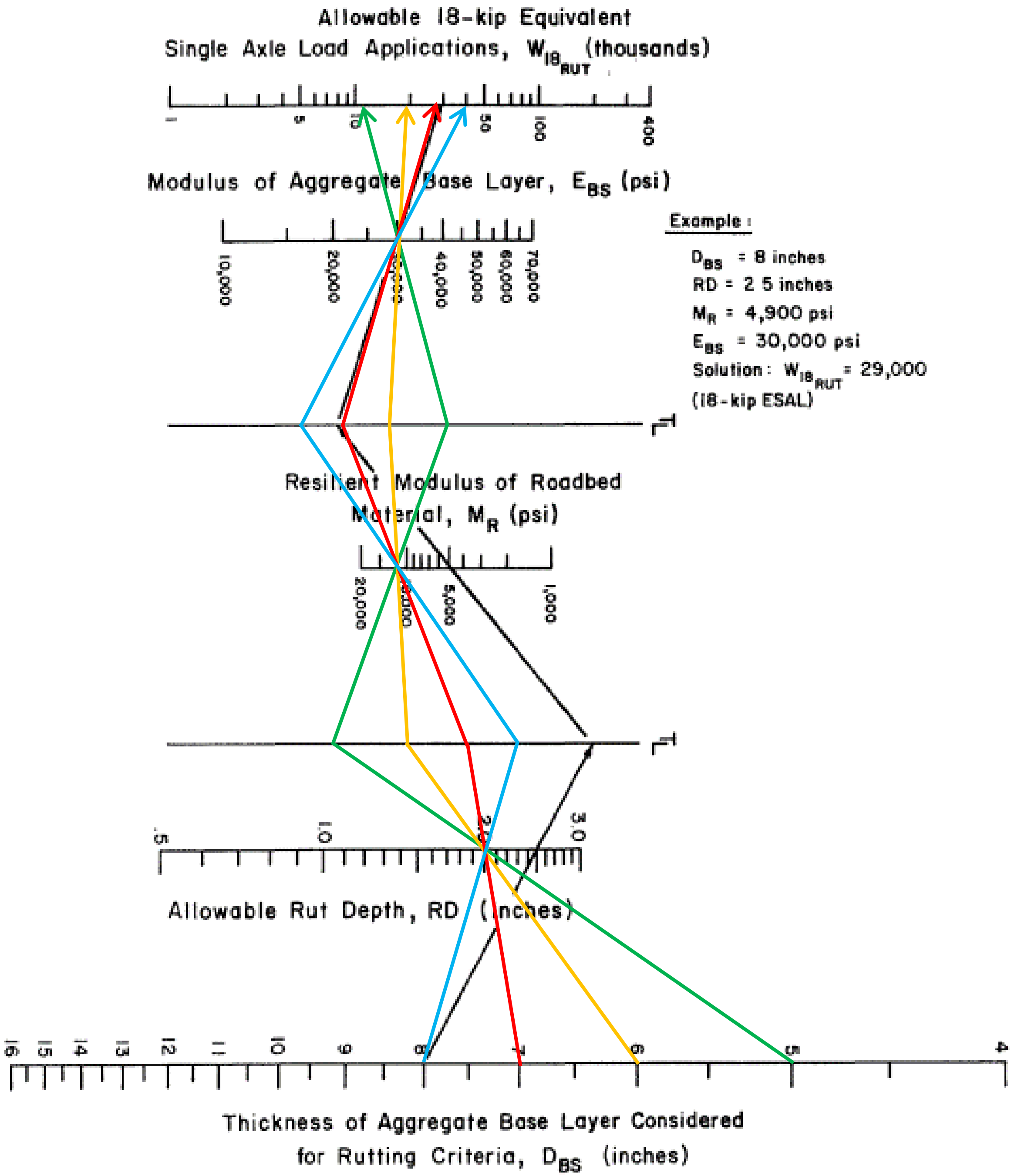
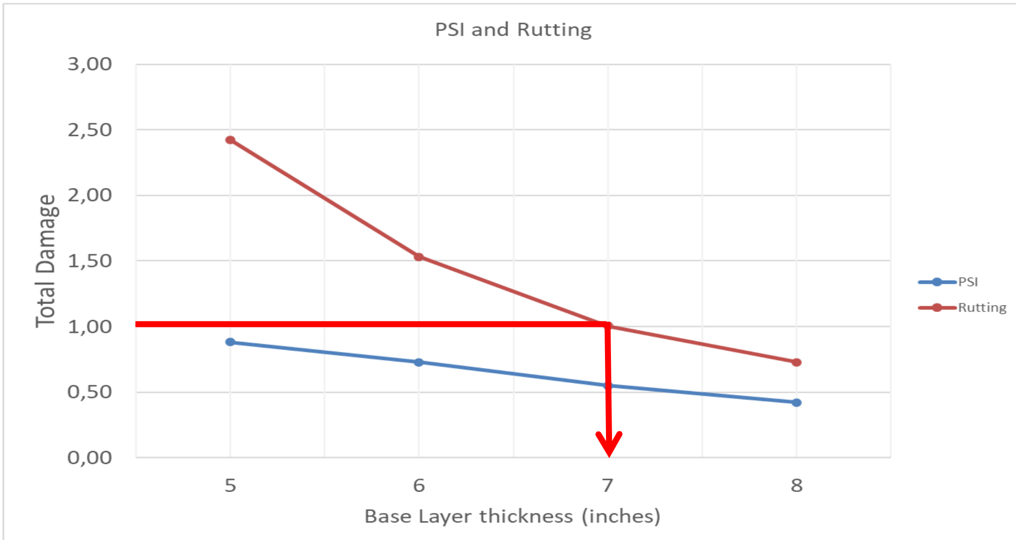


Figure 4.3. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting

Calculation of base thickness using damage criteria for rutting and Rutting



Conversion of base thickness to sub-base thickness

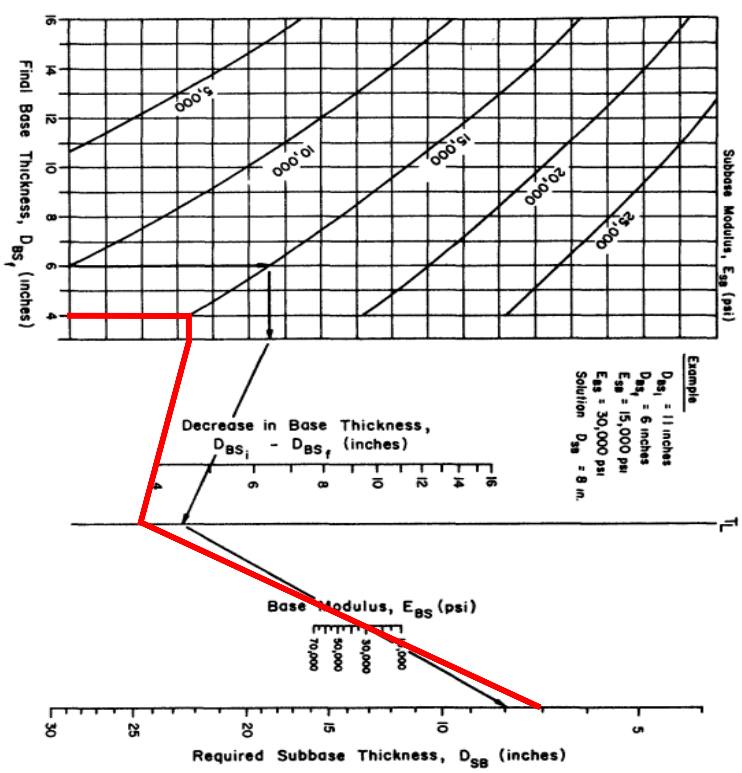


Figure 4.5. **Chart to Convert a Portion of the Aggregate Base Layer Thickness To an Equivalent Thickness of Subbase**

IRC

Cum. EASL / CBR%	10000-30000	30000-60000	60000-100000
CBR =2 (very poor)	200, 100 (mm)	75, 150, 100 (mm)	75, 100, 100, 100 (mm)
CBR =3-4 (poor)	200 (mm)	275 (mm)	75, 100, 150 (mm)
CBR =5-6 (fair)	175 (mm)	250 (mm)	275 (mm)
CBR =7-9 (Good)	150 (mm)	175 (mm)	225 (mm)
CBR =10-15 (very good)	125 (mm)	150 (mm)	175 (mm)

Gravel base

Granular sub-base

Improved sub-grade

Bituminous surface treated

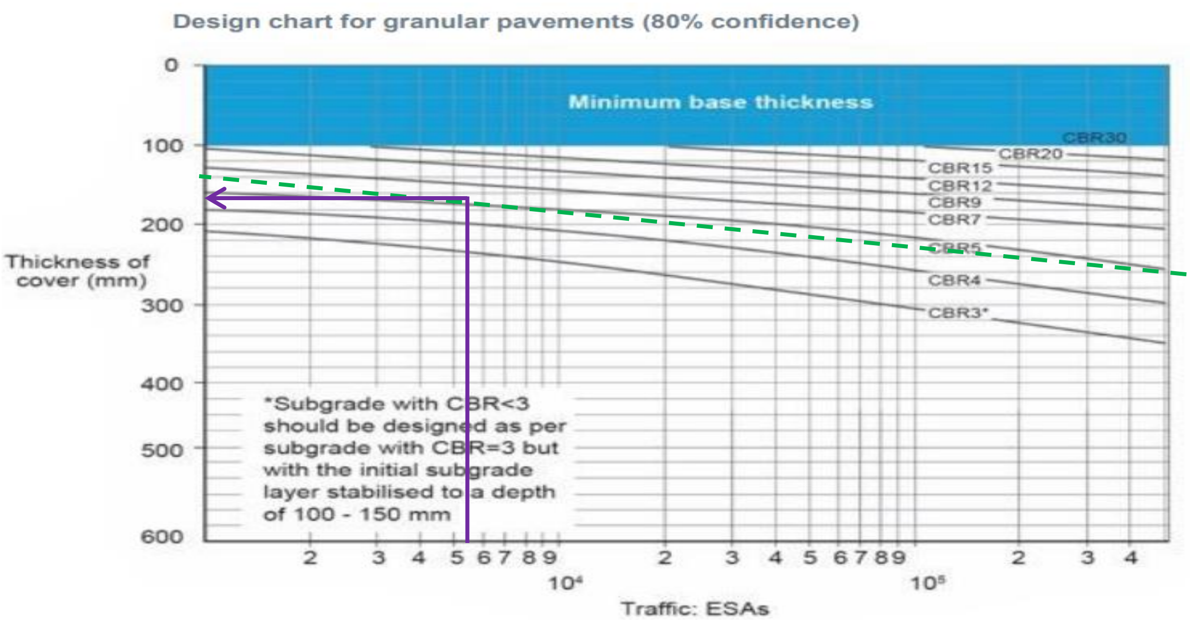
Base of Gravel

Appendix B

Case Study -2

Calculation of Design Thickness for the different Manuals

AUSTROADS



AASHTO

Trial thickness 1

Trial Base thickness, D_{BS} (inches)=			5	ΔPSI	1,5	RD (inches) =	3
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage$W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage$W_{18}/(W_{18})_{RUT}$
Winter	20000	22529,34	2016	180000	0,0112	20000	0,1008
Spring/Thaw	1500	22529,34	1008	2200	0,4582	1600	0,63
Spring/Fall	3300	22529,34	1008	2300	0,4383	3500	0,288
Summer(dry)	4900	22529,34	1568	3000	0,5227	4900	0,32
All seasons					1,43		1,34

Trial thickness 2

Trial Base thickness, D_{BS} (inches)=			**6**	$\Delta\, PSI$	1,5	RD (inches) =	3
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
Winter	20000	22529,34	2016	215000	0,009	35000	0,058
Spring/Thaw	1500	22529,34	1008	2330	0,433	2450	0,411
Spring/Fall	3300	22529,34	1008	2667	0,378	5750	0,175
Summer(dry)	4900	22529,34	1568	4000	0,392	7500	0,209
All seasons					1,21		0,85

Trial thickness 3

Trial Base thickness, D_{BS} (inches)=			**7**	$\Delta\, PSI$	1,5	RD (inches) =	3
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
Winter	20000	22529,34	2016	250000	0,008	50000	0,040
Spring/Thaw	1500	22529,34	1008	2330	0,433	3500	0,288
Spring/Fall	3300	22529,34	1008	3700	0,272	8200	0,123
Summer(dry)	4900	22529,34	1568	5800	0,270	12000	0,131
All seasons					0,98		0,58

Trial thickness 4

Trial Base thickness, D_{BS} (inches)=			**8**	ΔPSI	=1,5	RD (inches) =	3
Season Roadbed moisture condition	Roadbed Resilient Modulus, M_R (psi)	Base Elastic Modulus, E_{BS}(psi)	Projected 18-Kip EASL Traffic, W_{18}	Allowable 18-Kip Traffic, $(W_{18})_{PSI}$	Seasonal Damage $W_{18}/(W_{18})_{PSI}$	Allowable 18-Kip EASL Traffic, $(W_{18})_{RUT}$	Seasonal Damage $W_{18}/(W_{18})_{RUT}$
Winter	20000	22529,34	2016	290000	0,007	70000	0,029
Spring/Thaw	1500	22529,34	1008	2600	0,388	4500	0,224
Spring/Fall	3300	22529,34	1008	5000	0,202	11000	0,092
Summer(dry)	4900	22529,34	1568	7150	0,219	15000	0,105
All seasons					0,82		0,45

Nomograph for serviceability

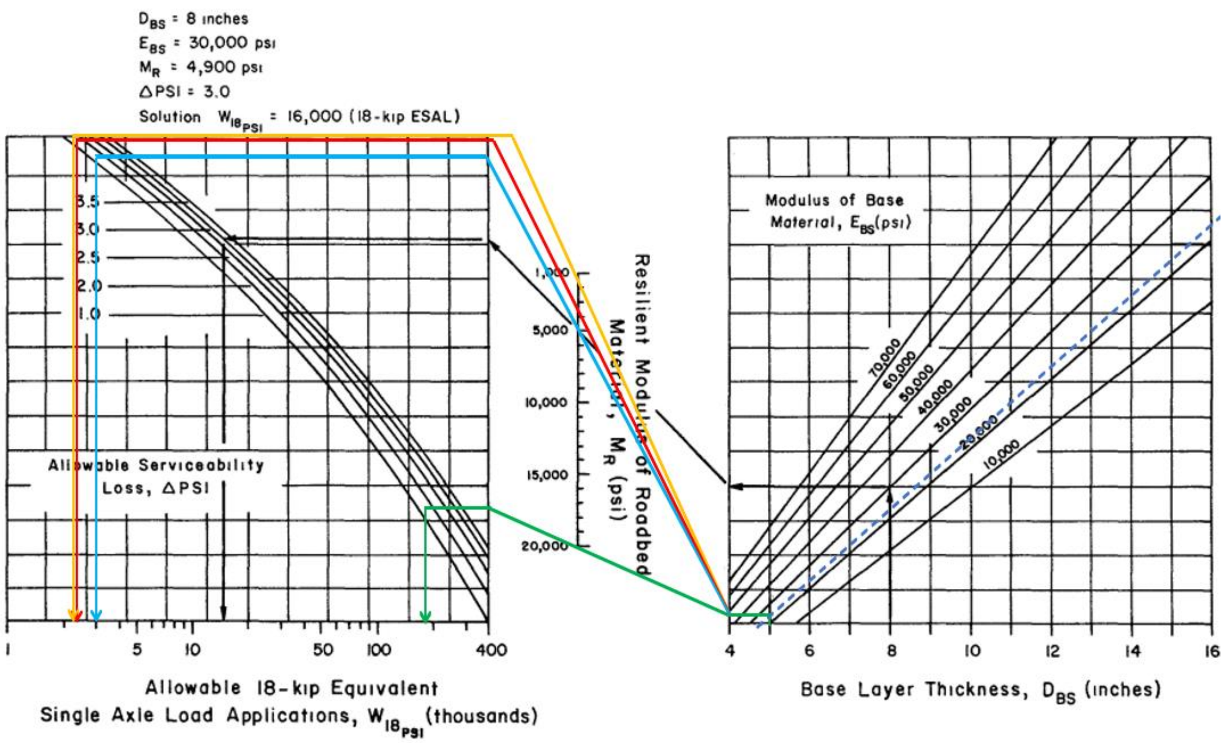


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

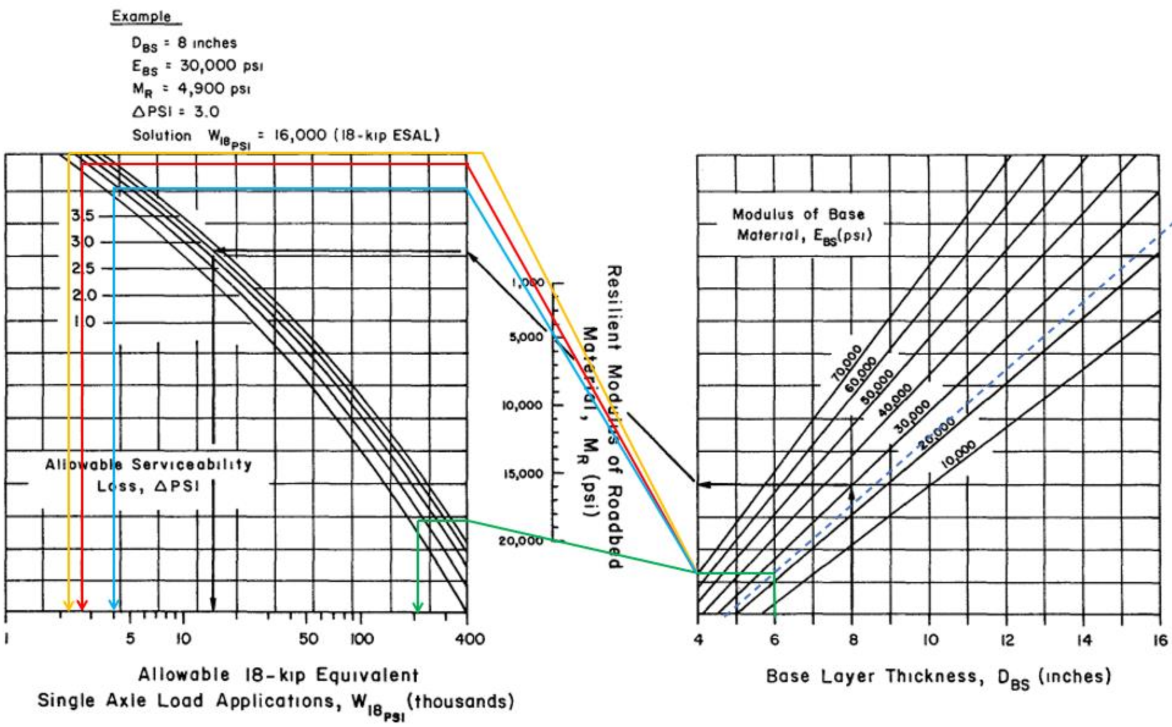


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

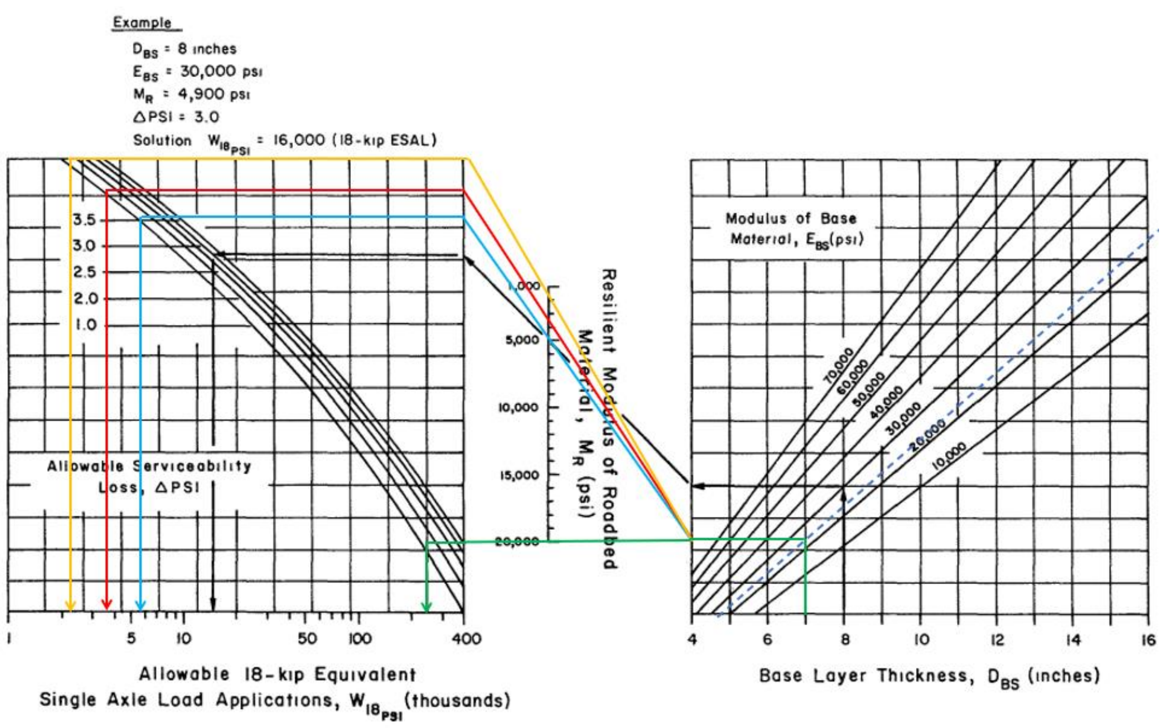


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

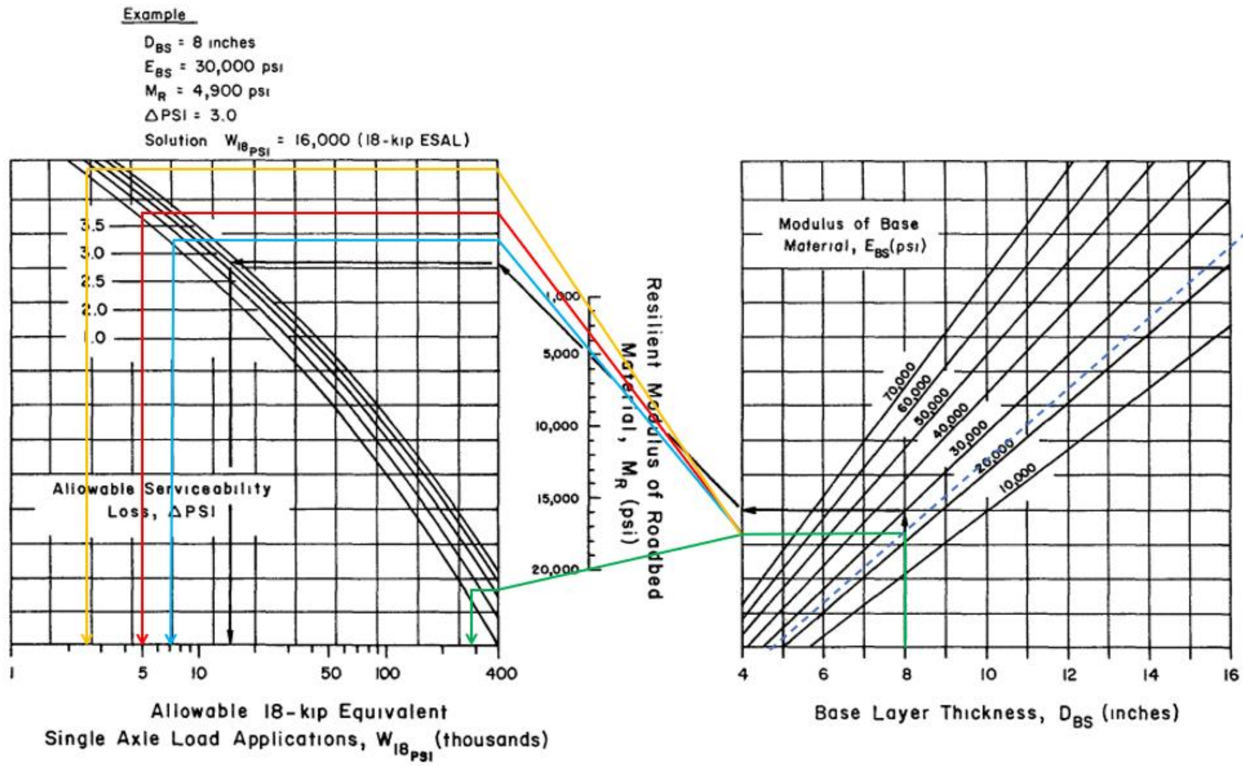


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

Nomograph for Rutting

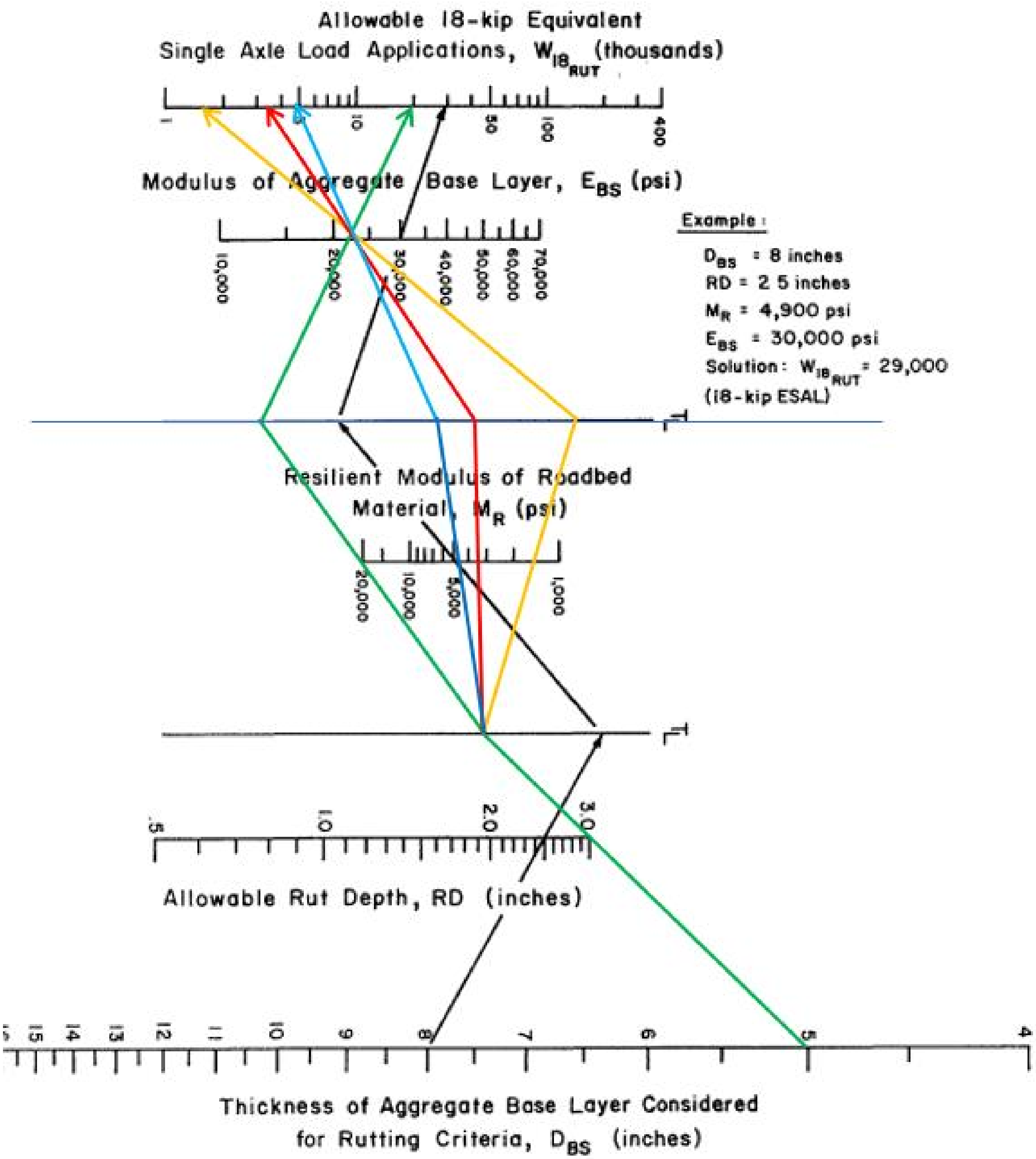


Figure 4.3. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting

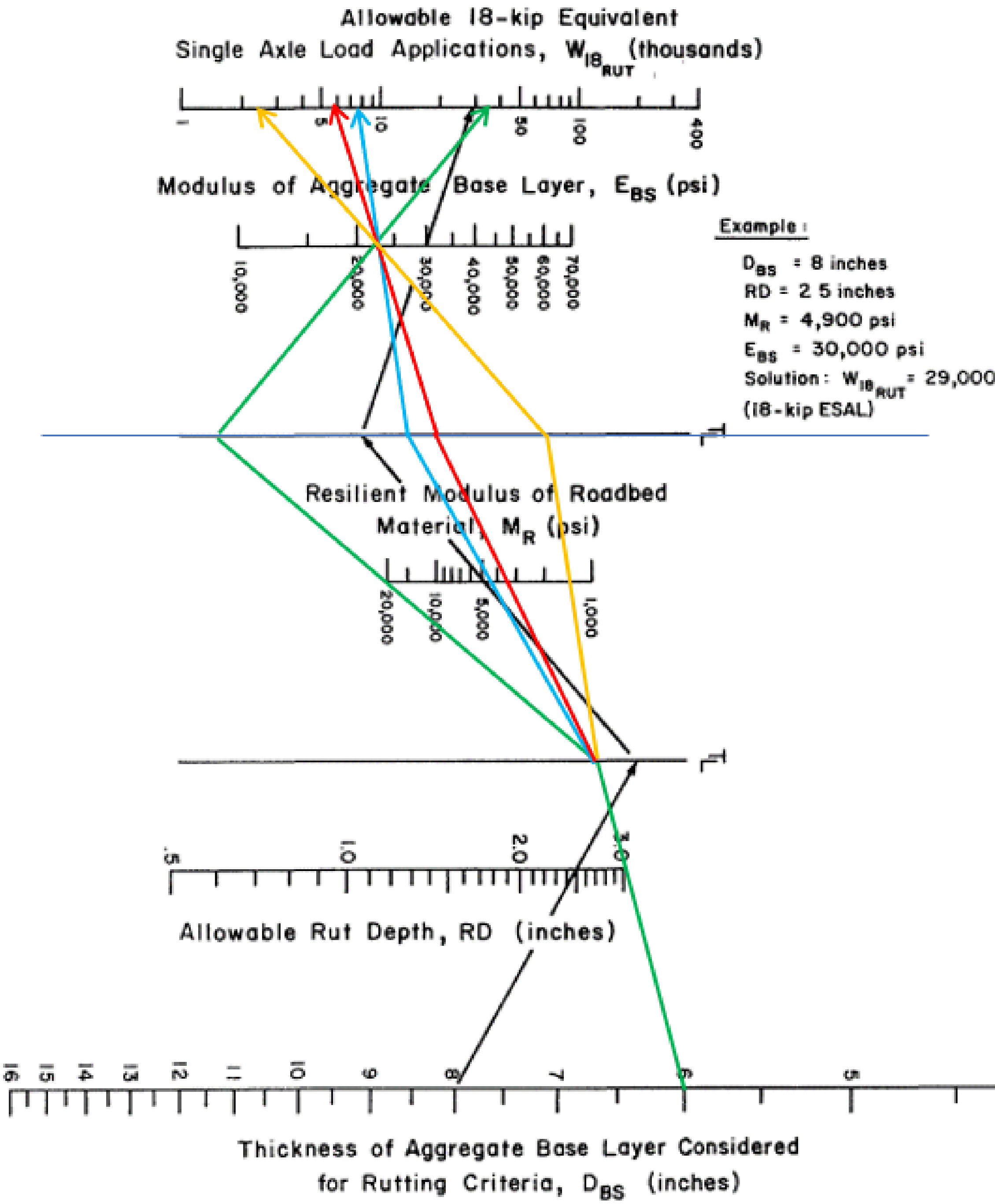


Figure 4.3. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting

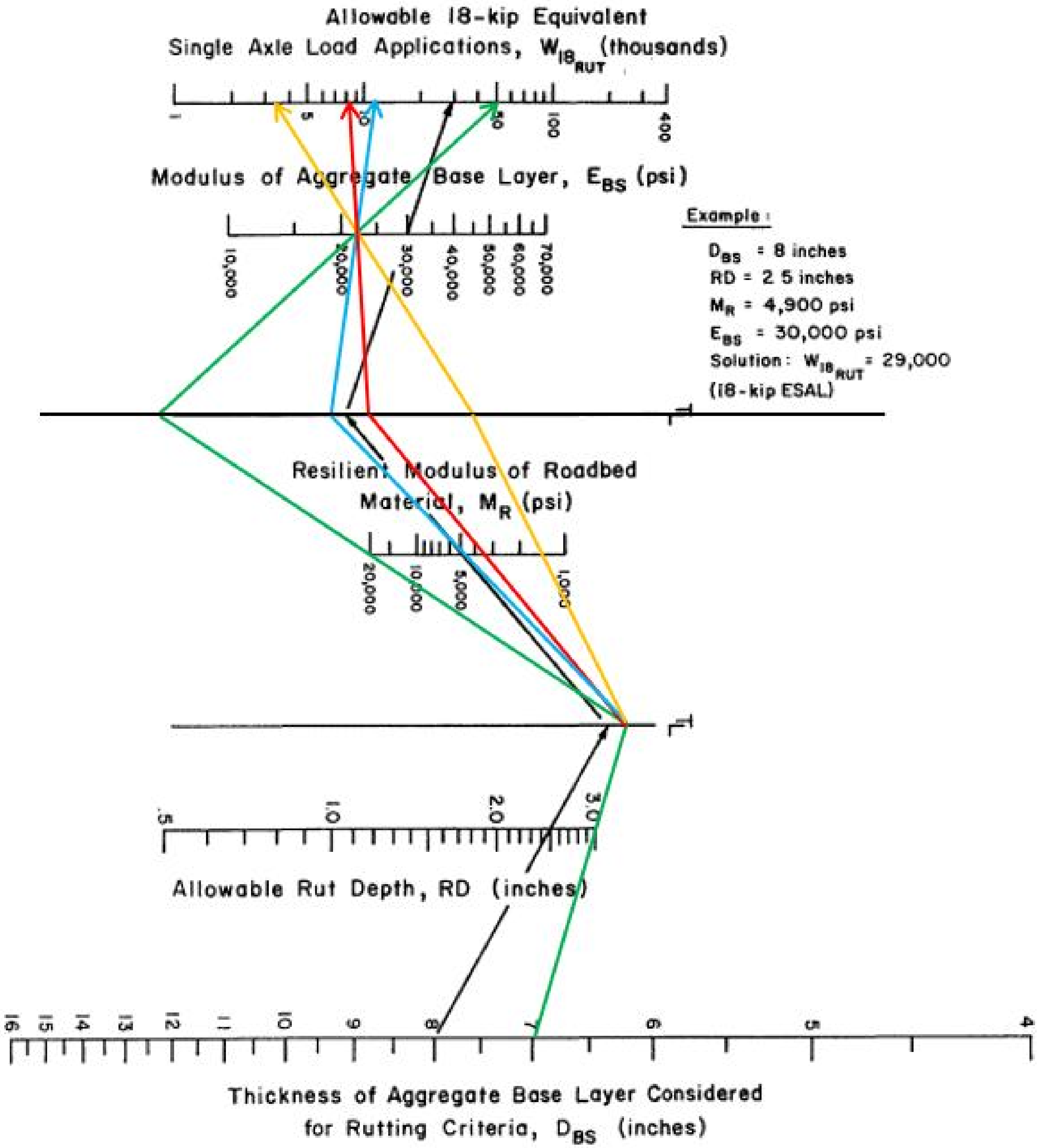


Figure 4.3. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting

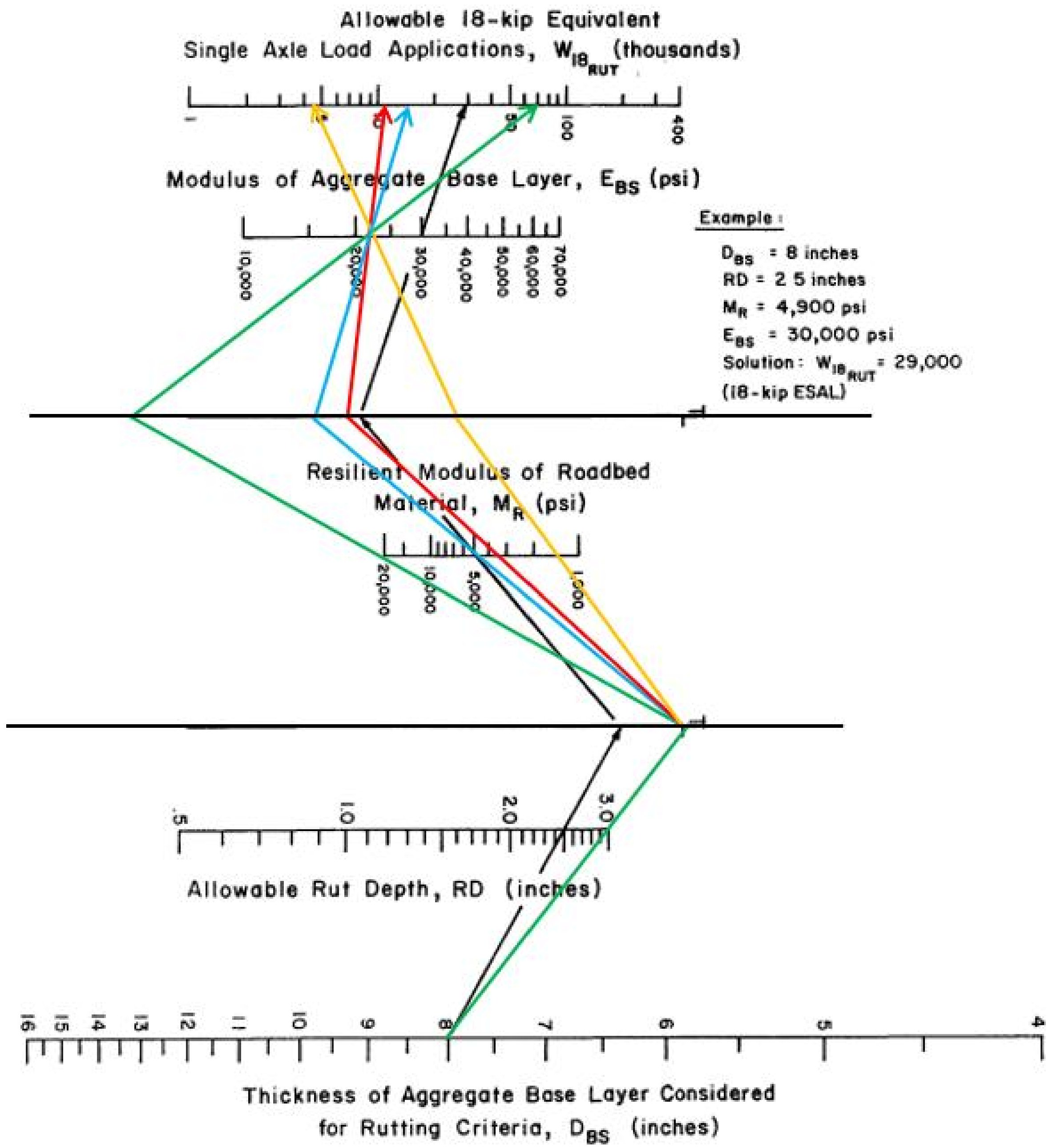
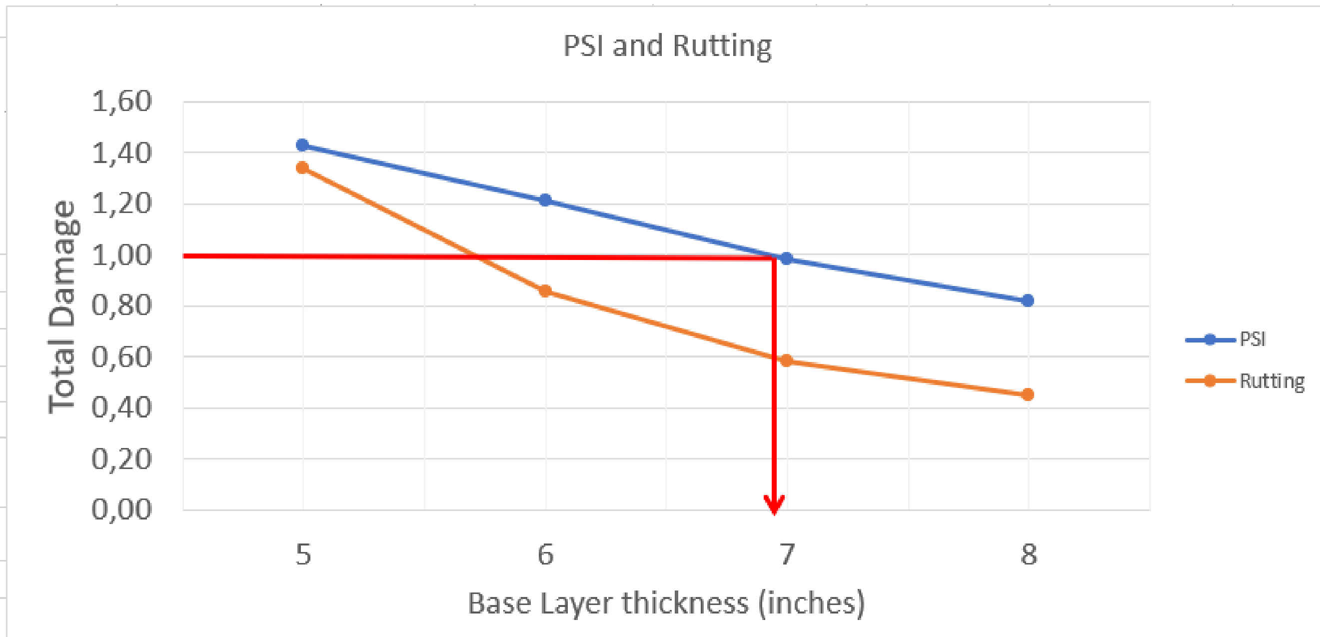


Figure 4.3. Design Chart for Aggregate-Surfaced Roads Considering Allowable Rutting

Calculation of base thickness using damage criteria for rutting and Rutting



Conversion of base thickness to sub-base thickness

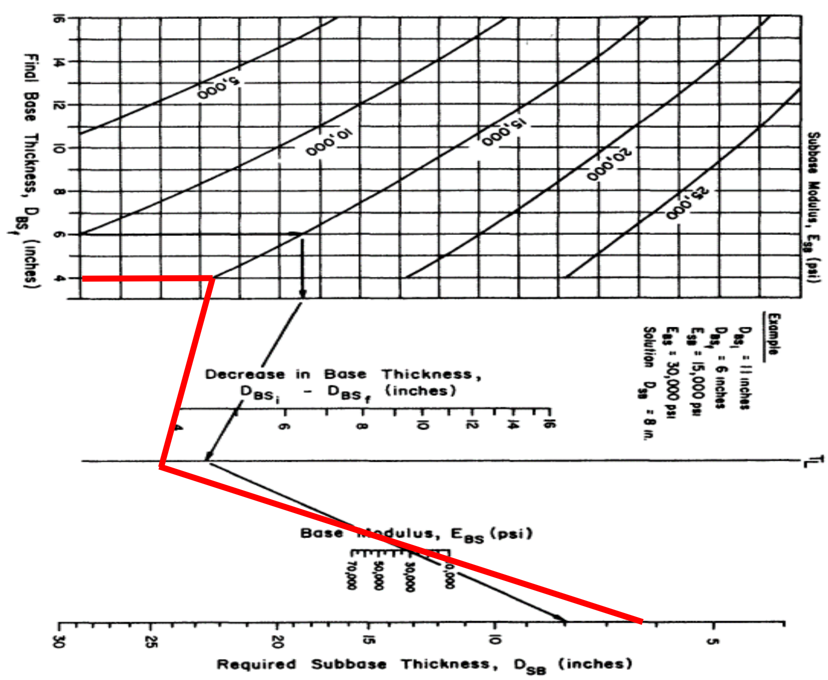


Figure 4.5. Chart to Convert a Portion of the Aggregate Base Layer Thickness To an Equivalent Thickness of Subbase

IRC

Cum. EASL / CBR%	10000-30000	30000-60000	60000-100000
CBR =2 (very poor)	200 100 (mm)	75 150 100 (mm)	75 100 100 100 (mm)
CBR =3-4 (poor)	200 (mm)	275 (mm)	75 100 150 (mm)
CBR =5-6 (fair)	175 (mm)	250 (mm)	275 (mm)
CBR =7-9 (Good)	150 (mm)	175 (mm)	225 (mm)
CBR =10-15 (very good)	125 (mm)	150 (mm)	175 (mm)

Gravel base

Granular sub-base

Improved sub-grade

Bituminous surface treated

Base of Gravel