Feasibility Study of Lean Oil Sand as Base and Surface Material on Gravel Roads in Alberta

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

ABSTRACT

A major concern has been raised by Imperial Oil Inc. that dust issues extensively occurred on the gravel roads in Kearl Lake Oil Sand site in northern Alberta, Canada. Some mitigation measures had been utilized to minimize the dust. Preliminary results showed that the dust has been well controlled by applying Lean Oil Sand (LOS)-granular mixtures.

A site survey has been conducted by the Centre for Pavement and Transportation Technology (CPATT) at University of Waterloo and Imperial Oil Inc. to assess the conditions of selected gravel roads at Kearl Lake site. Recommendations on road maintenance have been provided based on the findings through visual inspection, discussions with site personnel, and physical testing at the site.

The effectiveness of preliminary application of LOS on the gravel roads at Kearl Lake site prompted further research on the feasibility of applying LOS as surface or base material in gravel road design. A suite of laboratory tests, including moisture content test, extraction test, gradation test, proctor test and California bearing ratio test, have been designed and conducted at CPATT to evaluate the mechanical properties of LOS-granular mixture samples with three different mixing ratios. The results show that the bitumen content of LOS provided by Imperial was found in a range of 3-4.5% by weight which can be defined as low graded oil sand. The pure LOS and the 30% granular and 70% LOS mixture are only suitable for sub-base materials. The granular material used on site, 50% granular and 50% LOS mixture, and 70% granular and 30% LOS mixture are applicable for both base and sub-base materials. However, none of these materials are suitable for surfacing material due to lack of fine aggregates. The CBR values of the granular-LOS mixtures are mainly dependent on the granular percentage. Higher granular content generally results in a higher CBR value.

Pavement structural thickness design analysis has been performed considering the introduction of LOS. The AASHTO design chart method is converted into an equation-based method using digitization and regression analyses. A parametric study showed that for gravel roads that require high performance, LOS can be used as a binding agent and mixed with granular materials for mitigating the dust effect. A low mixing percentage of LOS such as the 70% granular and 30% LOS mixture should be used in this case to maintain the strength of base layer. For gravel roads which require a relatively lower performance, a higher amount of LOS may be applied into the mixture with granular materials. In this situation, the introduction of LOS can both save the cost of granular materials and mitigate the dust effect.

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Last but not least, I must express my profound gratitude to my parents and to my husband for providing me with unfailing support and continuous encouragement throughout my two years of study and through the process of researching and writing this thesis. This accomplishment would not have been possible without them.

DEDICATION

This thesis is dedicated to my family, whose love and support have made this a possibility.

Thank you, Dad, Mom, and Wei!

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Chapter 1
Introduction

A gravel road is a type of aggregate surfaced road structure which is typically composed of varying amounts of crushed stone, sand, and fines. Gravel roads extensively exist in the rural areas of North America. Approximately 60% (626,000 km) of the public road network in Canada consists of gravel roads (Statistics Canada, 2003), and gravel roads constitute about 53% (2.6 million km) of the national road network in the United States (FHWA, 2015).

Gravel roads generally carry lower volumes of traffic with a typical Annual Average Daily Traffic (AADT) less than 400 vehicles per day. However, lower volumes of traffic does not mean less importance. Gravel roads play a crucial role to provide access to remote and rural areas, which meets basic social and economic needs (Rashedi et al., 2018). Compared to paved roads such as asphalt or concrete pavement, gravel roads which can provide the basic transportation of local residence and industry are of simple structure type and are easy to construct. As a result, they often involve less design inputs (Almeida et al., 2007). In addition, construction of gravel roads usually requires simple equipment and lower operator skill levels which may lower the cost (Skorseth & Selim, 2005). When the volume of traffic is low, paving and maintaining a paved road becomes economically infeasible so that gravel roads can be a good option. Properly maintained gravel roads can serve traffic demands for many years. Another advantage of building gravel roads in Canada is that they are less vulnerable to freeze/thaw damage than asphalt roads. Although freeze/thaw is damaging as with other roads, maintenance is generally simpler.

Due to the structure configuration and material used for gravel road construction, some shortcomings exist in gravel roads which therefore require more frequent maintenance than paved roads. Vehicle motion shoves the surface gravel to the side of lanes, resulting in aggregate loss and rutting. The accumulation of loose aggregates at roadside will change the designed surface slope and may cause drainage malfunction. Furthermore, the deterioration will be aggravated when the road is in wet condition and when accommodating increased traffic. Simple re-grading, which pushes back gravel material back into shape, is usually sufficient to maintain a gravel road.

1.1 Research Background

The Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo and Imperial Oil Inc. (Imperial) had been communicating since 2013 regarding the condition of roads at Imperial's Kearl Lake site in northern Alberta. Imperial's Kearl Oil Sand site is located near Fort McMurray, Alberta, Canada. The site is devoted to an oil sand project which includes a large mine, processing and storage facilities, tailing ponds, and various building related to maintenance, administration and housing. The roads within the site serve various purposes for the project, such as site access, personnel transportation, and heavy haul roads. For the purposes of the current study, heavy haul roads were not investigated. All of the roads are made of gravel.

The most significant traffic loading consists of mostly of personnel transport in the form of buses. The Average Annual Daily Traffic (AADT) was estimated to be between 100 and 200 vehicles, with 20% truck traffic. However, it can be observed that there is a significant increasing above the original design traffic loading both in terms of volume and percentage of truck traffic, which contributes to the degradation of the roadway. The site is experiencing difficulties in maintaining the functional quality of their access roads. In most sections, the failure is experienced in the top 150-200 mm of the gravel surface course.

The major concern of Imperial Oil Inc. is that dust issues extensively occurred on the gravel roads at the site. Dust can present a safety concern for vehicle travelling behind other vehicles, by significantly reducing visibility. Dust can also be a concern for vehicle maintenance, as air intakes and filters can quickly become clogged with material. This increases the amount of required maintenance, and can affect vehicle performance. Gradually, the surface gravel may lose binder in the form of road dust which is also detrimental to the road structure integrity.

Some mitigation measures had been utilized to address the dust issue, for instance, strategically timed watering, application of dust suppressants, and surfacing a layer of Lean Oil Sand (LOS) or using LOS as a binding agent and mixed with granular material to form an asphalt-like material for construction. Preliminary results showed that the dust has been well controlled by applying the LOS-gravel mixtures. Therefore, it could be said that the LOS has been seen to be a successful dust suppressant. It is possible that future design may use the material successfully, with some modifications to the construction and geometric properties.

Oil sand is a natural mixture made up of sand, clay, and water and is covered with heavy oil called bitumen. The world's largest oil sand deposits are located in Alberta, Canada. There is approximately 1.7 to 2.5 trillion barrels of bitumen resource in place in three major deposits: Athabasca, Cold Lake and Peace River. Oil sand bitumen can be recovered by two methods: surface mining and in-situ technology. The only deposit that is shallow enough that can be surface mined is in the Athabasca area. An in-situ recovery is used to access oil sands deposits that lie deeper than 75m below the surface. About 80% of Canada's oil sand bitumen is recovered by the in-situ technology (Government of Alberta, 2016).

Based on the study conducted by Oil Sand Discovery Center under Government of Alberta, the bitumen content in oil sand varies from 1% to 18%. High bitumen content is more than 12%, less than 6% bitumen is considered as poor grade oil sands and not valued to mine economically (Government of Alberta, 2016). Anochie-Boateng & Tutumlue (2012) proposed that if the bitumen content in oil sands higher than 16% by weight, it is called high grade oil sands, and if the content is less than 9%, it is defined as low grade oil sands. Low grade oil sands are only used in the unbounded material for roads for operating haul trucks and equipment in oil sand fields.

Kearl Lake Oil Sands site is one of the highest quality oil sands deposits in Alberta, Canada. It has an estimated 4.6 billion barrels of bitumen resource, which could supply North America's energy needs for more than 40 years. The initial development in Kearl site began in April 2013 and then expanded in mid-2015, which leads to a production capacity of 220,000 bpd (Imperial, 2018). Oil sand mining requires removal and stockpiling of significant volumes of near-surface overburden material that is placed in large, above ground deposits. The overburden material such as Lean Oil Sands (LOS), cannot be processed economically but has low grade bitumen content in it, usually less than 7% by weight.

Placing a new gravel surface or increasing the thickness of granular layer can always be a way to restore the serviceability of gravel roads. However, it requires large amounts of good quality aggregates. It is understood that importing granular material to the site is greatly expensive. While the unit cost of gravel from the Hammerstone Corporation's quarry is estimated to be \$30/m^3, the final price after the material is transported and placed is close to \$100/m^3. The unit cost of LOS can be disregarded since it is locally available.

Therefore, it is meaningful to explore the use of LOS as construction material for gravel roads, which not only saves material cost but also resolves the dust issues.

1.2 Objectives of This Study

The objective of this work was to evaluate the feasibility of applying Lean Oil Sand (LOS) as surface or base materials for gravel roads through:

1. Reviewing stabilization techniques for improving structural integrity of gravel roads.
2. Assessing the condition of gravel roads by site survey; identifying typical issues for gravel roads, and advantages and shortcomings of the existing LOS application on gravel roads.
3. Conducting a suite of laboratory tests to evaluate the physical properties of LOS and LOS-granular mixtures.
4. Evaluating the feasibility of using LOS as base or surface materials on gravel roads by analyzing the test results.
5. Evaluating the effect of incorporating LOS on gravel road thickness design of base and sub-base course.

1.3 Organization of This Study

This thesis consists of seven chapters. A brief description of each chapter is included below.

Chapter 1: An introduction of research background and objectives of this study.

Chapter 2: Literature review of the structure and materials of gravel roads, drainage requirement, surface distresses, stabilization techniques on gravel roads, and current application of oil sands in geotechnical engineering.

Chapter 3: A condition survey of gravel roads at Kearl Lake oil sands site in Alberta.

Chapter 4: Conducting a suite of laboratory tests including moisture content test, gradation test, extraction test, proctor test, and CBR test on the LOS, granular material and LOS-granular mixtures.

Chapter 5: Analysis and discussion on laboratory test results for the LOS, granular material and LOS-granular mixtures.

Chapter 6: Developing an equation-based design method for gravel roads from the chart-based method. Performing parametric studies on thickness design of gravel roads and providing recommendations on incorporating LOS.

Chapter 7: Conclusions of this study and recommendations for future research related to the use of LOS on gravel roads.

Chapter 2
Literature Review

2.1 Gravel Road Structure

The typical cross section of a gravel road is shown in Figure 1. There are three basic components including crown, shoulder and ditch that must be followed as minimum requirements. An ideal crown is approximately 4% of cross slope and keeps a straight line from centerline down to the shoulder. A cross fall about 6% of shoulders is to further drain water to foreslope and ditches. The down edge of the shoulder should keep the same level of gravel surface, any higher slope is prevented which can result in a secondary ditch and excess water will weaken the subgrade and gravel material. Besides crown and shoulder, a ditch with flat bottom is needed to further collect water. Ditches are required at least 30cm below the top of subgrade. It is required to clean eroded soil or debris accumulated in the ditches regularly.

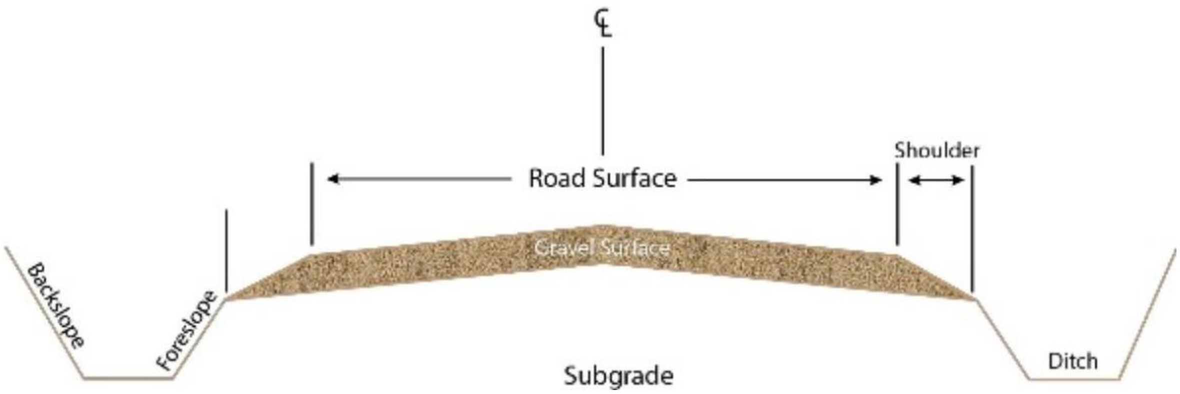

Figure 1 Typical Cross Section of Gravel Roads (Skorseth & Selim, 2005)

The building of cross section is a basic operation and must be properly maintained. Otherwise it will not meet the performance even in very low traffic condition. The primary objective of a proper cross section is to drain water away from roadway and provide a safe driving surface (Rashedi, Maher & Barakzai, 2018). The roadway culverts and bridges are also very important components for drainage purposes. Collapsed culverts or filled culverts with eroded soil and debris are bridges need to be cleaned through routine maintenance so that the drainage is unobstructed (Skorseth & Selim, 2005).

2.2 Materials

The road construction gravel is generally composed by three types of aggregates including gravel, sand and fines. The materials are usually from local sources such as quarry aggregate (limestone, quartzite and granite), glacial deposits and river run gravels (Skorseth & Selim, 2005). In general, the mixing percentage is 40-80%, 20-60%, and 8-15% for gravel, sand and fines, respectively (Woods, 1960). Aggregates are mixed with proper proportion and are compacted firmly to be placed above the subgrade, which provide a hard surface to resist traffic loads. There are two types of gravel surfaces, which are defined as traffic gravel and structural gravel.

2.2.1 Traffic Gravel

The traffic gravel is placed on top of subgrade. The function of traffic gravel is to add an all-weather surface for traffic and remain reasonable stable, but it provides little or none strength support. It contains a relative high content of fine materials with plasticity, such as natural clay, which gives binding characteristic to bound materials tightly especially in dry weather. However, the gravel surface will never perform like flexible pavements in which bituminous portion as binder to form a crust surface. Fine aggregate is easily lost and floats on surface causing dust and other distresses under traffic on gravel roads (TAC, 2013). Roads that serve light traffic, for example, minor access roads, rural agricultural access roads, recreational and scenic roads are mostly applied with this type of gravel surface (AT, 2010). The design thickness of traffic gravel varies and depends on different regional standards. It typically ranges from 20 to 40 mm in western Canada and 20 to 25 mm in eastern Canada (AT, 2010).

2.2.2 Structural Gravel

The function of structural gravel is primarily to provide structural strength to carry traffic loading, and it also gives a travelling surface. It contains a relative high amount of larger top sized gravel material for good strength purpose. Most major access roads, heavy traffic loading requests of industrial and commercial roads are chosen to be constructed by this type of gravel. Unlike the single layer of traffic gravel, structural gravel usually contains base and sub-base course layers similar to conventional flexible pavements.

2.2.2.1 Sub-base

Sub-base is a layer of gravel placed right above the surface of subgrade. The design purpose is to distribute traffic loads from overlying layers to supporting underlying embankment. It is the second main load-carrying layer after base layer. Furthermore, it has a non-frost susceptible layer to drain water away from base layer which protects subgrade. The sub-base layer is optional since it provides additional efficiency of distributing loads. (TAC, 2013).

2.2.2.2 Base

Base is a gravel layer placed on the top surface of sub-base. In some cases, agencies do not use sub-base layer, so that base layer is directly constructed onto subgrade. Similar to sub-base, the function of base is to transfer loads from surface layer to underlying layers of the structure and help to drain water away from surface layer. A base layer requires higher strength capacity therefore it is composed by higher quality aggregates than those for the sub-base layer since it is closer to traffic loading (TAC, 2013).

The thickness design method of structural gravel can be followed by flexible pavement design guide except for not placing asphalt concrete on surface layer. The general concept of the conventional asphalt pavement design is to satisfy two major criteria: the tensile strain on the underside of the asphalt shall be within the limit to prevent fatigue cracking, and the vertical compressive strain in the subgrade should be controlled to satisfy the limit rutting. The thickness of the asphalt pavement is primarily dominated by the tensile strain on the underside of the asphalt for thin pavement, but rutting in the subgrade is mainly dependent on the thicknesses of base and sub-base. Therefore, the design criteria for gravel road thickness design are mainly based on the vertical compressive strain criterion used in the conventional flexible pavement design (TAC, 2013).

Factors which influence the thickness of structural gravel include strength of subgrade, depth of frost, freeze-thaw cycles, spring load restrictions, winter weight limits, effective of drainage and percentage of truck traffic (TAC, 2013).

Since traffic gravel and structural gravel serve for different purposes, the requirement of graded aggregate for surface and base layers may not be identical. Same graded aggregates may not perform well on both surface and base layers. The judgment on design of gravel surface is based on purpose of the road and desired level of service from agencies.

2.3 Distress Types of Gravel Road

Due to the structure configuration and material used for gravel road construction, some distresses commonly occur on gravel roads including dust, loose aggregate, soft spots, washouts, washboarding, potholes and rutting. Unlike paved roads, distresses on gravel surface are revealed more rapidly. Therefore, gravel roads require more frequent maintenance. A maintenance program is helpful to resolve deterioration of road surface in a timely manner.

2.3.1 Dust

Dust is one of the most significant concerns on gravel roads. Dust is an inherent surface defect of dry gravel roads, and is due to both gravel composition and low moisture on road surfaces. Fines can be easily dislodged, and become airborne when vehicles pass quickly on a gravel road (Walker, 1989). Also, dust can be caused by climatic factors, such as low precipitation and high temperature which contributes to the dry surface, such as arid and semi-arid areas (Gebhart, Denight &Grau, 1999). Some typical negative effects of the dust problem on gravel roads are described as follows:

- Road safety: dust cloud can be formed and it lowers the visibility for drivers (Figure 2), thus increases the risk of traffic accidents.
- Air pollution: it is caused by fine suspended dust particles, which can affect human respiratory health and cause allergies. It can be harmful for residents in buildings adjacent to roads.
- Ecosystem sustainability: dust can contaminate nearby surface water by increasing turbidity and also clog the pores of plants and therefore impede crop growth (CTIC, 1989).
- Need and cost for vehicle maintenance: fine abrasive particles can considerably increase the wear and tear on the moving parts of vehicles. Also, uneven road surface leads to increased fuel costs and travel time (Carlsson, 1986).
- Road maintenance: a long-term dust problem can result in the instability of a gravel road due to the significant loss of fine material from the well-graded gravel surface leaving the coarser aggregate without a binding material. Therefore, more severe distresses such as potholes, washboarding and raveling can happen, which requires relatively high maintenance cost (Walker, 1989). Moreover, it can also reduce riding comfort for road users.

Figure 2 Low Visibility Resulted by Dust (The City of Grande Prairie, 2018)

Placing and compacting a new gravel surface is a solution to control dust, but the problem will arise again under heavy traffic and dry weather. Therefore, stabilizing surface gravel is a more effective solution to resolve the dust issue.

2.3.2 Loose Aggregate

Heavy dust can also result in loose gravel or aggregate. Improper gradation of the original gravel also causes this problem. Loose gravel produced with the action of traffic tends to accumulate and form ridges at the center of lanes and at the shoulders (Figure 3). Excessive loss of gravel creates a deteriorating effect on the surface condition and side slopes, severely weakening the traction, strength of a road, and causing future washboarding (AG, 2000).

To avoid potential problems caused by loose aggregate, Walker (1989) suggested that a well-graded mix of gravel is required to start with, and dust conditions should be controlled. On average low volume roads, re-gravelling the roadways is necessary once every three years if loose gravel leads to excessive loss of material. However, for specific cases where the traffic volume is much higher than typical, a more frequent maintenance schedule may be required (AG, 2000).

Figure 3 Loose Aggregate on Roadway (Zhang, 2009)

2.3.3 Soft Spots

When the gravel base is not sufficiently strong, soft spots appear on a gravel road where wheels sink in and create ruts on the surface (Walker, 1989). The problem is usually aggravated by poor drainage and heavy traffic, which can eventually lead to the failure of a road. This issue creates a poor driving experience, and the water trapped in the spots is also a potential hazard to motorists (Figure 4). Soft spots are very likely to develop into future pothole problems. While blading and rehabilitating the crown can correct small soft spots, a typical method for severe instances is to improve the drainage system and to reinforce the gravel base with better quality aggregate (Walker, 1989).

Figure 4 Soft Spots on Outside of Wheel Path

2.3.4 Washouts

Localized washouts describe road surface gaps formed due to too much water flowing in a narrow channel over soft material (Figure 5). Causes contributing to these problems include gravel displaced by traffic, erosion of material after heavy rains, and winter plowing operations (Skorseth & Selim, 2005). As the washouts grow, not only can the driving condition be hazardous, but the road would eventually be impassable.

The key for rehabilitation is to reshape the road surface and the shoulder by cutting gravel with a grader to the proper crown. With additional aid of a roller for compaction, the finished surface will be denser, stronger, and more smooth (Skorseth & Selim, 2005).

Most washouts encountered as part of this project were confined to the slopes beyond the road's shoulder. These issues are due to the grade of the slope combined with the low-cohesion nature of the soil in wet conditions. The washing out of these slopes leads to sediment accumulation at culverts and collection areas and loss of structural capacity at the edge of the roadway. Generally, these issues are solved using slope stabilization methods, such as vegetation, geotextile meshes, and rip rap or gabion baskets.

Figure 5 Localized Shoulder Washout

2.3.5 Washboarding

Washboarding or corrugation describes an effect where evenly spaced ripples are formed on the gravel surface in the transverse direction of a roadway (Figure 6). Three primary causes are lack of moisture, aggressiveness vehicle acceleration and braking, and poor quality of gravel (Skorseth & Selim, 2005). Soft roadbeds and improper grader operation can also contribute (Walker, 1989). Although washboarding does not make the passing vehicles bounce as severely as potholes, it does create a very uncomfortable ride, and can result in loss of control at higher speeds, similar to hydroplaning. Severe washboarding can also trap water, possibly leading to worse road issues and loss of vehicle control.

Figure 6 Centerline Wash Boarding on Gravel Road (Skorseth & Selim, 2005)

Based on the manual created by Skorseth & Selim (2005), good quality of gravel should have the right combination of fractured stone, sand, and fines of high plasticity. However, corrugation is always a problem even with the best of maintenance, and therefore it is an on-going concern. The best timing for repairing washboarding is usually after a rain due to an increased moisture level. While light washboarding can be eliminated with routine grading, the general solution for properly reducing the effect is to reshape the affected areas when they are damp. The steps are to cut the gravel to one inch deep below the corrugation, blend, and replace it to the right shape and compaction level (Skorseth & Selim, 2005).

As Walker (1989) suggested, crowns and super-elevations should be carefully maintained when fixing spot corrugations. It is essential that a grader operates at below 15 km/h. Additionally, dust suppressants can be applied to hold moisture, helping to strengthen the soil to avoid washboard and other problems.

2.3.6 Potholes

Potholes are bowl-shaped depressions or holes that develop in the gravel or surface when surface material is worn away or soft spots develop in underlying soils (Figure 7). The two main causes are the presence of water under the road and the action of traffic. It is often an inadequate crown that causes water to pond on the road. As water weakens the underlying soil, the forces exerted by the passing traffic in the affected area eventually damage the poorly supported gravel surface, forming the potholes (Walker, 1989).

Figure 7 Potholes on Wheel Path (FHWA, 2015)

The potholes in roads make passing vehicles bounce, which slows the traffic and creates a dangerous driving condition. The serviceability of affected roads is also greatly reduced. Therefore, it is highly recommended to repair potholes at an early stage by routine re-grading. Simply hand-filling the holes with new gravel and compacting the material is only effective for small isolated potholes, and usually results in that material being quickly displaced by traffic. It is generally required to add granular material and to scarify the area beforehand to ensure a good blend. Furthermore, common methods to prevent future potholes include restoring a crown through reshaping of the road, and improving the drainage system (Walker, 1989).

2.3.7 Rutting

Rutting forms along the wheel path of the road and depression is developed in the longitudinal direction. Repeated vehicle passes over soft spots is likely to further turn into a rutting problem. If the gravel road has a rounded parabolic crown and poor drainage, it will accelerate the formation of ruts. Routine maintenance of re-grading and drainage can mitigate this problem. If the depression depth is larger than 3 inches, it indicates a weak underlying structure strength and soft subgrade. Figure 8 shows an example of severe rutting in gravel roads. Severe rutting requires major rehabilitation or reconstruction. Rutting on gravel roads is sometimes less severe than flexible pavement due to frequent grading routine maintenance (Eaton, 1992).

Figure 8 Severe Rutting (Zhang, 2009)

2.4 Stabilization

Stabilization is widely utilized as an acceptable technique of improving the performance on gravel roads. Due to the shortage of conventional aggregates or quality aggregates in local areas, stabilization operations are often carried out to modify or upgrade marginal materials as economical measures. Stabilizers are applied to treat upper several inches of materials or gravel surface in order to meet the desired strength for anticipated traffic and avoid surface distresses. The primary advantages of stabilization on gravel roads include (Terrel, Epps, Barenberg, Mitchell & Thomopson, 1979):

- Reduce dust and other distresses
- Improve marginal materials
- Increase strength
- Improve durability
- Decrease volume change
- Improve workability
- Reduce thickness of gravel surface
- Reduce maintenance cost

There are mainly two broad categories for soil stabilization: chemical stabilization and mechanical stabilization. Chemical stabilizers are further divided into traditional stabilizers and non-traditional stabilizers. The most common traditional stabilizers include lime, fly ash, cement and bituminous materials. Some typical stabilizers are discussed in this section.

2.4.1 Lime

The form of hydrated lime is used to stabilize soil. Lime reacting with water and the ion exchange within the soil results in changes of soil structure. Lime stabilization is used to reduce volume change, reduce plasticity, increase workability and improve strength. Lime is often used on clayed soils or fine-grained soils that have a plasticity index greater than 15. It does not work well with silts and granular materials because of lack of aluminates and silicates. The application rate ranges of 5% to 6% by weight. Lime is rarely used as surfacing material due to its poor resistance to the abrasive action of traffic. Lime treated soils can increase unconfined compressive strength from 100 to 400 psi (Kestler, 2009).

2.4.2 Fly Ash

Fly ash is a residue of coal combustion. Fly ash contains pozzolans functioning at two different ways, one is self-cementing, and the other one requires an activation agent such as lime or cement for a pozzolanic reaction to bond the soil. Fly ash is not used as a surfacing material but mixed into granular material for base and sub-base layers. Fly ash stabilization can lower the water content and reduce shrink-swell potential of soil, increase the workability and obtain desired strength. The application rate ranges of 10% to 20% by weight. It usually can increase CBR values from 2-3 to 25-35 and unconfined compressive strengths from 100 to 500 psi. However, leaching tends to occur for fly ash and may affect ground water and nearby surface water and aquatic species (Kestler, 2009).

2.4.3 Cement

Cement can be used as soil stabilization except for highly organic soils that contain sulfates. It has pozzolanic material that creates a hard and impermeable layer rapidly. Cement stabilizer reduces swell potential, decreases compressibility, and increases strength and durability. Cement is more likely added to base layers rather than to surface material because they are brittle and susceptible to crack under traffic loadings. Cement also can be used for soil modification, which is to improve the quality of marginal materials by decreasing plasticity. However, cement treated bases is not applicable in areas with seasonal frost heave. The application rates range from 3% to 5% by weight by incorporating cement to a granular material. CBR values can increase from 2 to 40. The unconfined compressive strength can be improved from 125 to 500 psi for fine grained soil (Portland Cement Association, 2003). There is another cementitious type called Lean Concrete Base (LCB). It's a mixture of good quality aggregate, cement and water. It's similar to PCC pavement, in that materials are proportioned but the compressive strength is controlled in a range of 5.2 to 8.3 MPa to prevent high curling and warping stresses problems (ACPA, 2007).

2.4.4 Bituminous Materials

Different from cement or lime stabilization, the mechanism of stabilization using bituminous materials is to attribute waterproofing for fine-grained materials, and to provide both waterproofing and adhesion for non-cohesive coarse-grained materials, such as sand and gravel (Terrel et al., 1979).

Fine-grained materials coated with asphalt can thereby prevent and slow down the penetration of water and reduces the decreasing tendency of strength and elastic modulus. The durability can be increased by limiting the volume change in the soil as a result of freeze-thaw cycles. Coarse-grained materials have the similar mechanism of waterproofing, but aggregate particles adhere to asphalt by cohesive forces. Asphalt is acting as a binder to improve strength and stabilize granular materials.

By adding asphalt cement into materials of surface layer as surface treatment, it helps in eliminating the occurrence of surface distresses, such as dust, rutting, washboarding, aggregate loss, etc. Furthermore, the stabilized surface layer will protect the underlying subgrade material by preventing penetration of surface water.

Three typical asphalt materials can be used for granular base stabilization, including asphalt treated base, emulsified asphalt base and foamed asphalt base.

2.4.4.1 Asphalt Treated Base

Asphalt cement is usually heated to blend with granular materials as an asphalt treated base. Since the asphalt treated base is not directly exposed to traffic loads and weathering, it provides a stable layer. However, due to high cost of asphalt cement, the method is rarely used in recent years (TAC, 2013).

2.4.4.2 Emulsified Asphalt Base

Emulsified asphalt is the asphalt cement combined with water and emulsifying agents. Types and quantity of emulsifying agents used will determine the setting properties of emulsified asphalt. Medium to slow setting emulsion blended with granular materials can be used for in-place stabilization (Army, 1994).

2.4.4.3 Foamed asphalt base

Foamed asphalt is produced by adding a small amount of hot water into asphalt cement in a controlled expansion chamber and then blended with aggregate after foaming. One of the most remarkable advantages of using foamed asphalt is that the support surface is available to use after placement and compaction without any curing or setting time, and the placement process is not susceptible to ambient weather (TAC, 2013).

2.4.5 Recycled Asphalt

Recycled asphalt, also called reclaimed asphalt pavement (RAP), can be generated from existing or old asphalt concrete for environmental and financial benefits. During reconstruction or resurfacing of pavement, the old asphalt concrete is crushed and screened into a partially coated granular material which usually consists of high-quality, well-graded aggregates coated by asphalt cement. Therefore, this material may be used in new asphalt concrete or as base course (FHWA, 1998). Recycled asphalt is one of economical replacements materials when shortage of conventional aggregates is encountered. Recycled asphalt materials can improve shear strength, frost susceptibility, stiffness and durability on unbounded pavement layers (Saeed, 2008). A minimum of three inches compacted recycled asphalt should be placed as surfacing layer on gravel roads that have strong subgrade (Skorseth & Selim, 2005). The standard test methods for reclaimed materials are not available and the standards for natural materials may not be applicable for reclaimed materials. Therefore, the existing performance-based specifications are mainly used for evaluating the effectiveness of recycled asphalt (TAC, 2013).

Koch, Ksaibati & Huntington (2011) constructed several test sections by using recycled asphalt on gravel roads in two Wyoming counties. They found that when recycled asphalt was incorporated in gravel roads, it helped in dust reduction significantly with no adverse effects to roads' serviceability. Also, recycled asphalt is a cost-effective material by reducing the amount of granular materials and reducing maintenance cost of applying dust suppressant on gravel roads.

Mahajan (2015) states that CBR is the most appropriate test to evaluate recycled asphalt incorporated in gravel road surfacing materials. Both laboratory mixes and test sections samples on Goodhue County indicated that the CBR value of mixtures of recycled asphalt and aggregates decreased with increase of recycled asphalt percentage. Furthermore, a 28% dust reduction can be achieved by using a 50% recycled asphalt mixture in Carlton County. Both initial construction cost and maintenance cost will be reduced when incorporating recycled asphalt material.

2.4.6 Chlorides

A variety of non-traditional stabilizers have been developed and examined due to high cost and large demand of traditional stabilizers. Based on chemical components, non-traditional stabilizers can be grouped into seven categories, including chlorides, lignosulfonates, petroleum resins, ionic, enzymes, polymers, and tree resins. Chlorides which are simple to place are commonly used non-traditional stabilizers.

The mechanism of chlorides stabilization is the hydroscopic and deliquescent reactions. Chlorides absorb water from the air and under the road to keep gravel surface moist and resist evaporation. It is either used by spraying on the surface or mixed into aggregate in-place. Chlorides are most commonly used for dust control in gravel roads and are applied either in dry flakes or liquid solution. It is estimated that 95% of chloride-based dust suppressant is $CaCl_2$ in Canada in the year of 2000 (Environment Canada, 2005). However, for stabilization purpose, the application amount is about 3-5 times than that used for dust control. $CaCl_2$ is more effective at higher humidity while $MgCl_2$ works better in dry weather. Chlorides have high potential of leaching in rainfall. One major advantage of chlorides treated roads is that it can be re-compacted after being sheared by heavy vehicles and be recovered to its original constructed condition, which is called self-repair (Edvardsson, 2009). However, the shortcoming is that it is corrosive to metals so that it increases the potential damage to vehicles. Survey results presented by Birst and Hough (Birst & Hough, 1999) showed that the effective duration of $CaCl_2$ is 71% for 3-6 month and 21% for 6-12 month; and that of $MgCl_2$ is 33% for 3-6 month and 42% for

6-12 month. This indicates that chlorides stabilization is not effective after one year and is required to be reapplied frequently especially under increased truck traffic and vehicle speed.

2.4.7 Geosynthetics

Geosynthetics which are conventional mechanical stabilization techniques are widely accepted in practice. Geosynthetics are usually installed on top of subgrade in order to avoid aggregate and subgrade soils intermixing. There are two major functions of geosynthetics: separation and reinforcement.

Separation is essential to maintain the load carrying capacity and stability for base course. Geotextiles and geogrids are commonly used as separators to prevent penetration of the aggregates into the subgrade which results in localized bearing failures. Geotextiles are also capable of preventing intrusion of subgrade soil into base course aggregate. Geotextiles are mainly used for separation purpose, and sometimes for reinforcement. Geogrids are stiffer and more durable so that they can contribute to reinforcement of road structure. The primary benefits of reinforcement are to improve shear strength between surfaces and enhance aggregate interlock (Holz, 1998).

Based on Fannin & Sigurdsson's (1996) findings from field performance that a geosynthetic was most beneficial for the thinner gravel base layer of 25cm. And the effectiveness decreased with increasing of layer thickness. Kestle (2009) provided a solution of installing geotextiles due to a shortage of quality aggregates, which can reduce required amount of aggregate by 25%. If geogrids and geotextiles are both used, the amount of aggregate required can be reduced by 50%.

2.5 Application of Oil Sand on Geotechnical Engineering

Samieh & Wong (1997) has studied deformation behavior of Athabasca oil sands at low effective stresses under varying boundary conditions. They found that the stress-strain relation is dependent on the boundary conditions and the slenderness ratio of specimen in drained tri-axial compression tests. The results of oil sands are consistent with those obtained from dense sands. Wong (1999) examined the microstructural characteristics of sheared specimen which were used to further explain how the macro-deformation responses of Athabasca oil sands found in tri-axial compression are affected by local confining stresses with enlarged lubricated ends in drained tri-axial compression tests.

Gwilliam (2010) discussed the feasibility of using oil sands in asphalt pavement from an economical point of view. The cost of hot mix asphalt in-place paving and the cost of incorporating oil sand were compared. The oil sand used in the study was added extra 1% conventional asphalt binder. It has been concluded that there was predictable cost benefit of using oil sands for private industry, local and state government. It also showed that using oil sands is more environmentally friendly by reducing emissions due to a lower mix temperature of 104°C compared to a mix temperature of 177°C for asphalt cement.

Anochie-Boateng and Tutumluer (2012) has conducted a suite of laboratory tests on three types of oil sand with different bitumen contents to investigate the strength, elastic modulus and deformation characteristics of the materials under realistic traffic loading and climatic conditions. A suite of laboratory test procedure has been developed for oil sand material. Material characterization and performance models have also been developed to characterize field behavior of oil sand material under both static and dynamic load conditions.

Vrtis (2013) has performed a feasibility study on incorporating oil sand, which acts as binder material to replace the conventional asphalt binder, into an open-graded aggregate for a performance-based mix design. A field trial was also performed to realize the laboratory mix to full-size scale. Mix samples obtained from the field project have also been measured and compared with the laboratory mix results and the results from conventional hot asphalt mixes. The bitumen content of oil sand used in this research was measured to be 12.97% by weight. It proved that a mix consisting of 67% aggregate and 33% oil sand showed potential to be used as paving material for rural or low volume roads. However, it also found the variability in the test results due to the inherent properties of natural unmodified material of oil sands.

2.6 Research Gaps

A few researchers have investigated the strength, modulus and deformation characteristics of oil sands by laboratory and field tests, and the feasibility of using oil sand in asphalt pavements. All the results showed that there is potential use of oil sands as a paving material by replacing conventional asphalt binder. Applying oil sands as binder on pavement is environmentally friendly and consumes less energy than hot mix asphalt used in flexible pavement surface courses. However, there are no studies on utilizing low-graded oil sand such as Lean Oil Sands on low volume roads or gravel roads.

LOS which generally has bitumen content less than 7% is an underutilized nature resource and is extensively available in northern Alberta. Preliminary application of LOS on the gravel roads at Kearl Lake site has demonstrated that the LOS can be a successful dust suppressant. It is meaningful to further study on incorporating LOS in the future design of gravel roads.

Chapter 3
Road Condition Survey at Kearl Lake Site

The Centre for Pavement and Transportation Technology (CPATT) at University of Waterloo and Imperial Oil Inc. (Imperial) had been communicating since 2013 regarding the condition of roads at Imperial's Kearl Lake site in northern Alberta. Roads within the site serve various functions to support the operation of the plant. For the purpose of this study, heavy haul roads were not investigated, and all roads of interest in this study are generally categorized as gravel roads. Significant dust, which presents a number of issues on driving safety and maintenance cost of vehicles, had been encountered on many of the roads at site. In order to identify the existing condition of the roads and develop appropriate mitigation strategies, a site survey was conducted in May 2017 by CPATT and Imperial. The existing condition of a number of selected gravel road sections was evaluated through visual inspection, discussions with site personnel, and physical testing.

3.1 Description of Site Road Sections

Six road sections chosen by Imperial that were specifically considered for the survey. They were named Pit-4 Road, Control Room Road, K2 Laydown Road, OPP2 Access Road, Main Plant Access Road (MPAR), and Canterra Road.

Pit-4 Road is located on the eastern edge of the site. The traffic is mostly light volume, but includes tri-axle vehicles, graders and excavators. It has several sections running past material stockpiles and one uphill, which is a north-south section. The uphill is the main evaluation area. On the eastern edge, a dug ditch separated the road from bush. On the western edge, a slight slope encountered with a small berm. Beyond the berm, the area was deeply excavated. The slope of the crowned road was estimated to be approximately 2-3%. Figure 9 shows the section which was investigated, view from north to south.

Control Room Road is an east-west orientation and adjacent to the plant area. Parking lots and equipment sheds are along with south edge of the road. Plant facilities are along the north edge of the road. Dug-out ditch is along each side of edge, while culverts were constructed on driveways. Figure 10 shows the Control Room Road section, view from east facing west.

Figure 9 Pit-4 Road (south-facing view)

Figure 10 Control Room Road (west-facing view)

K2 Laydown Road is an east-west section and located on south of on-site work management offices. In south side of the roadway, a pipe lay-down area was located which provides effective drainage. In the northern side, the roadway was lined with concrete blocks to prevent vehicles from falling down the steep slope. A crown was constructed on this section. Figure 11 shows the layout of K2 Laydown Road.

Figure 11 K2 Laydown Road (west-facing view)

OPP2 Access Road is a north-south road section. There was no significant crown observed. On the western edge, a gradual slope down and away from the road. On the eastern edge, there was a shallow ditch adjacent to the road, and there was a berm constructed beyond it. The purpose of the berm was assumed to provide a physical barrier between the natural water area with vegetation and pollutants from the roadway. But it appeared that the berm had a negative effect on the road by allowing rainfall to pond and gradually seep into supporting layers, which may lead to weak foundation. Figure 12 shows a portion of this section. The material of LOS was also applied on this road. Unlike the other LOS applied section, LOS was mixed with granular material and treated as a binding agent. The mixed combination was 50/50. The surface was graded by a roller-compacted in place.

Main Plant Access Road (MPAR) runs from Canterra Road into the project site. The road passes through the areas of muskeg and forest. In general, large buses transport people to airstrips and off-site housing. Moreover, trucks carry materials and supplies to site. Figure 13 shows the layout of MPRP road. It is a gravel road and has well-maintained ditches, also a center crown.

Figure 12 OPP2 Access Road (south-facing view)

Figure 13 Main Plant Access Road (MPRP) Road

Canterra Road is a provincial road with a higher speed limit of 70km/h and it also used by other companies. Canterra Road runs from the East Athabasca highway in the southeast, almost to Route 63 in the northwest, near Fort Mackay. It’s about 50km of gravel road. Figure 14 shows southeast facing view of Canterra Road, it has gradually sloped shoulders adjacent to trees.

Figure 14 Canterra Road (southeast facing view)

3.2 Findings of Road Condition Survey

The selected site roads mainly serve site access and personnel transportation. All of the roads are gravel roads. The pavement strength, surface roughness and distresses were evaluated by physical testing and visual inspection.

3.2.1 Pavement Strength Evaluation

The Light Weight Deflectometer (LWD) device is used to measure the stiffness and strength of in-situ pavements. Stiffness is a good indicator of structural capacity, but may change drastically due to a number of factors, especially in gravel roads which are permeable to water. LWD is operated by one person, highly portable and light load impact device, which can provide a single point deflection measure. The applied load and deflection are recorded through a portable computer, and linked to a GPS location. The LWD data were collected on each of the inspected sections at selected spots on both lanes. It provides the relationship between design modulus and the actual site moduli value. The surface deflection modulus is computed using the relationship defined by Equation (1) (Ullidtz, 1987). It is calculated directly under the point of loading at the maximum deflection. Figure 15 shows an engineer conducting a LWD test on a road section.

$$E_0 = \frac{f \times (1-\mu^2)\sigma_0 a}{d_0} \quad \textbf{(1)}$$

Where,

E_o is surface modulus at maximum deflection.

f is factor for stress distribution and a value of 2 is used here.

μ denotes Poisson's ratio usually chosen as 0.35.

σ_o represents physical thickness of a layer.

a is radius of the loading plate; and d_o denotes center deflection.

Figure 15 Light Weight Deflectometer (LWD) Test on MPAR Road

Figure 16 indicates individual surface deflection modulus points on each road section inspected. The mean values and standard deviations and compared in Figure 17. It can be seen that K2 Lay Down road has the highest stiffness, while Pit-4 Road has the lowest mean stiffness with the lowest standard deviation, which means the low pavement strength on this road is not a localized issue. This might indicate a design or construction concern in the area. Control Room has the second highest average modulus value. Three MPAR sections that had chemical dust suppressant treatment had relatively consistent surface stiffnesses. OPP2 Access has the similar value as MPAR road. The surface modulus on Canterra Road exhibits large variability and is generally low. It may be caused by the fact that the inspected section was adjacent to an area of standing water. The drainage issue should be recognized and addressed.

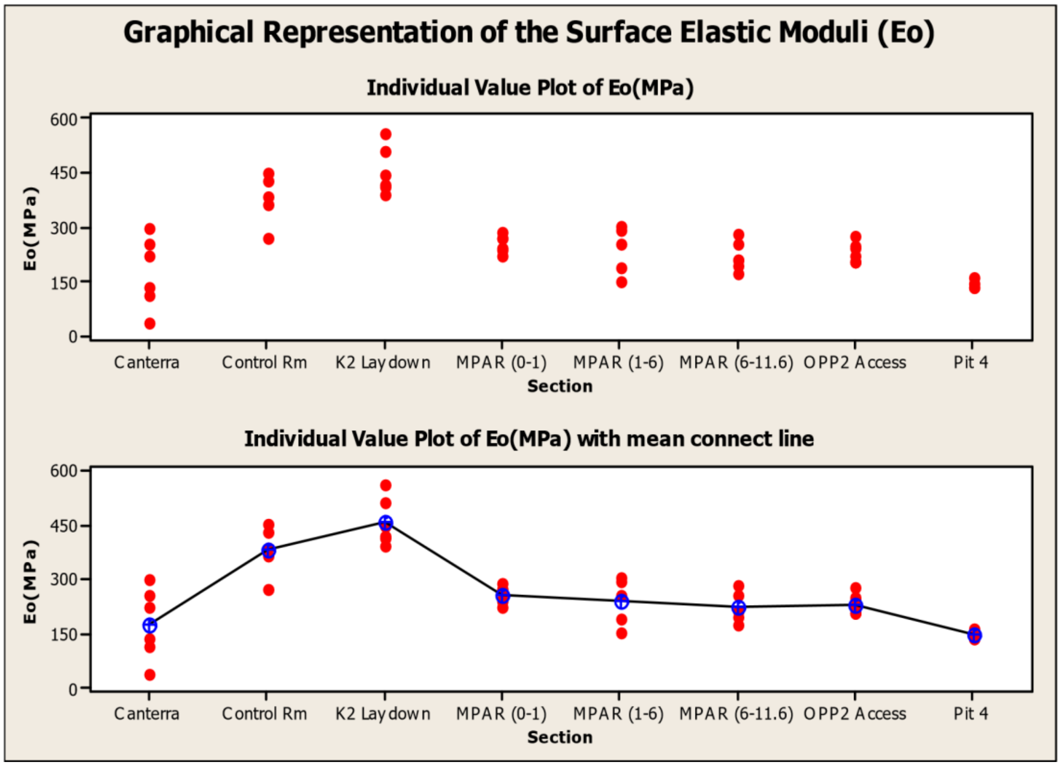

Figure 16 Surface Elastic Moduli of Road Sections

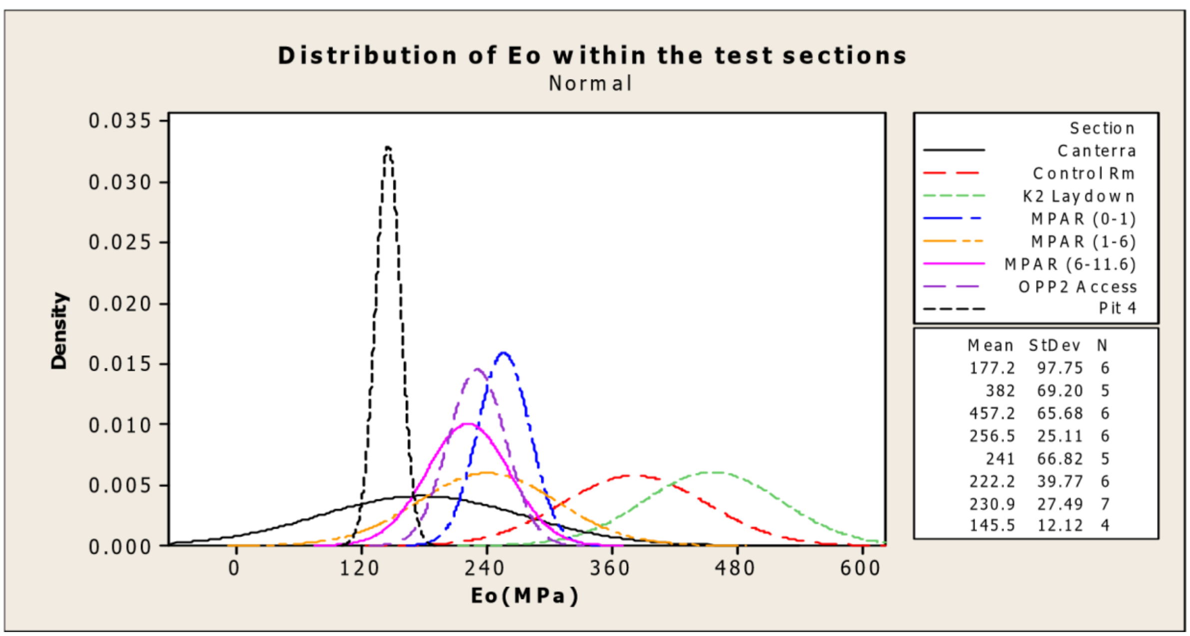

Figure 17 Distribution of Surface Deflection Modulus E_0 within Test Sections

3.2.2 Pavement Roughness Evaluation

Roughness is a good indicator of ride quality of a road surface. The ride quality is often more concerned in high volume of roads as trigger point for maintenance. In this case, it can have effect on vehicle maintenance. Low roughness value may lead to significant jarring forces on vehicle travelling at speed, causing vehicle maintenance issues. The improvement of roughness deterioration would result in savings in both user costs and agency costs. The roughness of the roads on the site was measured using hardware and software provided by Rival Solutions. The hardware included a hood-mounted camera which took photographs of roadways throughout the site, and a dash-mounted device which measures the roughness of the roads as measured from the vehicle. The photographs can be used for evaluating locations at later dates remotely. The software associated with these devices allows all readings to be geo-referenced to a GIS map, allowing for easily reviewable data. The software converts roughness data into roughness index (rufindex) and then as raw data calculated to Road Condition Index (rRCI). rRCI values rate on a scale of 0-10. 0 means very poor condition and 10 means new construction condition. A higher rRCI value represents a smoother surface. The rRCI scale is generally defined by a user as the hardware is often customized for usage. A detailed description of rRCI values are presented in Table 1.

Table 1 rRCI Value Descriptions

rRCI Value	General Rating	Description
0-2	Very Poor	-Very rough throughout entire interval -High Risk Condition to operator safety and machinery damage
2-4	Poor	-Rough throughout entire interval
4-6	Moderate	- Operator behavior beginning to be affected by roughness (vehicle slowing, avoiding distresses, etc) - Good candidate for maintenance in PMS
6-8	Good	- Good, no need for maintenance
8-10	Very Good	- Very smooth condition - New construction condition

A rRCI value of 6 can be treated as the trigger point for maintenance, below this point, vehicle drivers begin to be affected, including reducing speeds, and avoiding surface distresses. However, there are no standards providing minimum acceptable roughness value for gravel roads. The threshold value for maintenance can be set according to available equipment or machinery. Otherwise, it can be adjusted by experienced engineers who define the value meaning in the field. Five roads were measured except for Control Room Road. Data was collected on every 50 m interval. Figure 18 to Figure 22 show rRCI and rufindex values for each road. If a trigger point of 6 is chosen, such as for Pit-4 Road, maintenance activities could be focused on the road between 0-0.1km, 0.15-1.2km, 0.425-0.5km as show in Figure 18. In general, maintenance activities are based on a larger segment, for example, 1-2km. Table 2 summarizes values considering the homogeneous segments on each road. The roughness data collected by hardware were used to identify homogeneous segments of the inspected roads. Delineating the road into homogeneous sections allows for consideration of the road in sections that are behaving generally consistently, in terms of roughness. In Table 2, Pit-4 Road has no sections requiring maintenance. In either option, this focused maintenance planning can make for more efficient work to avoid reaching the poor to very poor stages.

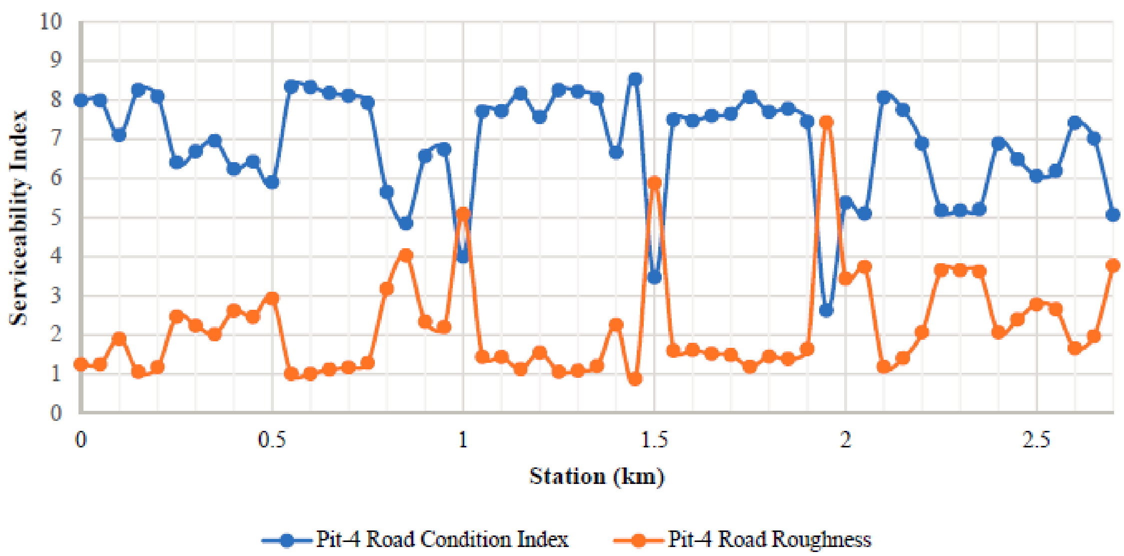

Figure 18 rRCI and rufindex of Pit-4 Road

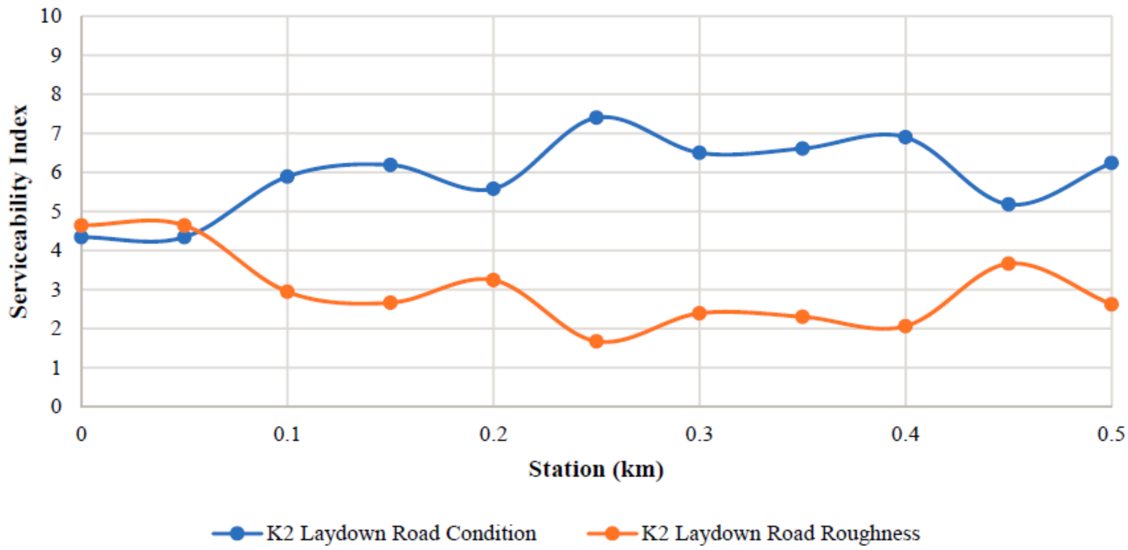

Figure 19 rRCI and rufindex of K2 Laydown road

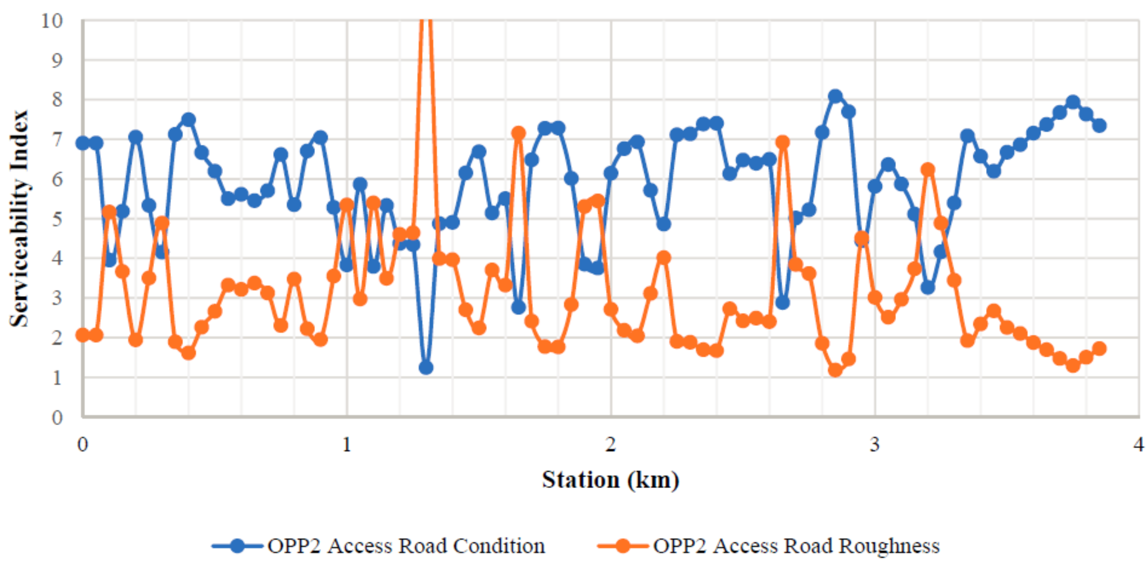

Figure 20 rRCI and rufindex of OPP2 Access Road

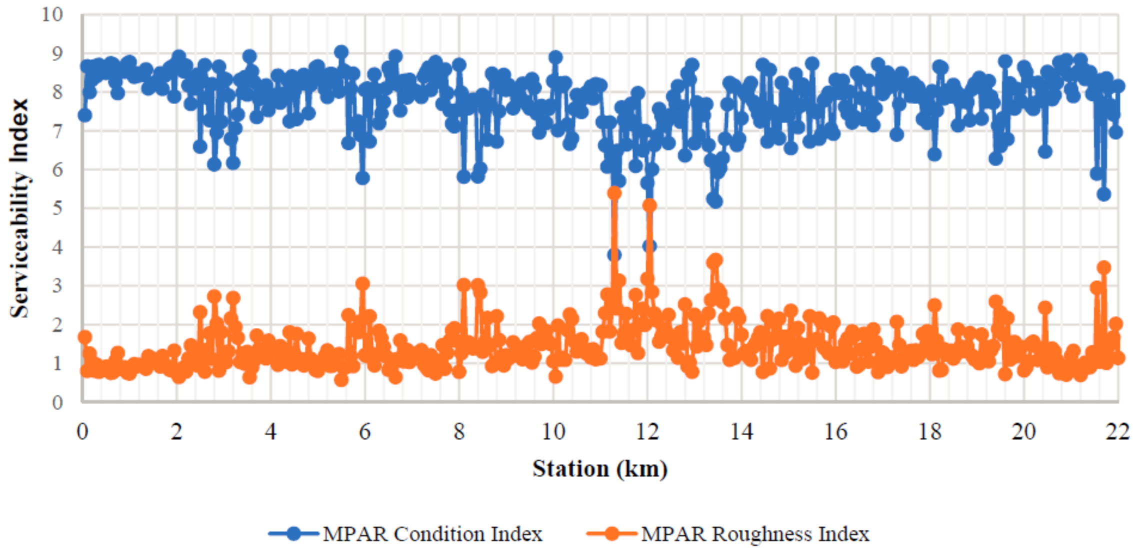

Figure 21 rRCI and rufindex of MPAR

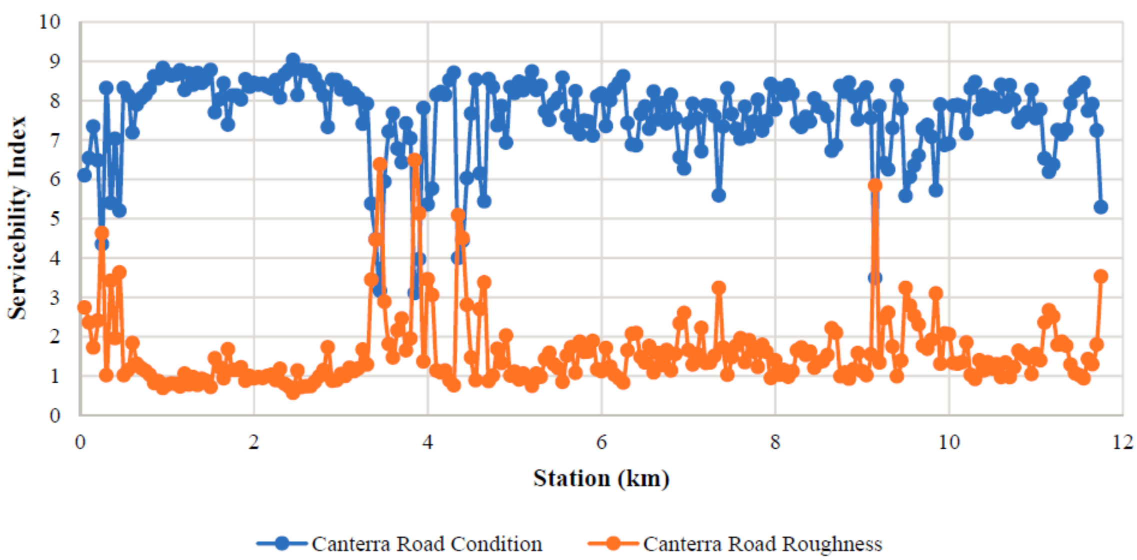

Figure 22 rRCI and rufindex of Canterra Road

Table 2 rRCI Values of Roads

Section	Homogenous Segment (km)	Roughness Index (rufindex)	Road Condition Index (rRCI)
Pit-4 Road	0.0 – 1.9	1.90	7.2
	1.9 – 2.7	3.00	6.0
K2 Laydown Road	0.0 – 0.2	3.37	5.5
	0.2 – 0.4	2.00	6.9
	0.4 – 0.5	3.00	5.7
OPP2 Access Road	0.0 – 2.0	3.59	5.5
	2.0 – 3.85	3.00	6.3
MPAR	0.0- 8.0	1.19	8.1
	8.0 – 22.5	1.56	7.6
Canterra Road	0.0 – 0.4	2.53	6.4
	0.4 – 3.4	1.00	8.2
	3.4 – 4.6	3.00	7.0
	4.6 – 9.1	1.46	7.7
	9.1 – 10.0	2.00	6.7
	10.0 – 12.0	2.00	7.7

3.2.3 Pavement Surface Distresses Evaluation

Pavement surface distresses evaluation is to inspect and rate all of imperfections and concerns on pavement surface. Surface distress surveys are usually conducted using manual, semi-automated and fully automated methodologies on pavement. Manual field data were collected by experienced, trained and specialized evaluators throughout the site. Most of provincial transportation agencies develop their own procedures. The procedure followed on site was surface condition rating manual from Alberta Transportation (AT, 2003). Although there are different rules of procedures from agencies, most methods require the same three aspects to be recorded, including distresses types, extent of distress, severe level of the distress along with possible causes and maintenance recommendations. Distress data were described detail in and recorded on evaluation forms and photographs were taken (TAC, 2013). There were mainly six types of distresses observed to be prevalent throughout the site, including dust, loose gravel, soft spots, washouts, washboarding and potholes, typical distresses in gravel roads. The detailed surface distress evaluation on each road section is discussed below.

3.2.3.1 Pit-4 Road

Pit-4 Road is a gravel road. Dust issues appeared to be a concern, even if it has lower traffic speed limit. It is especially seen at the bottom of the hill where a sharp turn can make poor visibility a safety concern. Another distress should be pointed out is washboarding. It was observed throughout the full length of climbing lane, and especially remarkable at the lower section of the lane. Localized washout appeared in the southwest corner of the section as shown in Figure 23. Poor drainage results in shear failure in the edge of the roadway. The berm running along the western edge of the roadway's ditch yield water from the roadway flowing to this one point at the bottom of the hill. However, the ditch should be properly stabilized to eliminate the erosion issue. The super-elevation of the curve also results in all water from this area of the road surface flowing into this corner, as well as significant runoff.

Figure 23 Local Washout on Pit Road 4 Section

3.2.3.2 Control Room Road

Water pooling issue on roadway may happen during rain seasons, since there was no discernible longitudinal grade and no crown in this road section. As shown in Figure 24, the slope wash-out into ditch which provides acceptable drainage function resulted from run-off can cause accumulation of sediment in the culverts beneath the driveway sections. Slope stabilization is essential to maintain slope stability under rainwater flow conditions. LOS has been applied in this section in 2006 in order to solve the dust issue. The dust issue had been controlled and reduced. However, the outside wheel paths in each direction seem to have less LOS treatment left, and some dust can be seen as vehicles passing the roadway. Along the edge of the roadway, loose gravel and clumps of LOS were also observed.

Figure 24 Ditches on Control Room Road

3.2.3.3 K2 Laydown Road

LOS had been applied on this road, but was no longer visually apparent. Loose gravels and clumps of LOS were found at the edges and on the shoulders of the road. This road is also experiencing a dust issue, and it was often treated by watering.

3.2.3.4 OPP2 Access Road

It was found the surface layer was very inconsistent: some areas contained more aggregates, while some areas contained only fine materials. It might be caused by two reasons. The first reason is inappropriate construction operations, and the other one is poor mixing of two materials. Loose aggregates and clumps of LOS were observed on sides of roadway which may be due to the surface layer placement method or a result of traffic loads. In addition, washboarding and potholes were found in some areas, and more were observed in the northbound lane as shown in Figure 25. It indicates more traffic in this direction and poor drainage condition on the eastern edge. It was noted that there was almost no concern of dust on this road which may be contributed by the application of the LOS.

Figure 25 Distresses in OPP2 Access Road Section (Washboarding and Potholes)

3.2.3.5 MPAR Road

The primary concern found on this road is dust. Large clouds of dust can be seen behind moving vehicles and the adjacent trees were covered by heavy coating of dust, which indicates the loss of fine materials. Strategically timed water application was applied on the roads throughout the day to mitigate the dust effect. However, it is noted that the water application was effective for a very short time only especially in sunny and windy weather conditions. In order to resolve the dust issue, a program in which three different chemical suppressants were trialed along the MPAR road was carried out in 2016. The products included Dust Treat, Entac and DustBind. MPAR road was divided into three sections (0-1km, 1-3km, 6-11.8km) and the suppressants were applied on to gauge the effectiveness. Some other distresses were observed including localized potholes and washboarding, but not significant.

3.2.3.6 Canterra Road

Figure 26 shows the curve at the bottom of a slight hill and side ditches on some areas of Canterra Road where water is collecting. This water ponding would seep into the road structure which resulted in various localized issues including washborading, potholes, and soft spots as shown in Figure 27. These issues, in combination with loose gravel found on the shoulder throughout Canterra's length, can make for unsafe driving conditions. These conditions are further exacerbated by the higher speed limit (70 km/h) in this area. Canterra also had similar dust issues and the dust was controlled by frequent watering application.

Figure 26 Water Pond on Canterra Road

Figure 27 Washboarding and Soft Spots in Canterra Road

3.3 Pavement Condition Rating

There are several indices that are acceptable and recognized for gravel roads surface condition assessment including Pavement Condition Rating (PCR) (Chong, 1989b), Unsurfaced Road Condition Index (URCI) (Eaton, 1987) and Gravel Condition Index (GCI) (MacLeod, 2008). The PCR method is the simplest and efficient to evaluate the roadway condition. It is subjective and generally used by civil engineers (TAC, 2013). A pavement condition number is determined and it is the representative of a pavement's overall condition. The number ranges from 0 to 100. 0 means extensive distress or very poor condition and 100 means no distress or very good condition. The PCR rating guidelines are shown in Table 3 (Chong, 1989a). According to the PCR rating guideline, Table 4 summarizes the major types of distresses identified on each road section. Based on the rater's field experience, the severity of distresses on each road has been determined and a level of PCR rating is assigned. The road with the highest condition level (80-100) is K2 Laydown road. This road is well shaped with no surface distress manifestations and good drainage condition. The other four roads are classified within the level of 60-79. This means these roads have slight to moderate distresses but without major effect on the ride comfort. The Canterra Road has the lowest condition level (40-59). Distresses of dust, washboarding, washout, loose gravel and pothole were found on this road. This road is a provincial road with a higher traffic volume and higher traffic speeds which may accelerate the deterioration.

Table 3 PCR Rating Guidelines

Level	Description
80-100	The roadway is well shaped with well-defined shoulders. There are no surface distress manifestations, and there is no more than a slight classification for dust and loose gravel. There are no frost heaves or soft spots and there is good drainage for surface runoff.
60-79	The roadway surface is well shaped. There are some distress manifestations in the slight to moderate category, such as loose gravel, dust and potholes. There may be a few soft spots in the spring. There is good drainage of surface runoff.
40-59	A mixture of properly shaped roadway surface and improperly shaped areas. Shoulder distress manifestations such as surface drainage are in the slight to moderate class, as are various surface distress manifestations, including washboards and potholes. There may be localized soft spots. Increased routine maintenance is required and spot gravel application may be necessary.
20-39	The majority of the roadway is improperly shaped. Surface drainage is impeded. There are some localized breakup areas and various distress manifestations, such as washboards, potholes or distortions, are in the severe classification. Maintenance with the addition of gravel is necessary. Some portions may need rehabilitation.
0-19	A flat or reverse crown characterizes the surface or there are severe roadway distresses such as washboards, loose gravel and potholes. Water is trapped along the edge of the travelled lane. There is little or no gravel. Rehabilitation is necessary.

Table 4 Distresses and PCR Rating on Road Sections

Section	PCR Rating	Distress Types				
		Dust	Washboarding	Washout	Loose Gravel	Potholes
Pit 4 Road	60-79	x	x	x	x	
Control Room Road	60-79	x		x	x	
K2 Laydown Road	80-100	x			x	
OPP2 Access Road	60-79		x		x	x
MPAR Road	60-79	x	x		x	x
Canterra Road	40-59	x	x	x	x	x

3.4 Recommendations

Based on the existing condition throughout the site, recommendations are made as below:

- **Mechanically stabilizing sections of loose gravel and soft spots**. For example, loose gravel issue on MPAR road can be solved by blending loose gravel with fine soils and compacting at optimum moisture content. Soft spots issue in Canterra Road can be remediated by scarifying and recompacting the upper layer with adequate compaction.
- **Maintain crowning** during typical maintenance operations to reduce surface ponding and allow surface water flow to ditch areas.
- **Remove roadside berms** while possible, the berms tend to cause water ponding in shoulder area and later reduce strength in wet weather. It can be improved with deeper ditches, or culverts which allow water to drain away from the area. Some cases that the ditches sit close to the groundwater level and can be drained effectively. It may be necessary to build up a thicker road structure to provide a large depth of strength and permeable material above subgrade that is prone to significantly weaken in wet conditions.
- **Stabilize ditch slopes** where possible. Slope stabilizing mats should be used throughout the site. Stabilization helps to avoid sediment blocking culverts and also maintain slopes adjacent to roadways, which avoid structural capacity loss issues. However, it should be noted that this treatment may be expensive.
- **Lean Oil Sands** (LOS) should be investigated for potential use as surface or base materials on gravel roads. LOS treated roads, including Control Room and K2 Laydown and OPP2 Access Roads, were found reduced dust significantly. The LOS has been successfully used as a dust suppressant previously. The strengths of LOS surface treated roads were higher than the road having LOS blended with granular materials and applied as structural gravel. That is the possibility of reduced granular material. However, loose gravel was observed at the edges and shoulders of roadway, and some clumps of LOS were presented. It is recommended to study on optimum mixing ratios of LOS with granular materials, other than 50/50, and improved the stability and consistency of LOS-granular mixture material.

In the long term, it is recommended to make a network level pavement management system so that maintenance operations will be prescheduled and implemented in a timely manner. Roadways will be kept in good condition and likely extend their service life.

Chapter 4
Laboratory Test for LOS-Granular Mixture

It has been demonstrated through the preliminary site survey at Kearl Lake Oil Sand site that application of LOS as a binding agent and mixed with granular material in a volume ratio of 50/50 to form an asphalt-like material provides a number of benefits including saving material cost and mitigation of dust issues.

In order to quantify mechanical properties of LOS-granular mix material with different mixing ratio, a suite of laboratory tests have been conducted by Centre for Pavement and Transportation Technology (CPATT) at University of Waterloo. The characteristic material property of gravel base used for design is the resilient modulus. For many years, standard California Bearing Ratio (CBR) tests were utilized to measure the base and subgrade strength parameter as a design input. Since the resilient modulus test equipment is currently not present in many laboratories, researchers have developed correlations to converting CBR values to approximate resilient modulus values. In this study, CBR tests are conducted to quantify the material property of the LOS-granular mix materials.

The test materials including granular material and LOS were provided by Imperial Oil Inc. The granular material used in these tests is either from a limestone quarry or alluvial gravel deposits (i.e. Susan Lake, GMS, etc.). The LOS of bituminous content less than 9% by weight is produced at Kearl Lake Site, Alberta, Canada. Both LOS and granular material were shipped in sealed buckets from Imperial Oil Inc. to CPATT laboratory in University of Waterloo.

4.1 Moisture Content Test

Moisture content tests have been conducted for the granular and LOS materials following the standard test methods in AASHTO T225 “Total Evaporable Moisture Content of Aggregate by Drying” (2011). A moisture content test aims to determine the evaporated moisture of a test sample by comparing the wet mass of the sample and oven dried mass of the sample. The total evaporable moisture content can be calculated as follows

$$Z = 100(\mathrm{W_{original}} - \mathrm{W_{dried}}) / \mathrm{W_{dried}} \qquad \textbf{(2)}$$

Where,

Z (%) is the total evaporable moisture content of sample.

$W_{original}$ (g) is the mass of original sample.

W_{dried} (g) is the mass of dried sample.

Moisture content of a sample includes both surface moisture and moisture in the aggregate pores. The oven temperature is set to 110 ± 5°C which is intended not only to dry the sample thoroughly but also to avoid loss of particles. The drying time is typically 24 hours to reach a constant mass. Figure 28 shows that samples are placed for oven drying in the CPATT lab. Sample sizes for test materials are dependent on the maximum nominal size of aggregate. The minimum sample mass required based on the nominal maximum size of aggregate has been followed as required in the standard. Based on the results of sieve analysis in Chapter 5, the nominal maximum size of LOS and granular material are 12.5 mm and 19 mm, respectively. Therefore, the minimum sample sizes for moisture content tests are 2 kg and 3 kg for the LOS and granular material, respectively. A sample of laboratory moisture content data collection and calculations are presented in Appendix A. The results of moisture content will be used as sample preparation inputs for the subsequent extraction, proctor and CBR tests.

Figure 28 Oven Dry Samples for Moisture Content Test

4.2 Lean Oil Sand Extraction Test

The purpose of the extraction tests is to quantify the asphalt binder content in the LOS provided by Imperial Oil Inc. The tests have been performed following the procedure in ASTM D2172/2172M "Standard Test Methods for Quantitative Extraction of Asphalt Binder from Asphalt Mixtures" (2017).

The Centrifuge Extraction, which is referred to as test method A in the procedure, was selected to perform the tests. Specific apparatus are required for this method including centrifuge extraction unit bowl (as shown on the left part of Figure 29) and a filterless centrifuge extractor (as shown on the right part of Figure 29).

Figure 29 Centrifuge Extraction Apparatus

The minimum testing sample size shall be governed by the nominal maximum aggregate size of the mixture, which is provided in Table 5. A minimum mass of 1.5 kg sample was required for the LOS material in this test. Riffle splitter was used to prepare and separate samples following AASHTO R47-08 "Reducing Samples of Hot Mix Asphalt (HMA) to Testing Size" (2010). Samples were uniformly distributed from edge to edge so that approximately equal amounts will flow through each chute, the rate of flow should be controlled through the chutes so that the materials pass freely into the receptacles below (Figure 30). The LOS was reintroduced to the receptacles as many times as necessary to obtain the desired test sample size. Different portions can be combined as long as each portion remains a valid split.

The moisture content of LOS samples was obtained before this test so that the measured mass loss could be corrected for moisture. The method involves adding solvents to LOS samples to obtain the asphalt binder through extraction using centrifuge extraction unit bowl equipment. The speed of centrifuge is slowly and gradually increased to a maximum of 3600r/min. The procedure is repeated at least 3 times, adding sufficient additional solvents till the extract is not darker than a light straw color. Figure 31 shows the extraction tests performed for the LOS samples.

Table 5 Size of Sample – Extraction Test

Nominal Maximum Aggregate Size (mm)	Minimum Mass of Sample (kg)
4.75	0.5
9.5	1
12.5	1.5
19	2
25	3
37.5	4

Figure 30 Sample Separation using Riffle Splitter

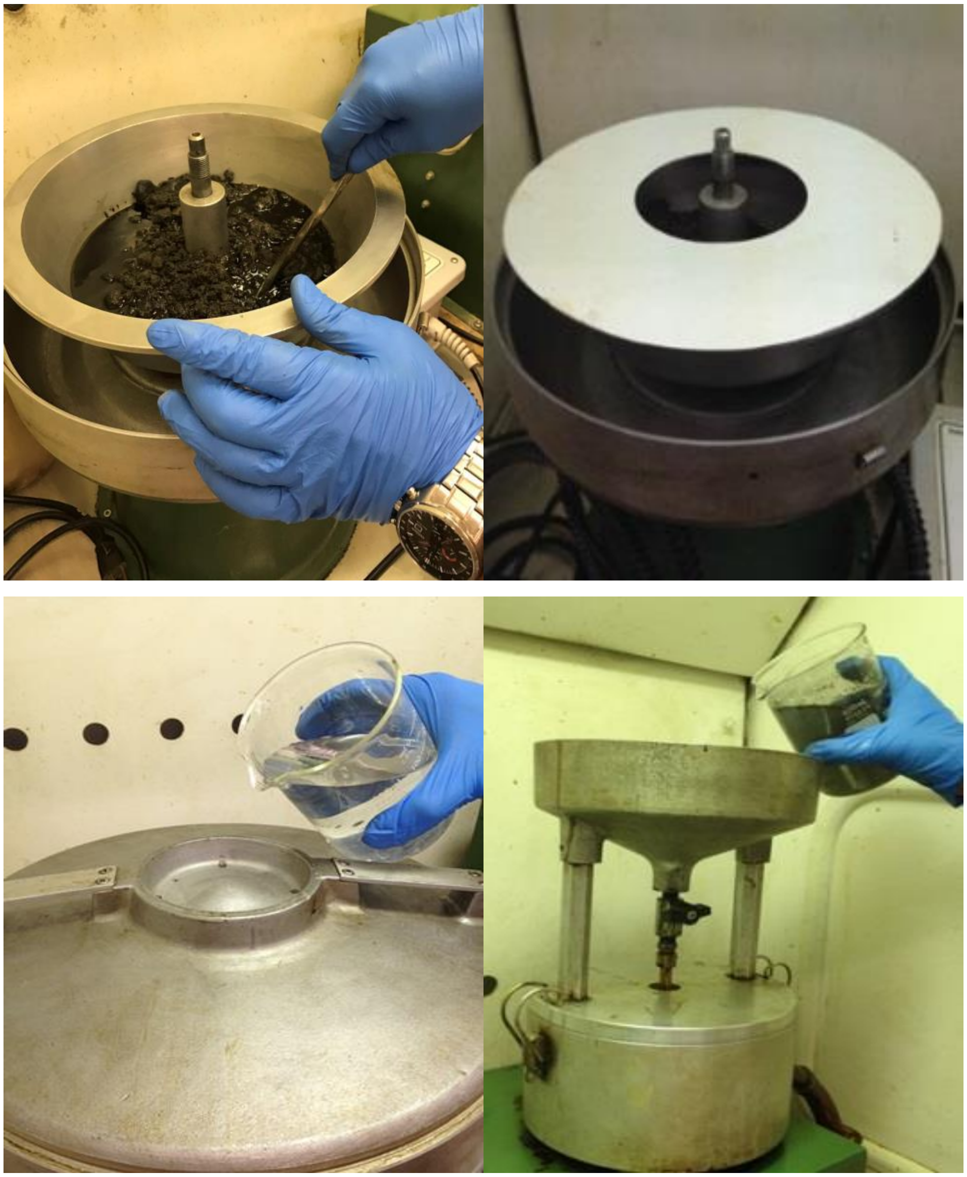

Figure 31 Extraction Tests for LOS Samples

Mineral fines can be collected both in filter paper from centrifuge extraction unit bowl and in cast aluminum cup from filterless centrifuge extractor. The purpose of using filterless centrifuge extractor is to collect further mineral fines at a higher speed of 11,000 r/min. Figure 32 shows two steps to collect mineral fines from the extractor.

Figure 32 Mineral Fines Collected from Extractor

The data and calculations can be found in Appendix B. The mass of dry samples from centrifuge extraction unit bowl is calculated using Equation 3:

$$W_2 = \frac{100W_1}{100 + Z} \quad \textbf{(3)}$$

Where,

Z (%) is the moisture content;

W_1 (g) is the mass of LOS sample;

and W_2 (g) is the mass of LOS dry sample.

The final total dry mass of sample and amount of asphalt binder extracted are used to calculate the asphalt binder content in the test portion, which can be determined using the following equation. The asphalt binder content is expressed as a mass percent of moisture-free mixtures:

$$W_6 = \frac{100[W_2 - (W_3 + W_4 + W_5)]}{W_2} \quad \textbf{(4)}$$

Where,

W_2 (g) is the mass of LOS dry sample;

W_3 (g) is the mass of fines in filter paper;

W_4 (g) is the mass of extracted aggregate after extraction;

W_5 (g) is the mass of fines in the cup;

and W_3+ W_4+ W_5 (g) is the mass of dried aggregate.

After the extraction test, the extracted asphalt binder was recovered in accordance with ASTM D5404-17 “Recovery of Asphalt from Solution Using the Rotary Evaporator” (2011), solvent of trichloroethylene was then recycled. Figure 33 shows the recovery test being performed. The asphalt solution from extraction test was transferred into distillation flask for preparation of recovery testing as shown in Figure 34.

Figure 33 Recovery of Asphalt from Solution Using Rotary Evaporator

Figure 34 Preparation for Recovery Test

4.3 Preparation of Mixtures

The mixtures of LOS and granular blended materials were prepared for the following tests, including gradation, proctor and CBR tests. Based on the survey of road condition assessment in Chapter 3, a 50/50 blended mixture which was applied at the site road for trial is considered as reference mixing ratio. Two other mixing ratios 30% granular material with 70% LOS, and 70% granular material with 30% LOS were selected for the tests. The objective is to investigate effect of application of LOS on the properties of different mixtures. It should be noted that the mixing ratios are expressed in terms of volumes rather than masses which in accordance with packing and mixing operation performed on Kearl Lake construction site.

4.4 Gradation Test

The purpose of performing gradation test is to determine if the grading of materials is qualified for construction of gravel roads. The physical properties of aggregates are essential to the performance of a road structure.

According to AASHTO T27-11, "Sieve Analysis of Fine and Coarse Aggregates" (2011), the minimum mass of sample is decided based on four times of the mass required as shown in the standard or AASHTO T2-91, "Sampling of Aggregates" (2010), whichever gives a greater mass. For LOS with a nominal maximum size of 12.5 mm, the size of test sample is 8 kg. For granular material with a nominal maximum size of 19 mm, the mass of sample size is 20 kg.

Different opening sizes of sieves are selected for the gradation test including 25 mm, 19 mm, 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 0.6 mm, 0.3 mm, 0.15 mm and 0.075 mm. The sieves are placed in order of a decreasing size from the top to bottom. Samples need to be dried to constant mass at a temperature of 110 ± 5°C before the gradation test. The sample were split into more portions and place a portion on the top sieve and allow mechanical sieve shaker to bounce and tumbler the particles to different orientations for a sufficient period. It needs several times of sieving operation to obtain the total size of each sample. A brush was used to gently transfer all aggregates on sieves to corresponding containers especially for finer aggregates. Finally, the masses of all material retained on a specific sieve were combined and weighed. Figure 35 shows gradation test performed in the laboratory.

Figure 35 Gradation Test

The percent of passing and percent of retaining for a sieving analysis is determined using Equations (5) and (6):

$$P_{retained} = \frac{100W_{sieve}}{W_{total}} \tag{5}$$

$$P_{passing} = \frac{100W_{below}}{W_{total}} \tag{6}$$

Where,

P_{retained} (%) is the individual or cumulative percent retained;

W_{sieve} (g) is the individual or cumulative mass of retained aggregate;

W_{total} (g) is the mass of total dry sample;

and W_{below} (g) is the mass of aggregate below the current sieve, not including the current sieve's aggregate.

4.5 Proctor Test

A proctor test aims for determining the moisture-density relations of soil-aggregate mixtures. During the construction, loose soils must be compacted to improve strength by increasing unit weight. In order to get maximum density and stability, water is added to achieve the optimum moisture content. The effect of compaction is illustrated in Figure 36.

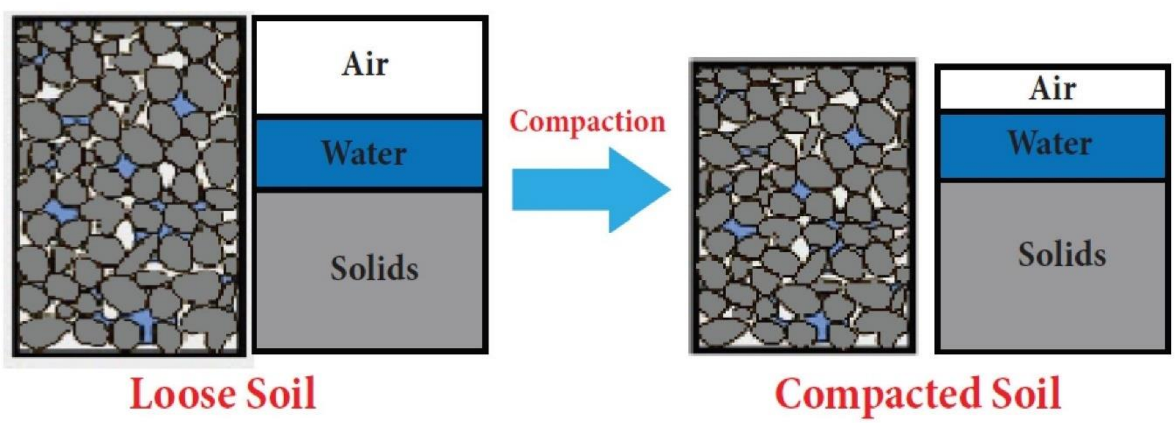

Figure 36 Illustration of Compaction Effect

The measurement was conducted following AASHTO T99-10 "Moisture-Density Relations of Soils Using a 2.5-kg Rammer and a 305-mm Drop" (2011). The standard method C was used since 30 percent or less aggregate was retained on the 19 mm sieve for all mixtures. The testing aggregate materials have been dried at a temperature less than 60°C and then broken up, avoiding reduction of the natural sizes of particles. The mass of material retained on the 19 mm sieve is less than 5% so that correction for oversize particles shall not be applied. The materials retained on 19 mm sieve were discarded. The final dry mass of a representative sample is approximately 5 kg. Each of the three layers has been compacted by 25 uniformly distributed blows using a hammer which is manually operated at a drop distance of 305 mm. After compaction, the compacted specimen has been trimmed with top of the mold using a metal straightedge. Any holes in the surface by removal of gravel sized particles were patched with smaller sized particles. Finally, the specimen was removed from the mold and sliced vertically through the center to obtain a representative portion of three layers. The corresponding masses have been weighted at each step for calculation. The procedure has been repeated by adding 1-2% increment of water each until there is a decrease in the wet mass. The test performed in the laboratory is shown in Figure 37.

Figure 37 Proctor Test for LOS-Granular Mixtures

4.6 California Bearing Ratio Test

The test is to determine the CBR value of the three different LOS-granular mixture materials from laboratory compacted samples. The results are intended to evaluate the potential strength of materials for use in gravel road design. The CBR test is only conducted for samples with optimum moisture content which is obtained from previous Proctor tests. For preparation of samples, the material retained on 19 mm sieve should be removed and replaced by an equal mass of material which is passing 19 mm and retained on 4.75 mm sieve. Approximate 6 kg of the sample material was used for one test. The sample is compacted in the mold with a diameter of 152.4 mm and a height of 177.8 mm as shown in Figure 38.

Figure 38 CBR Test Compacted Mold

Following ASTM D1883-16 “Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils” (2016), 75 blows per layer have been applied as the maximum dry density that was determined from Proctor test in the 101.6 mm diameter of mold. As shown in Figure 39, the swell plate with adjustable stem was put in the mold and surcharge of a minimum 4.54 kg on the plate was placed above the sample. An intensity of loading which equals to the total mass of the materials was applied. The mold had been immersed in water to ensure water freely flow to the top and bottom of the samples. A tripod with dial indicator was used for recording initial reading and final reading of soaked mold after 4 days.

Figure 39 CBR Soaking Test

The penetration test was conducted after 4 days of soaking. The penetration depths were selected as 0.64 mm, 1.27 mm, 1.91 mm, 2.54 mm, 3.81 mm, 5.08 mm, 7.62 mm, 10.16 mm and 12.7 mm. The rate of penetration is 1.3 mm/min (AASHTO T193-10, 2011). Load is recorded with the corresponding penetrations. Figure 40 shows the penetration test using master loading machine.

Figure 40 CBR Penetration Test

Chapter 5
Statistical Analysis of Test Results

A suite of laboratory tests, including moisture content test, LOS extraction test, preparation of LOS-granular mixture samples, gradation test, proctor test and CBR test, have been conducted. In this chapter, the test results are presented and statistical analyses of the results are performed to quantify the properties of the LOS-granular material with the three different mixing ratios.

5.1 Extraction Test Results for LOS

In order to minimize the effect of variations or inconsistency in content of the supplied LOS materials, all the LOS materials provided by Imperial Oil Inc. were mixed thoroughly at first. Then riffle splitter was used to separate the LOS material and store them in six buckets sealed. The extraction test was conducted on six LOS samples, one obtained from each bucket. The bitumen content was measured to be 4.24%, 3.70%, 2.97%, 3.06%, 4.43% and 4.02% for the six LOS samples. The binder content varies from approximately 3-4.5% and with an average of 3.74%. Government of Alberta has presented that bitumen content less than 7% is characterized as low or poor grade oil sand (Government of Alberta, 2016). Detailed measured data and calculations are presented in Appendix B.

5.2 Gradation Test Analysis

The purpose of a gradation analysis is to determine if the testing material can be applied as traffic and/or structural gravel, and if the quality of the material is adequate to meet requirements for design and construction. Gradation tests had been performed for the LOS and granular materials provided by Imperial Oil Inc. and the mixture samples of LOS and granular material in this study.

Quality aggregates depends on availability of local quarry sources. Canadian transportation agencies follow a variety of standards but have some uniformity in the requirements. Usually, at the regional level, it follows national or provincial widely used specification and then modifies it based on local requirements. Typical requirements of gradation for granular base and sub-base materials are listed in Table 6. The gradation of top surface layer on gravel roads should be well graded material. Many agencies do not have specifications for surface gravel requirements but conform to local experiences. The major difference between structural and traffic gravel is the content of fine aggregates. A high amount of fines, i.e., 15-20%, provides a smooth and tight gravel surface. It performs well in dry weather with reduced aggregate loss. However, higher fine content decreases the structural strength, and it tends to rut in wet weather and increases frost susceptibility of gravel material. The amount of fines should be based on purpose of the road, the desired level of service and local conditions. Generally, if the road will be paved in the future, the amount of fines should not exceed 10% (TAC, 2013).

Table 6 Gradation Requirements for Base and Sub-base Materials (TAC, 2013)

Sieve Size (mm)	Percent Passing			
	Sub-base	Well Graded Base		Open Graded Base
		25 mm	19 mm	
75	100			
37.5				100
25	55-100	100		
19			100	50-100
16		70-90		
12.5				25-70
9.5		50-75	50-80	15-50
4.75	25-100	35-60	40-70	5-15
2.36	15-80			
1.18	15-45	15-40	15-40	0-8
0.6	10-35			
0.315		8-20	8-25	
0.15	5-15	5-15	5-18	
0.075	0-8	2-8	2-8	0-5

5.2.1 Gradation of Lean Oil Sand

Gradation tests have been performed for three LOS samples. The averaged result for the three LOS samples is shown in Table 7. According to standards provided in Table 6, it only meets the requirement for being sub-base material. For aggregates retained on 4.75 mm sieve, it is classified as gravel; aggregate between 0.075 mm and 4.75 mm is classified as sand; for the aggregate size smaller than 0.075 mm, a plastic limit could not be determined by an Atterberg Limit test. It is reported as a non-plastic material. Therefore, this portion of the material is defined as silt.

Table 7 Gradation Test Results for Lean Oil Sand

Sieve opening size (mm)	Average percentage of weight passing (%)
19	100.0
12.5	98.9
9.5	91.1
4.75	70.8
2.36	65.0
1.18	44.8
0.6	33.8
0.3	31.0
0.15	10.3
0.075	3.2

Based on the averaged gradation test results for the LOS, the material is composed by 29.2% of gravel, 67.7% of sand and 3.2% of silt. Figure 41 plots the gradation graph for each test sample.

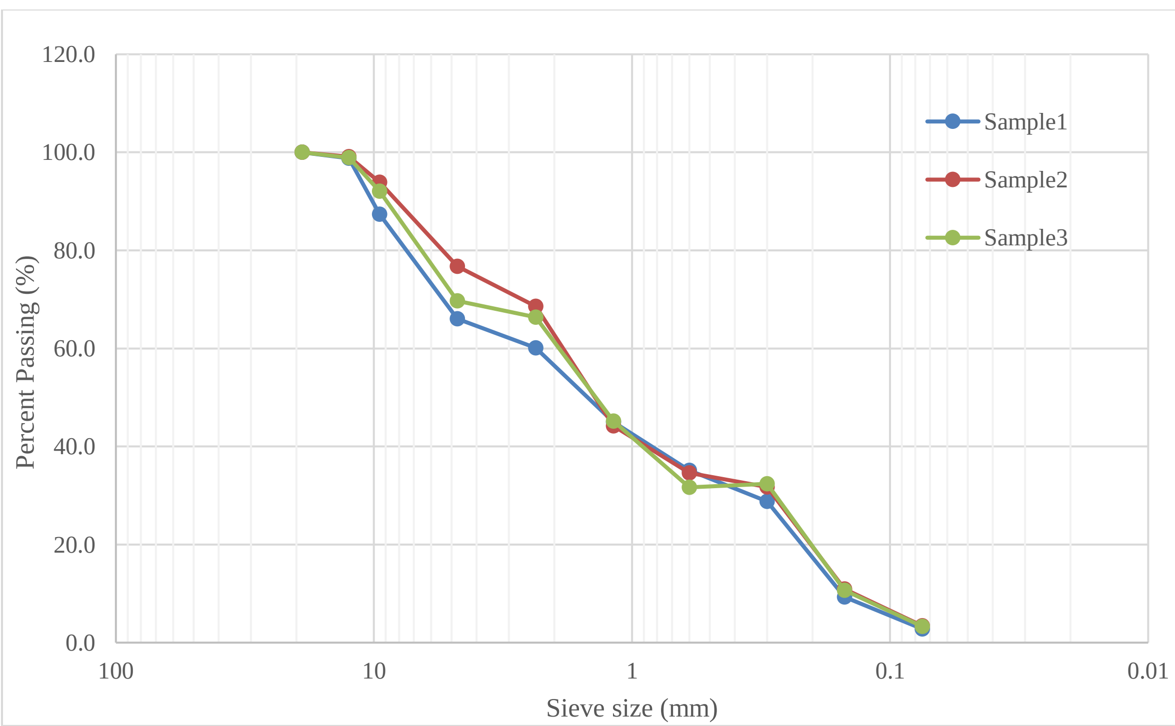

Figure 41 Gradation Graph for Lean Oil Sand

5.2.2 Gradation of Granular Material

The gradation test results for the granular material are presented in Table 8 and Figure 42. It can be seen that the granular material provided by the Imperial Oil Inc. is suitable for both sub-base and 25 mm/19 mm well graded base. Based on the averaged gradation results from three test samples, the granular material consists of 53.1% of gravel, 42.7% of sand and 4.3% of silt. Compared to the LOS, the granular material has 80% more coarse aggregate which provides sufficient strength to satisfy the requirement for granular base material.

Table 8 Gradation Results for Granular Material

Sieve opening size (mm)	Average percentage of weight passing (%)
25	100.0
19	92.9
12.5	72.9
9.5	61.2
4.75	46.9
2.36	39.0
1.18	34.0
0.6	24.4
0.3	13.7
0.15	7.5
0.075	4.3

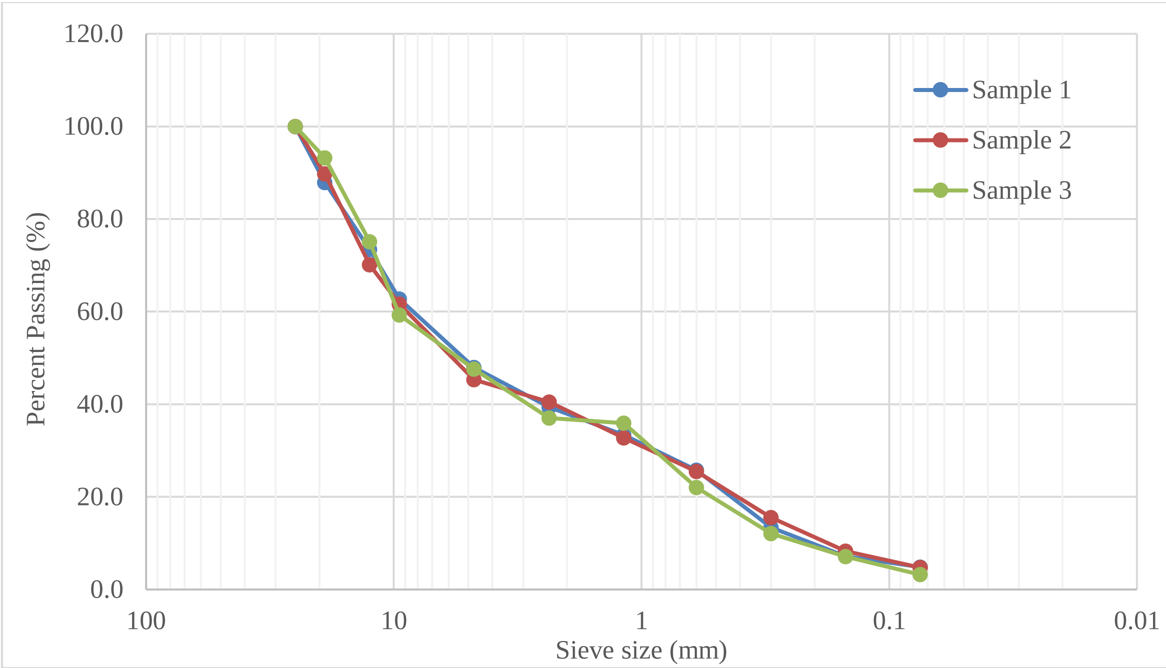

Figure 42 Gradation Graph for Granular Material

5.2.3 Gradation of LOS-Granular Mixture

Three types of LOS-granular mixture were developed for the gradation tests including 30% of granular material and 70% of LOS (Sample 30G/70LOS), 50% of granular material and 50% of LOS (Sample 50G/50LOS), and 70% of granular material and 30% of LOS (Sample 70G/30LOS). Three samples are selected from each type of blend for the gradation test. Table 9 shows the averaged gradation test results for each mixture type. The result gives a general idea on whether the mixture type is applicable to be used as sub-base or base materials through the sieving analysis. It was found that the mixture of 30% granular material and 70% LOS can only be used as sub-base material, and the mixtures of 50% of granular material and 50% of LOS, and 70% granular material with 30% LOS are suitable for sub-base and base material. It indicates that a certain percentage of large sized material is needed to maintain the load-carry capacity of base course. For traffic gravel, it requires more materials passing 0.075 mm sieve than base gravel. However, all of the three mixtures contained less than 4% materials passing 0.075 mm sieve. In addition, traffic gravel needs some plastic material such as natural clay, which can provide binding characteristic. Neither the LOS nor the granular material in this test contains clay content. Therefore, the current two materials do not meet the requirements for surface gravel. The complete gradation laboratory data and plots are presented in Appendix C.

Table 9 Gradation Results for LOS-Granular Mixtures

Sieve opening size (mm)	Average percentage of weight passing (%)		
	30G/70LOS	50G/50LOS	70G/30LOS
25	100.00	100.00	100.00
19	97.88	96.47	95.05
12.5	91.12	85.91	80.70
9.5	82.15	76.16	70.18
4.75	63.67	58.89	54.11
2.36	57.21	52.00	46.79
1.18	41.56	39.39	37.23
0.6	30.99	29.11	27.24
0.3	25.78	22.32	18.85
0.15	9.48	8.92	8.36
0.075	3.50	3.71	3.93

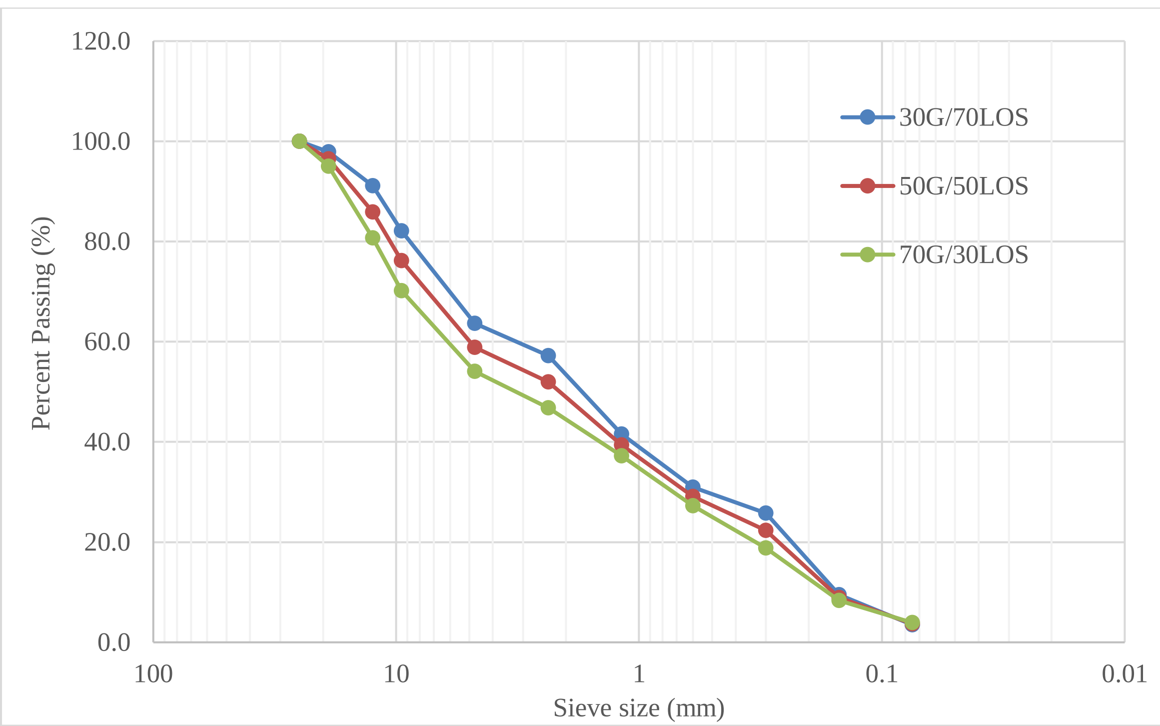

Figure 43 Gradation Graph for LOS-Granular Mixtures

5.3 Proctor Test Analysis

The compaction curve which represents the relationship of dry densities versus moisture contents was developed from the proctor tests. The dry mixed sample was divided into several portions and was compacted with same magnitude of load but with increment of water contents. It's difficult to compact dry soils with no or litter water because of high friction resistance. Water is added to lubricate the soil particles that get closer till they reach the tightest arrangement. Water with air should be sufficient to fill the voids and the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) is reached at this stage. By adding more water after reaching the OMC, the dry density decreases because excessive water begins occupying space of soil particles (Proctor, 1993). The dry densities with corresponding moisture content are calculated and are tabulated in Table 10. Regression analyses were performed to develop the trend curves which give the best fit for compaction curves for the three different mixtures as shown in Figure 44. The dry density of test samples increases with increasing moisture content up to the OMC and then decreases with increasing moisture content. The exact MDD value is the peak value that can be obtained from the equations, displayed in the curves. The OMC values and the corresponding MDD values are summarized in Table 11 for the three blends. The MDD of the mixtures ranges from 2.08 to 2.17g/cm^3. The results indicate that a higher percentage of granular material or lower percentage of LOS material leads to lower OMC but higher values of MDD. The mixtures of 70G/30LOS has the highest MDD and lowest OMC, anticipated to result in higher shear strength, lower permeability and compressibility, better soil stability and reduced frost damage. The calculations of wet density and zero air void line are included in Appendix D.

Table 10 Averaged Proctor Test Results

30% Granular and 70% LOS		50% Granular and 50% LOS		70% Granular and 30% LOS	
Moisture Content (%)	Dry Density (g/cm^3)	Moisture Content (%)	Dry Density (g/cm^3)	Moisture Content (%)	Dry Density (g/cm^3)
3.6	1.9722	2.79	2.0525	2.1	2.0437
4.08	2.0246	4.09	2.1116	4.37	2.1509
5.26	2.0592	5.56	2.1483	6	2.1637
7.27	2.0751	6.11	2.1409	6.96	2.1403
8.62	2.0156	6.86	2.11	7.95	2.0575
9.24	2.0131	7.61	2.0667		

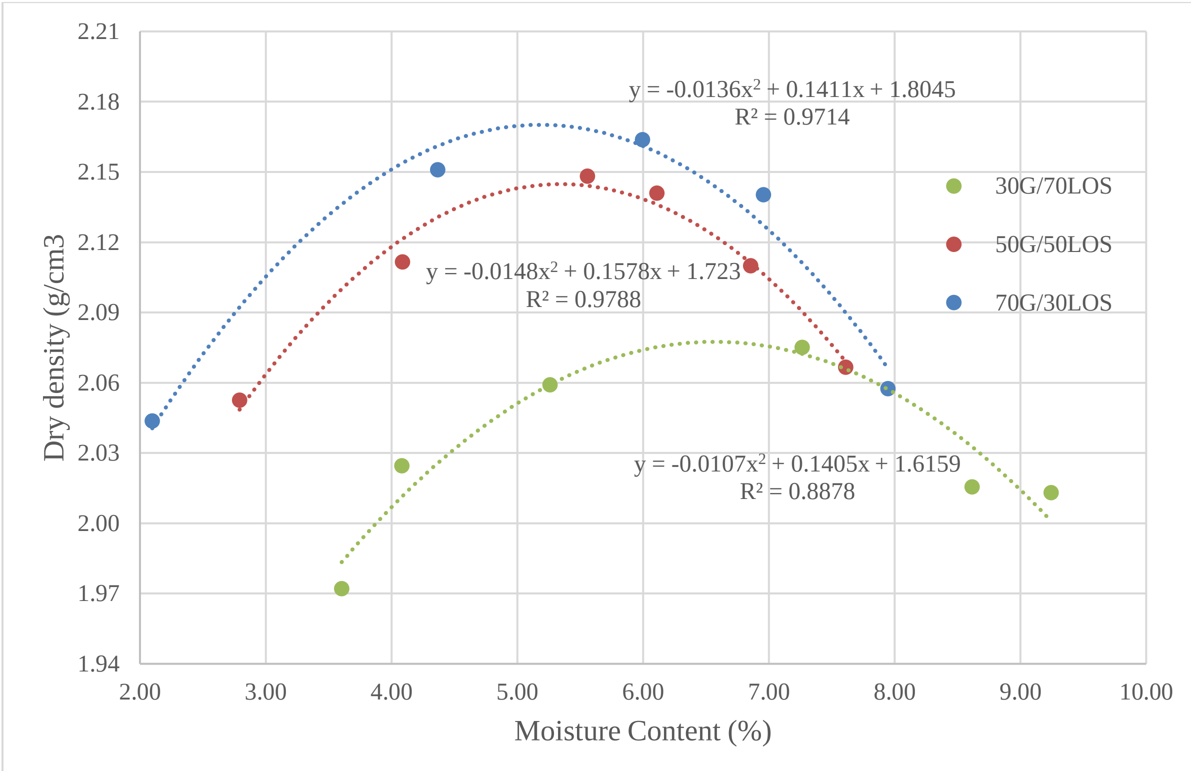

Figure 44 Regression Analysis of Proctor Results

Table 11 Summary of Optimum Moisture Content and Maximum Dry Density

Mixtures	Optimum Moisture Content (%)	Maximum Dry Density (g/cm^3)
30% Granular and 70% LOS	6.67	2.08
50% Granular and 50% LOS	5.33	2.14
70% Granular and 30% LOS	5.19	2.17

5.5 CBR Test Analysis

The CBR values of the compacted specimens with the optimum moisture content obtained in proctor compaction tests are determined. A sample calculation of the amount of water added to the mixtures is provided in Appendix E. Properties of some material are likely subjected to the changes of moisture content, for example, rainfall, ground water or any surfacing water. The strength will decrease as the moisture content increases. Therefore, it is important to record swelling percent during soaking. If the swelling percent exceeds 3%, it is identified as a swelling soil; if it is less than 3%, it is classified as non-swelling soil (Military Soils Engineering, 1992). The laboratory results of the three mixtures showed around zero percent swelling indicating that none of mixtures has the potential of swelling. The relationship between the penetration and stress in piston is plotted in Figure 45 for the three mixtures. It can be seen that the rate of change decreases as the penetration increases and approaches to zeros when the penetration reaches a certain level (less than 13 mm). The tendency of the three curves is similar, but the stresses for three mixtures are varied for a given penetration. The mixture of 70G/30LOS curve corresponds to the highest stress values and mixture of 30G/70LOS curve corresponds to the lowest value.

CBR values are determined from dividing the stresses values at penetrations of 2.54 mm and 5.08 mm by the standard stresses of 6.9 MPa and 10.3 MPa for crushed stone. Normally, it takes the value at 2.54 mm penetration. If the ratio at 5.08 mm is greater, a new set of tests should be conducted. Equation (7) shows the calculation of CBR value:

$$CBR = \frac{100A}{B} \tag{7}$$

where *A* (MPa) is the stress on the piston for 2.54 mm or 5.08 mm; *B* (MPa) is the standard stress values for well graded crushed stone. For 2.54 mm penetration *B* = 6.9 MPa; For 5.08 mm penetration *B* = 10.3 MPa.

The results of average CBR values of three mixtures are shown in Table 12. The CBR value increases with the increase of granular material content, which means a higher amount of coarse aggregate in the mixture. When granular material content increases from 30% to 50%, the CBR value is increased by 130%. When granular material content increases from 50% to 70%, the CBR value is increased by 49%. Combined with the results from the proctor tests, it is found that CBR value has a significant correlation with the MDD and OMC. The CBR value increases with the increasing of MDD and decreasing of OMC. Generally, higher stiffness of a mixture leads to a higher CBR value. A high-quality crushed aggregate has a CBR value above 80% which is applicable for base course material. A CBR value above 30% is suitable for sub-base course material. However, there is no universal standard for CBR design values of sub-base and base course. Selection of construction material is always combined with engineers' field experience and regulations of local governments. The CBR values were used to design the thickness of sub-base and base layers, and a high CBR value may allow for the reduction of the design thickness of constructed materials.

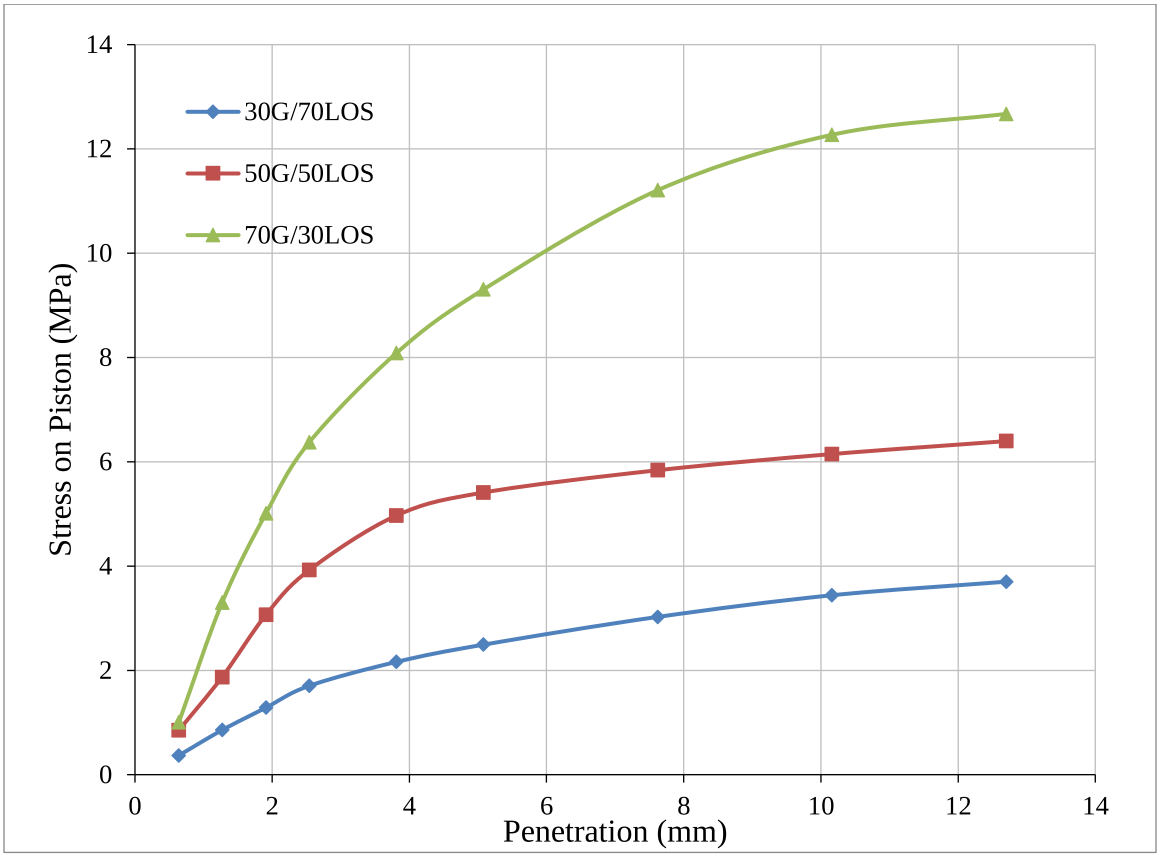

Figure 45 Averaged Load Penetration Curves for Three Mixtures

Table 12 Summary of CBR values

Mixtures	Averaged CBR (%)
30% Granular and 70% LOS	24.69
50% Granular and 50% LOS	56.85
70% Granular and 30% LOS	84.60

5.6 Summary

A suite of laboratory tests have been conducted for determining bitumen content of the LOS, gradation properties of the LOS and granular material, and OMC, MDD and CBR values of the LOS-granular mixture samples. Results from the extraction test show that the bitumen content in the LOS provided by Imperial Oil Inc. ranges from 3-4.5%. Therefore, the LOS can be classified as low graded oil sands. From gradation analysis, it shows that 30G/70LOS mixture is suitable to be used as sub-base course material, and 50G/50LOS and 70G/30LOS mixtures can be good materials for both sub-base and well-graded base courses. MDD and OMC relationship for the three mixtures are obtained by proctor tests. A higher percentage of granular material leads to a relatively lower OMC but a higher MDD value. The mixture of 70G/30LOS has the highest MDD and lowest OMC values. The CBR tests indicate that increasing the amount of LOS will decrease the CBR value which is related to the material strength. The survey of the gravel road named OPP2 Access Road applied with 50G/50LOS at Kearl Lake site showed that more distresses have been found on the lane with heavier traffic. This may be caused by insufficient granular content in the mixture and the mixture 70G/30LOS should be a better choice for the gravel surface.

Chapter 6
Gravel Road Thickness Design

6.1 Introduction

Two design methods are provided by AASHTO (1993) for gravel road design. The first method is a design chart-based method. This method employs nomographs to determine the gravel layer thickness accounting for a number of parameters including allowable serviceability loss and rutting depth, roadbed resilient modulus, elastic modulus of the aggregate layers and design traffic volume. The second method is based on a design catalog. This method is very general and is used only when the more detailed design approach is not possible.

In this chapter, analytic solutions will be developed based on the design chart-based method utilizing the digitization technique and regression analyses. The equation-based method allows designers/engineers to implement the method by programing it into computational tools such as Excel and Matlab. The implementation of the method will be verified by comparing with the results from design chart solutions. A parametric study is conducted to identify key factors that affect the gravel road thickness design and recommendations on application of LOS to gravel road design are provided.

6.2 Implementation of AASHTO Design Chart for Gravel Road Design

AASHTO design chart procedure provides a guideline for thickness design of aggregated-surfaced road. This design chart procedure consists of 10 major steps and requires a graphic solution. An overall picture of the design chart procedure and the relationship between the steps is illustrated in Figure 46.

This method is based on a trial-and-error approach. A trial thickness of base layer is selected; then the expected damage due to acceptable serviceability loss and rutting depth is calculated using design charts based on the design inputs including design loads, environment conditions, resilient modulus of roadbed and elastic modulus of base materials. The thickness that yields a damage of 100% is the one selected as the design thickness.

There are three design charts which are used to determine allowable number of 18-kip Equivalent Single Axle Loads (ESALs) application considering serviceability loss, to determine allowable number of 18-kip Equivalent Single Axle Loads (ESALs) application considering rutting, and to convert a portion of base thickness to equivalent sub-base thickness. To be consistent with the AASHTO standard, the design information is based in Imperial units. Unlike the design chart for flexible pavement recommended by AASHTO where an analytical solution is also provided along with the nomograph, the equations which produce the nomographs in the design chart of aggregated-surfaced road are not available. Therefore, this graphic method which requires a number of trials may not be efficient for practical use in some circumstances, for example, a parametric study on the effect of different design input parameters on the design thicknesses of gravel roads. In order to implement this design method in an efficient manner, digitization of the design charts is carried out, and the relationship between the parameters is quantified in analytical or numerical ways which makes this design chart procedure programmable. This method is implemented using Matlab and a standalone application was developed. The design thickness of an aggregate surfaced road can be obtained readily with inputting basic design parameters. The results of the implementation will be verified by comparing with graphic solutions given in the examples of the AASHTO design guide and other references.

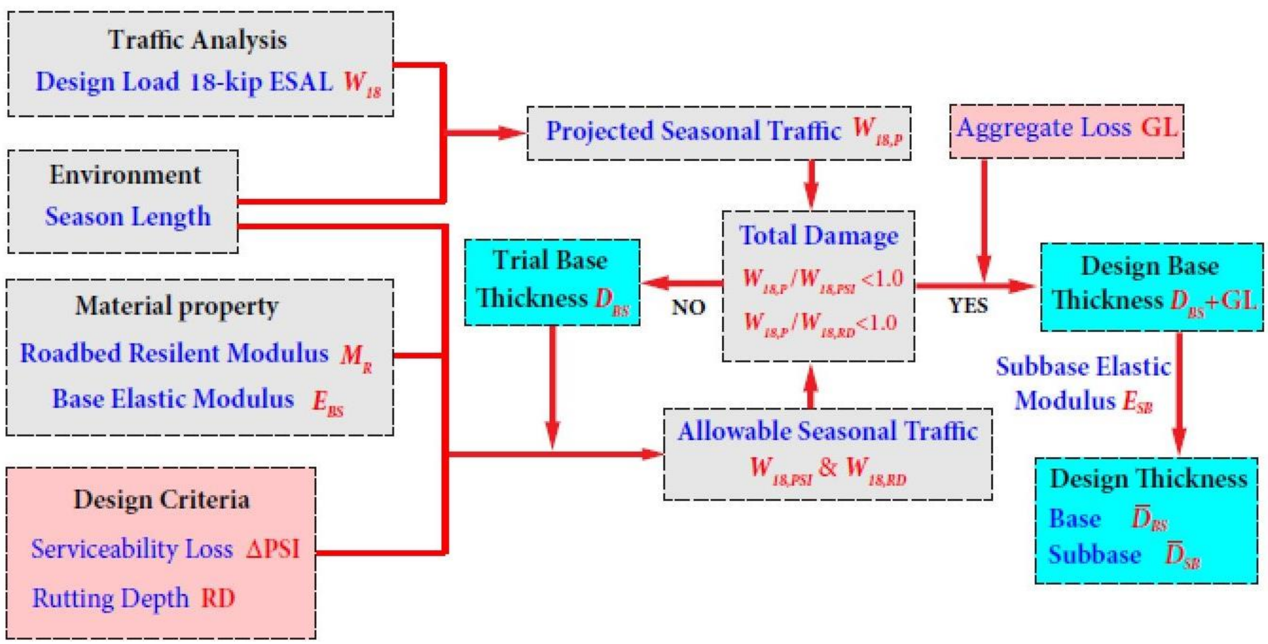

Figure 46 AASHTO Design Chart Procedure for Aggregate Surfaced Road

6.2.1 Determine Allowable ESAL Considering Serviceability Loss

Figure 47 shows the design chart for determining the allowable number of 18-kip ESAL application $W_{18\text{-PSI}}$ considering serviceability loss. The design chart is divided into three portions for processing including the right chart, the nomograph in the middle which converts intermediate parameters between different scales, and the left chart. An axis title "dimensionless parameter *Y*" is added to the vertical axes of the charts as shown in Figure 47 to facilitate the digitization.

The digitized data of the right chart, which represents the relationship between the input base layer thickness D_{BS} (inch) and the value of the intermediate parameter Y_0 for different elastic modulus of base material E_{BS} (psi), are tabulated in Table 13. Linear regression analyses are performed for each set of data, the trend lines are shown in Figure 48. The base layer thickness D_{BS} and dimensionless parameter Y_0 follow a linear relationship which can be expressed as:

$$Y_0 = k * D_{BS} + b, \tag{8}$$

where the slope *k* is dependent on the elastic modulus of base material E_{BS} and the intercept *b* exhibits little variations. The average of the intercept values in Figure 48 which equals to -4.941 will be used in Equation (8). It is found that the relationship between the elastic modulus of base material E_{BS} and the slope *k* can also be well quantified by a liner model:

$$k = 0.1104 * E_{BS} + 0.782. \tag{9}$$

Substituting equation (9) into equation (8) yields,

$$Y_0 = (0.1104 * E_{BS} + 0.782) * D_{BS} - 4.941. \tag{10}$$

Since the resilient modulus of the roadbed M_R (psi)is read on a scale different from the scale of the dimensionless parameter *Y*, the relationship between the two scales needs to be established. This is accomplished by digitizing the M_R values in to the *Y* scale, then using a regression analysis to quantify the relationship between the two scales. The digitized data are presented in Table 14, for instance, a M_R value of 10,000 corresponds to a Y_0 value of 6.3636 on the *Y* scale. A regression analysis as shown in Figure 49 leads to a linear relationship between the two scales given as:

$$Y_1 = -0.4136 * M_R + 10.5. \tag{11}$$

Once Y_0 and Y_1 are obtained, Y_2 can be determined by using the properties of similar triangles as shown in Figure 47:

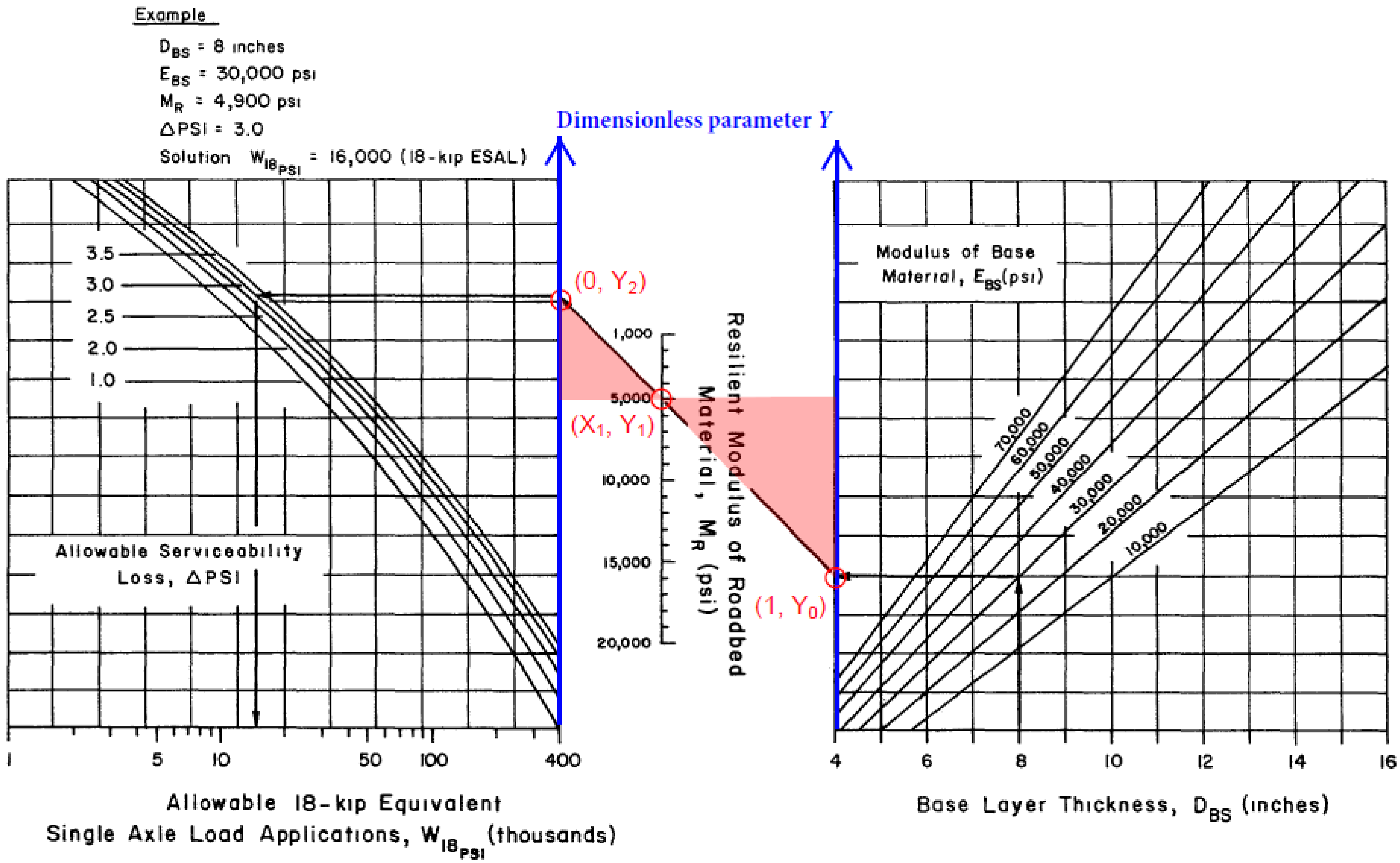

Figure 47 Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss (excerpt from AASHTO 1993)

$$\frac{Y_2 - Y_1}{Y_1 - Y_0} = \frac{X_1}{1 - X_1} \tag{12}$$

where X_1 is the horizontal distance between Y_1 and Y_2 assuming that the horizontal distance between Y_0 and Y_2 is equal to unity. The value of X_1 which equals to 0.375 is obtained from the digitization. Rewritting Equation (12) leads to:

$$Y_2 = \frac{0.375}{1 - 0.375}(Y_1 - Y_0) + Y_1 \tag{13}$$

Combining Equations (10), (11) and (13), Y_2 becomes a function of the design input parameters including resilient modulus of the roadbed M_R, the base layer thickness D_{BS}, and the elastic modulus of base material E_{BS}.

The curves in the left design chart in Figure 47 are digitized into five sets of data (Table 15). Each set of data corresponding to an allowable serviceability loss △PSI consists of 20 points as shown in Table 15. For given Y_2 and △PSI, the allowable number of 18-kip ESAL application $W_{18,\ PSI}$ can be obtained by applying linear interpolations on log-linear scale, i.e., linear interpolations on the data sets of $\log_{10}$ ($W_{18,\ PSI}$) and Y_2 in Table 15.

6.2.2 Determine Allowable ESAL Considering Rutting

An equation for the design chart that is used to determine allowable number of 18-kip ESAL application $W_{18,\ RD}$ considering rutting depth is provided in Yapp, Steward & Whitcomb's review paper (1991):

$$W_{18,RD} = 0.1044 * RD^{2.575} * (\log_{10} D_{BS})^{5.155} * \left(\frac{E_{BS}}{1800}\right)^{3.434} * \left(\frac{M_R}{1800}\right)^{1.048} \tag{14}$$

where RD represents allowable rutting depth (inch), the other input variables and the associated units have been introduced in Subsection 6.2.1.

Table 13 Digitized Data of Base Layer Thickness D_{BS} and Dimensionless Parameter Y_0

E_{BS} = 10,000 psi		E_{BS} = 20,000 psi		E_{BS} = 30,000 psi		E_{BS} = 40,000 psi		E_{BS} = 50,000 psi		E_{BS} = 60,000 psi		E_{BS} = 70,000 psi	
D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0	D_{BS} (in.)	Y_0
5.99	0.31	5.62	0.68	5.04	0.62	4.85	1.00	4.43	1.00	4.48	1.59	4.48	2.02
6.68	0.96	6.58	1.62	5.99	1.74	5.70	2.05	5.20	1.99	5.44	2.96	5.10	3.02
7.58	1.74	7.64	2.71	7.11	2.99	6.69	3.27	5.97	3.05	6.19	3.98	5.98	4.39
8.54	2.61	8.76	3.83	7.99	3.98	7.67	4.45	7.01	4.48	6.96	5.20	6.70	5.48
9.52	3.45	9.82	4.85	9.16	5.29	8.58	5.60	8.02	5.79	7.84	6.44	7.58	6.81
10.40	4.29	11.02	6.13	10.36	6.63	9.78	7.06	9.11	7.28	8.88	7.90	8.64	8.46
11.41	5.16	12.11	7.16	11.45	7.75	11.03	8.56	10.18	8.62	9.76	9.15	9.52	9.86
12.50	6.13	13.14	8.18	12.48	8.96	12.04	9.80	11.24	10.08	10.61	10.42	10.21	10.92
13.62	7.16	14.10	9.12	13.60	10.20	13.08	11.08	12.26	11.39	11.51	11.70	11.15	12.44
14.87	8.28	15.27	10.33	14.75	11.51	14.25	12.51	13.35	12.94	12.63	13.35	11.95	13.66

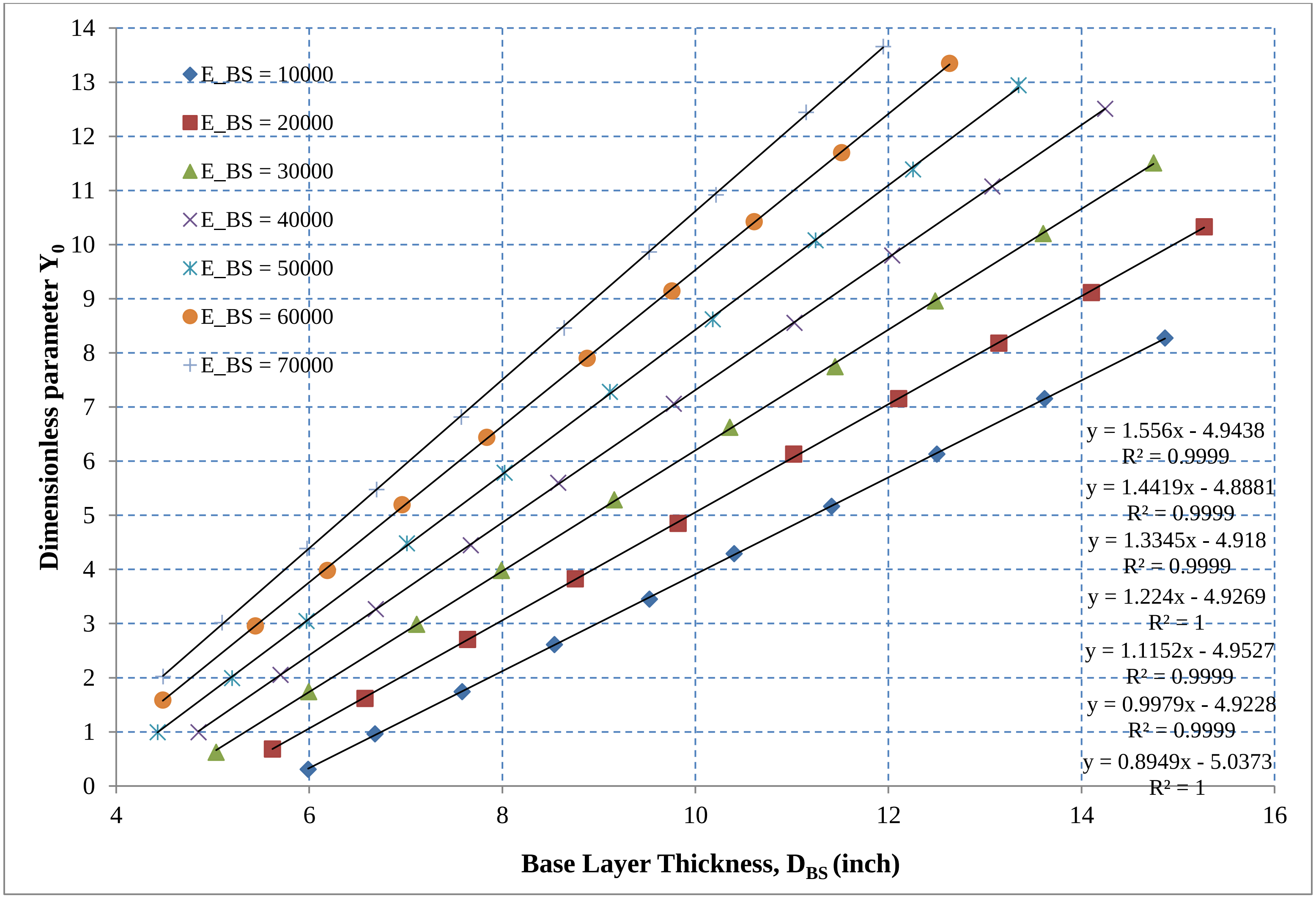

Figure 48 Regression Analysis Between Base Layer Thickness D_{BS} and Dimensionless Parameter Y_0

Table 14 Digitized Data of Resilient Modulus of Roadbed M_R on Y Scale

Resilient modulus of roadbed M_R (psi)	Dimensionless parameter Y_1
20,000	2.2273
15,000	4.2955
10,000	6.3636
5,000	8.4318
1,000	10.0545

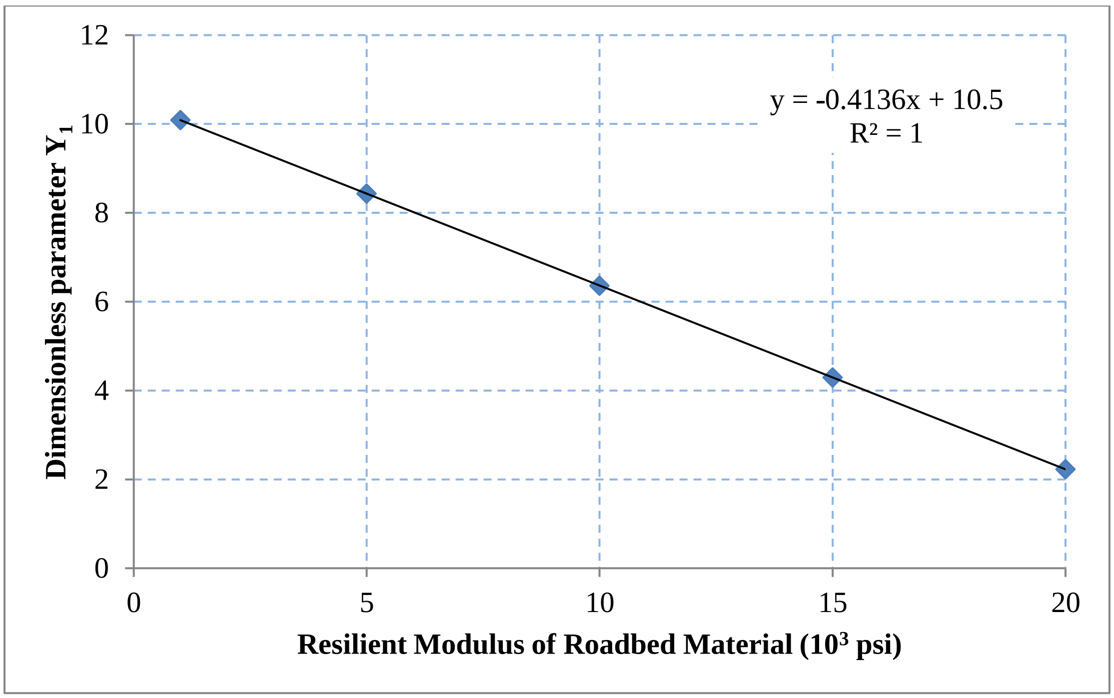

Figure 49 Regression Analysis Between Resilient Modulus of Roadbed M_R and Dimensionless Parameter Y_1

Table 15 Digitized Data of Allowable ESAL $W_{18,\ PSI}$ and Dimensionless Parameter Y_2

ΔPSI = 1.0			ΔPSI = 2.0			ΔPSI = 2.5			ΔPSI =3.0			ΔPSI = 3.5		
W_{18}	$\log_{10}(W_{18})$	Y_2	W_{18}	$\log_{10}(W_{18})$	Y_2	W_{18}	$\log_{10}(W_{18})$	Y_2	W_{18}	$\log_{10}(W_{18})$	Y_2	W_{18}	$\log_{10}(W_{18})$	Y_2
2.18	0.34	13.90	2.53	0.40	14.00	2.89	0.46	14.01	3.30	0.52	13.98	3.62	0.56	14.04
2.70	0.43	13.52	3.38	0.53	13.45	3.66	0.56	13.57	4.35	0.64	13.48	4.65	0.67	13.54
3.63	0.56	12.98	4.35	0.64	13.01	5.17	0.71	12.92	5.82	0.77	12.92	6.22	0.79	13.01
5.23	0.72	12.28	5.67	0.75	12.51	7.20	0.86	12.30	8.00	0.90	12.33	8.55	0.93	12.42
7.33	0.87	11.65	7.58	0.88	11.92	9.88	0.99	11.67	10.42	1.02	11.80	11.29	1.05	11.86
10.13	1.01	10.95	10.15	1.01	11.39	13.40	1.13	11.05	13.40	1.13	11.30	14.71	1.17	11.33
14.19	1.15	10.22	13.40	1.13	10.77	18.65	1.27	10.34	17.01	1.23	10.77	19.67	1.29	10.74
19.61	1.29	9.49	18.16	1.26	10.09	24.30	1.39	9.71	21.87	1.34	10.24	25.30	1.40	10.18
25.34	1.40	8.85	24.29	1.39	9.43	32.51	1.51	9.09	27.38	1.44	9.71	33.84	1.53	9.50
33.18	1.52	8.22	30.82	1.49	8.87	42.35	1.63	8.44	35.67	1.55	9.09	45.87	1.66	8.78
42.29	1.63	7.55	40.69	1.61	8.19	55.17	1.74	7.78	44.66	1.65	8.56	60.55	1.78	8.10
52.47	1.72	6.95	50.94	1.71	7.60	67.27	1.83	7.22	57.42	1.76	7.91	77.85	1.89	7.44
65.10	1.81	6.35	64.63	1.81	6.97	85.34	1.93	6.57	69.09	1.84	7.41	98.76	1.99	6.76
78.61	1.90	5.75	81.99	1.91	6.23	105.44	2.02	5.92	86.50	1.94	6.79	122.01	2.09	6.11
98.85	1.99	5.05	105.40	2.02	5.45	130.27	2.11	5.26	106.87	2.03	6.17	148.77	2.17	5.48
122.64	2.09	4.41	133.70	2.13	4.70	165.26	2.22	4.52	133.80	2.13	5.48	179.02	2.25	4.98
156.28	2.19	3.59	165.18	2.22	3.99	204.16	2.31	3.80	172.00	2.24	4.77	209.78	2.32	4.46
196.49	2.29	2.76	212.31	2.33	3.15	255.59	2.41	3.08	215.32	2.33	4.02	242.58	2.38	3.93
275.14	2.44	1.52	280.21	2.45	2.21	311.60	2.49	2.37	276.76	2.44	3.15	299.68	2.48	3.18
390.48	2.59	0.19	384.70	2.59	0.97	384.92	2.59	1.56	385.08	2.59	2.00	385.20	2.59	2.34

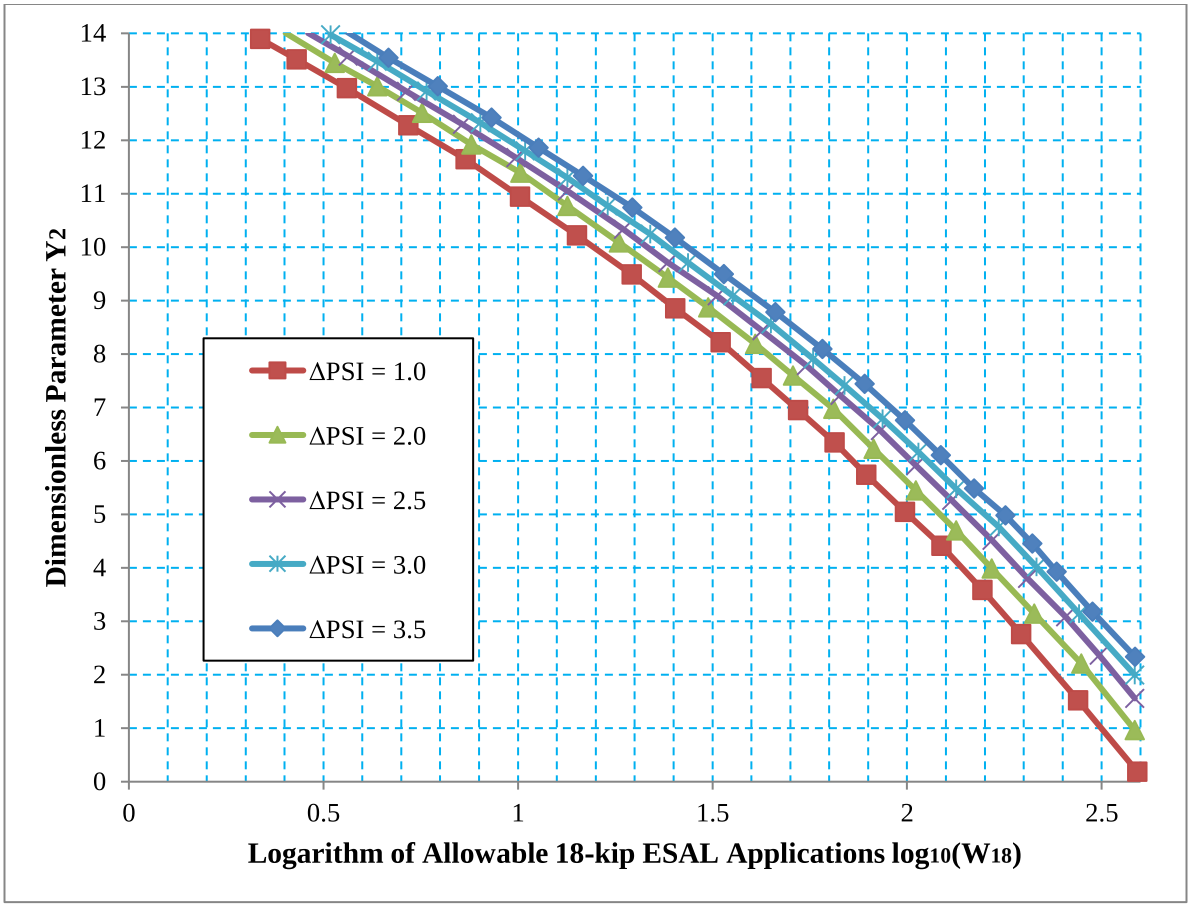

Figure 50 Plot of Allowable ESAL $W_{18, PSI}$ and Dimensionless Parameter Y_2 in Log-Linear Scale

6.2.3 Convert Base to Equivalent Sub-base Thickness

In the design of low volume roads with aggregate surface, significant reduction in the cost may be achieved by using local materials more extensively. An inferior sub-base material can be placed under the base layer, which can reduce the thickness of the more expensive base layer. A nomograph as shown in Figure 51 in this design chart procedure can be utilized to convert a portion of the base layer into an equivalent thickness of sub-base layer D_{SB}.

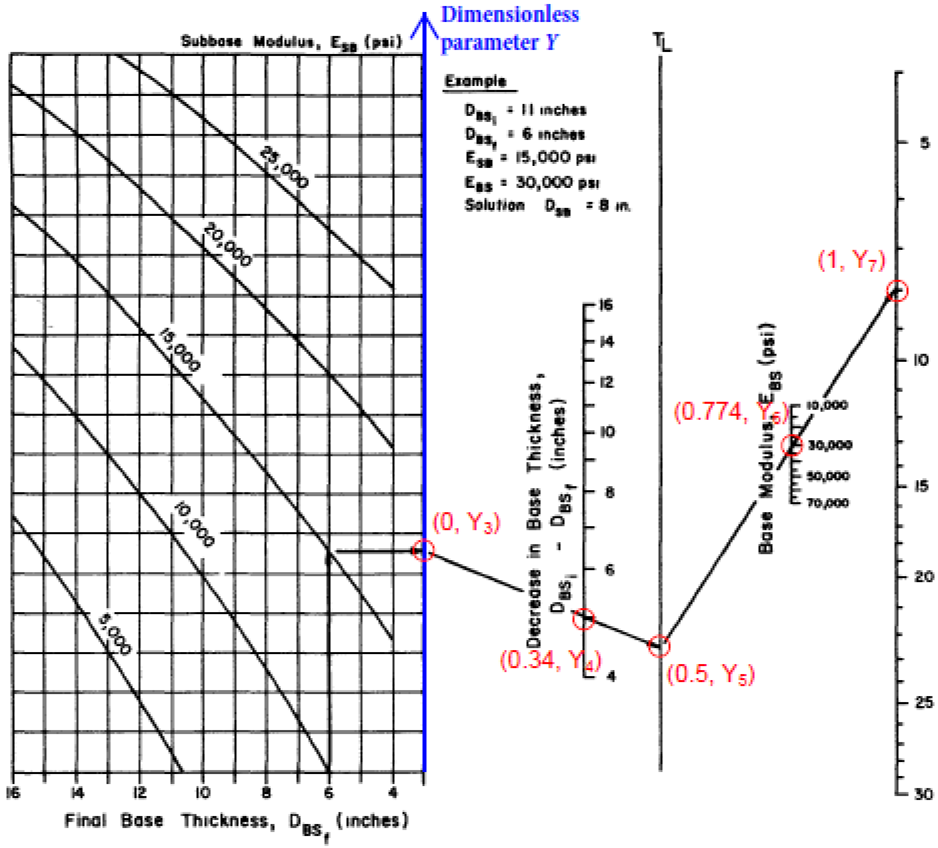

Figure 51 Design Chart for Converting Base to Sub-base Thickness (excerpt from AASHTO 1993)

Following the method used in Subsection 6.2.1, introducing a dimensionless parameter Y to the nomograph in Figure 51 to transfer variables between different scales and performing digitization.

The relationship between the final base layer thickness $D_{BS,f}$ and the dimensionless parameter Y_3 can be characterized by a second order polynomial obtained from regression analyses of the digitized data:

$$Y_3 = a * D_{BS,f}^2 + b * D_{BS,f} + c, \quad \textbf{(15)}$$

where the coefficients a, b, and c are tabulated in Table 16 for different the elastic modulus of sub-base E_{SB} (psi). Linear interpolation will be employed to evaluate Y_3 for intermediate values of E_{SB}.

Table 16 Coefficients in Equation (15) for Different Elastic Modulus of Sub-base E_{SB}

Elastic modulus of sub-base E_{SB} (psi)	a	b	c
25,000	-0.0134	0.9021	8.7581
20,000	-0.0141	1.0301	4.358
15,000	-0.016	1.2251	-1.1806
10,000	-0.0269	1.6406	-8.7051
5,000	-0.0286	1.9678	-17.617

Similar to the method determining Y_1, Y_4 can be expressed as

$$Y_4 \ = \ 15.43 * \log_{10}\left(D_{BS} - D_{BS,f}\right) - \ 6.9195. \tag{16}$$

Applying the properties of similar triangles:

$$Y_5 \ = \ Y_4 - \frac{(0.498 - 0.339)}{0.39} * (Y3 - Y4) \tag{17}$$

Repeat the steps in equations (16) and (17) to find Y_6 and Y_7:

$$\log_{10} Y_6 \ = \ -2.26 \times 10^{-6} * E_{BS} + 0.9794 \tag{18}$$

$$Y_7 \ = \ \frac{1 - 0.774}{0.774 - 0.498} * (Y6 - Y5) + Y6 \tag{19}$$

The final step is to convert the dimensionless parameter Y_7 to the sub-base thickness D_{SB}. Since multiple scales present in the D_{SB} scale, D_{SB} (inch) is expressed by a piecewise function of Y_7:

$$\log_{10} D_{SB} \ = \begin{cases} -0.0324 * Y_7 + 1.4526, & Y_7 < 4.794 \\ -0.0558 * Y_7 + 1.5692, & 4.794 < Y_7 < 10.213 \\ -0.0549 * Y_7 + 1.5600, & Y_7 > 10.213 \end{cases} \tag{20}$$

6.2.4 Implementation and Verification

The study in Subsections 6.2.1, 6.2.2 and 6.2.3 converts a chart-based method into equation-based solutions which facilitates to program the design procedure of gravel roads by basic computational tools. MATLAB is employed to implement the equation-based method, and a computational package named Gravel Road Designer (GRD) is developed. The source code of the computation can be found in Appendix F.

In order to verify the accuracy of the equations developed and validate the implementation of the method, results obtained from GRD program are compared with those obtained by the chart-based method which are documented in AASHTO Guide for Design of Pavement Structures (1993) and Gravel Road Maintenance and Design Manual (Skorseth and Selim, 2000) issued by US Department of Transportation (USDOT). The design input summary for the two benchmark examples are summarized in Table 17.

Table 17 Design Input Summary for Two Examples

Design Input	AASHTO	USDOT
Traffic W_{18} (18-kip ESAL)	21,000	35,000
Climatic Region	III	VI
Quality of Roadbed Material	Poor	Good
Elastic Modulus of Base Material E_{BS} (psi)	30,000	25,000
Elastic Modulus of Sub-base Material E_{BS} (psi)	15,000	15,000
Initial Design Thickness of Base Layer D_{BS} (in.)	8	10
Allowable Serviceability Loss ΔPSI	3.0	2.0
Allowable Rutting Depth *RD* (in.)	2.5	2.0
Allowable Aggregate Loss *GL* (in.)	2.0	1.0

The plots of total damage versus base layer Thickness generated by the GRD program are compared with the plots excerpted from the two examples as shown in Figure 52 and Figure 53. The GRD output damage curves for the two design criteria, serviceability loss and rutting depth, are represented by blue and pink solid lines, respectively. It can be seen that the damage curves produced by the GRD program agree well with the results in the benchmark examples. The design thickness of base layer for the AASHTO example generated by GRD program is 10 inches which exactly equals the benchmark result. The design thickness is dominant by the serviceability loss criteria in this example. The design thickness of base layer for the USDOT example generated by GRD program is 13.1 inches, which is close to the benchmark result 12.9 inches. Herein, the design thickness is controlled by the rutting criteria. One set of intermediate trial solutions are also outputted to compare with the benchmark solutions as shown in Table 18 and Table 19 for the two examples. It shows that the GRD solutions are comparable to the benchmark solutions. The only notable difference is marked by red at Column 5 of Table 19 which may be a typo in the example as this value will not lead to the result in Column 7.

In order to account for the loss of aggregate during the performance period, half of the estimated gravel loss shall be added on the base layer design thickness which results in the base layer design thickness equal to $D_{BS} + 0.5\times GL$ (AASHTO, 1993).

In the two examples, a portion of the base layer is converted into equivalent sub-base thicknesses. The elastic moduli of sub-base materials used in the two examples are both 15,000 psi. In the AASHTO example, 5 inches of good gravel base is converted to an equivalent sub-base layer with a thickness of 8 inches. In the USDOT example, 7.5 inches of good gravel base is converted to an equivalent sub-base layer with a thickness of 11 inches. The equivalent sub-base layer thicknesses produced by the GRD program are 7.6 inches and 11 inches for the AASHTO and USDOT examples, respectively.

Therefore, it can be demonstrated the GRD program can produce design thickness of gravel roads with sufficient accuracy. Taking advantage of the equations, the design process is programmed and can generate results in seconds rather than performing a number of iterations by the original chart-based design procedure. The GRD program will be used in the parametric study of the application of Lean Oil Sand on gravel road design in the subsequent section.

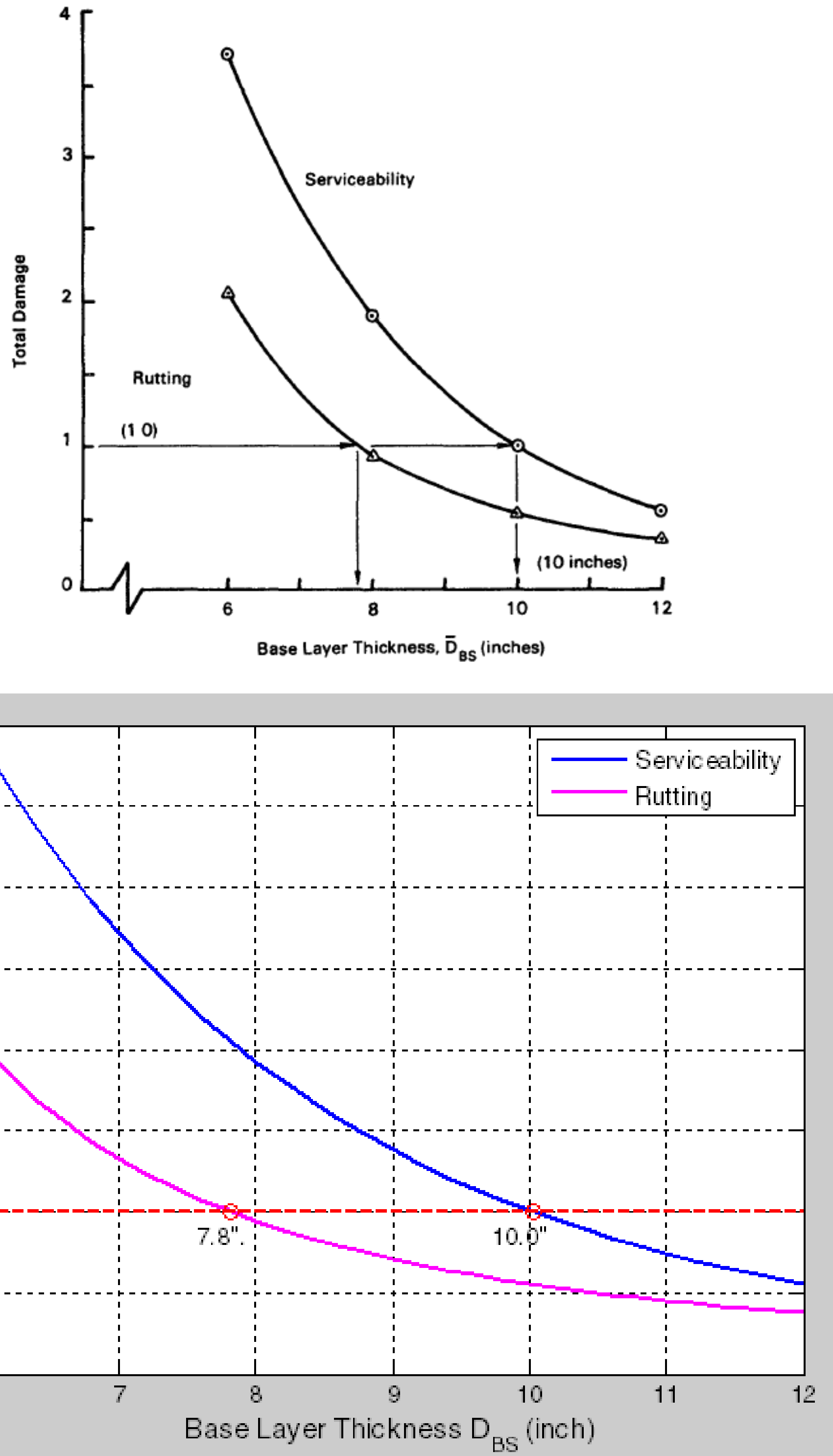

Figure 52 Comparison of Total Damage versus Base Layer Thickness Plot (AASHTO Example)

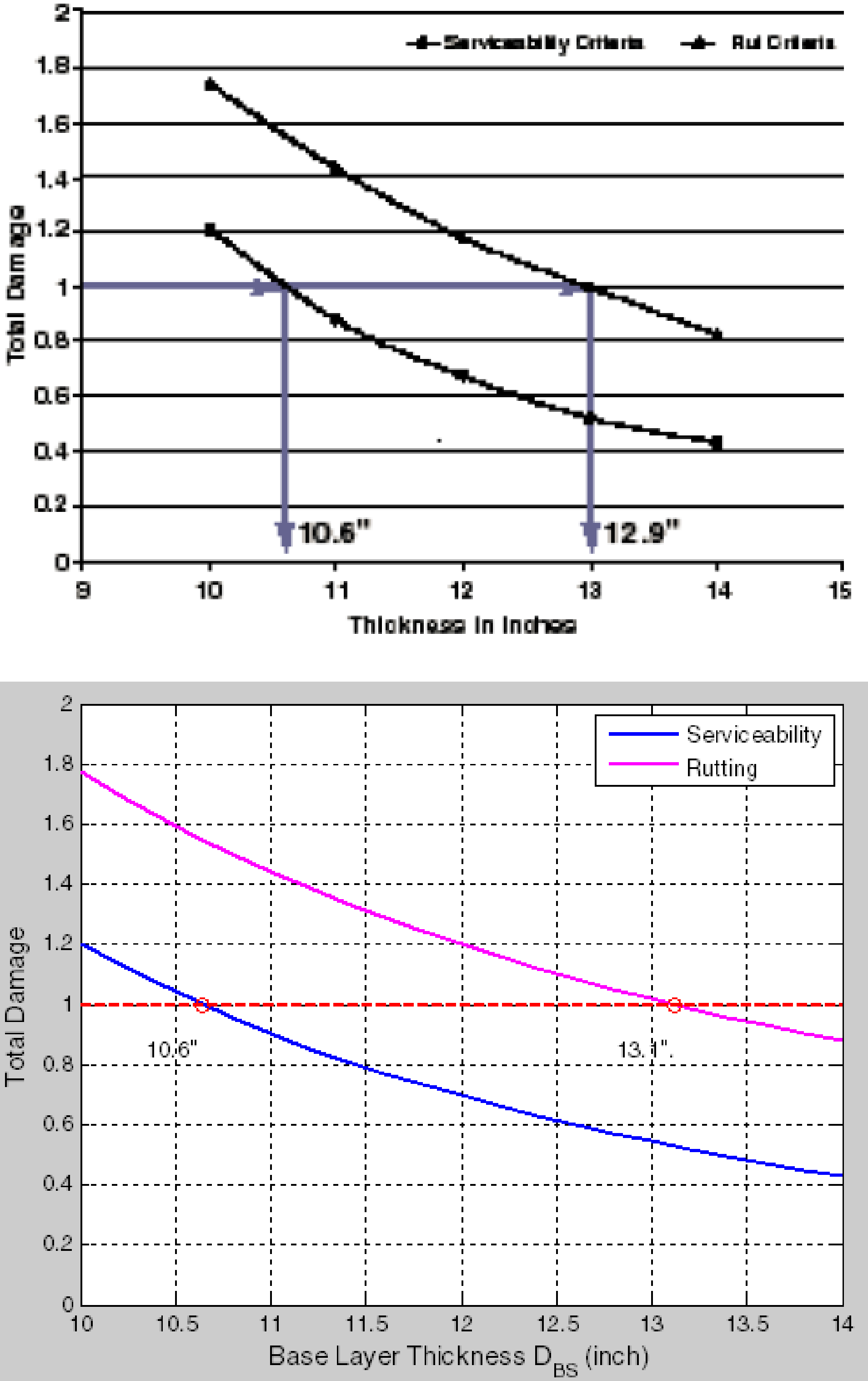

Figure 53 Comparison of Total Damage versus Base Layer Thickness Plot (USDOT Example)

Table 18 Comparison of Trial Solutions between GRD and AASHTO Example

Trial Base Thickness D_{BS} = 8 in.					Serviceability Criteria ΔPSI = 3.0				Rutting Criteria RD = 2.5 in.			
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, W_1 /$W_{18,PSI}$		7 Allowable 18-kip ESAL Traffic ($W_{18,RD}$)		8 Seasonal Damage, W_1/$W_{18,RD}$	
			GRD	AASHTO	GRD	AASHTO	GRD	AASHTO	GRD	AASHTO	GRD	AASHTO
Winter (Frozen)	20,000	30,000	4,375	4,400	400,000	400,000	0.01	0.01	127,930	130,000	0.03	0.03
Spring/Thaw (Saturated)	1,500	30,000	2,625	2,600	4,486	4,900	0.59	0.53	8,473	8,400	0.31	0.31
Spring/Fall (Wet)	3,300	30,000	7,000	7,000	8,335	8,400	0.84	0.83	19,360	20,000	0.36	0.35
Summer (Dry)	4,900	30,000	7,000	7,000	14,219	16,000	0.49	0.44	29,297	29,000	0.24	0.24
Total Traffic =			21,000	21,000	Total Damage =		1.93	1.81	Total Damage =		0.94	0.93

Table 19 Comparison of Trial Solutions between GRD and USDOT Example

Trial Base Thickness D_{BS} = 10 in.					Serviceability Criteria ΔPSI = 2.5				Rutting Criteria RD = 2.0 in.			
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, W_1 /$W_{18,PSI}$		7 Allowable 18-kip ESAL Traffic ($W_{18,RD}$)		8 Seasonal Damage, W_1/$W_{18,RD}$	
			GRD	USDOT	GRD	USDOT	GRD	USDOT	GRD	USDOT	GRD	USDOT
Winter (Frozen)	20,000	25,000	8,750	8,750	400,000	400,000	0.022	0.022	65,122	80,000	0.134	0.109
Spring/Thaw (Saturated)	2,000	25,000	4,375	4,375	6,947	18,500	0.630	0.643	5,831	5,800	0.750	0.754
Spring/Fall (Wet)	6,000	25,000	8,750	8,750	24,367	25,000	0.359	0.350	18,440	19,000	0.475	0.461
Summer (Dry)	10,000	25,000	13,125	13,125	68,089	67,000	0.193	0.196	31,495	31,500	0.417	0.417
Total Traffic =			35,000	35,000	Total Damage =		1.203	1.211	Total Damage =		1.776	1.741

6.3 Application of Lean Oil Sand on Gravel Road Design

Pavement design is mainly a matter of selecting appropriate layer materials that comprise the pavement and determine the layer thicknesses. Using the LOS as a binding agent and mixed with granular materials to form an asphalt-like base material over existing gravel roads can be beneficial to dust control which is a common distress for gravel road and to save the cost of gravel material in spite that it may lead to a certain reduction of the elastic modulus of the layer. The reduction of the elastic modulus of the layer material mainly depends on the mix ratio between LOS and gravels. Using the GRD program developed in Section 6.2, a parametric study is performed to investigate the feasibility of application LOS on gravel roads from the structural adequacy point of view.

6.3.1 Parametric Study on Gravel Road Design

The design input parameter affected by introducing LOS into the gravel base material is the elastic modulus of the base material. Generally, a higher mixing percentage of LOS will lead to a lower elastic modulus of the LOS-gravel mixture. In this study, assuming that the climatic region is VI where the resources of LOS in Canada are located and the quality of roadbed material is categorized as “very good”. The relationship between the elastic modulus of the base layer E_{BS} and the resultant design thickness D_{BS} is investigated for different magnitude of traffic load W_{18}. Two extreme cases for design criteria are considered:

(1) Allowable serviceability loss ΔPSI = 1.0 and allowable rutting depth RD = 1 inch (Figure 54);

(2) Allowable serviceability loss ΔPSI = 3.5 and allowable rutting depth RD = 3 inches (Figure 55);

where the design thickness curves satisfying the allowable serviceability loss ΔPSI and allowable rutting depth RD are represented by solid and dashed lines, respectively. And the pairs of the solid-dash lines with different colors denote various magnitude of design traffic load W_{18}. For example, the green solid line in Figure 55 represents the design base layer thickness $D_{\mathrm{BS,PSI}}$ which satisfies the design criteria ΔPSI = 3.5 versus the elastic modulus of base material E_{BS} under traffic load W_{18} = 50,000 application of 18-kip ESAL. Each point on the curve is obtained from the intersection point between total damage curve (in blue) and total damage threshold (in red) which is equal to 1.0 as shown in Figure 52 and Figure 53. Similarly, the green dashed line in Figure 55 represents the design

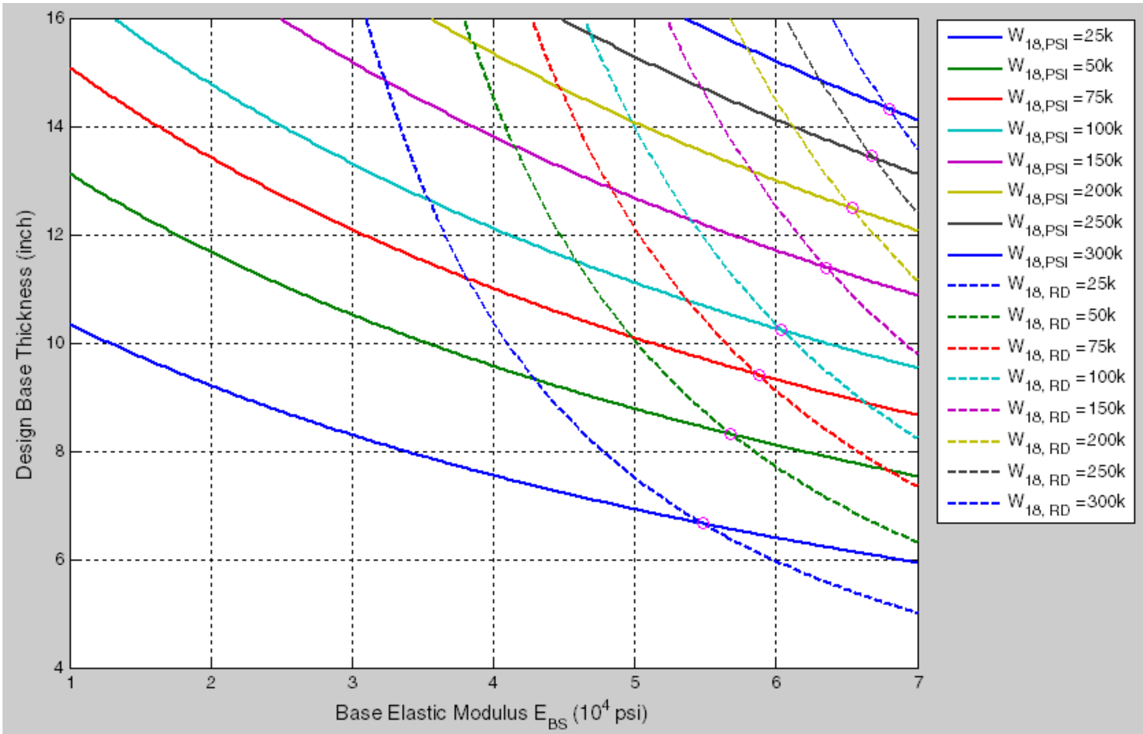

Figure 54 Plot of E_{BS} versus D_{BS} under different W_{18} (ΔPSI =1, *RD*= 1 in.)

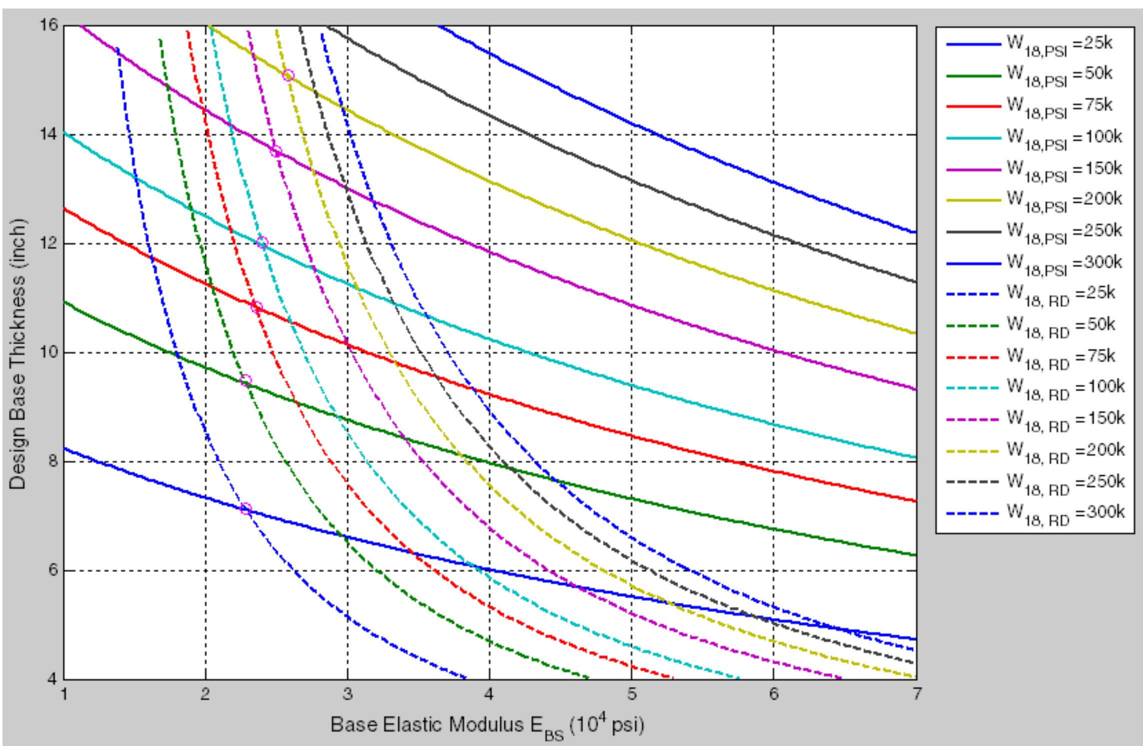

Figure 55 Plot of E_{BS} versus D_{BS} under different W_{18} (ΔPSI =3.5, *RD*= 3 in.)

base layer thickness $D_{BS,RD}$ which satisfies the design criteria RD = 3 inches versus the elastic modulus of base material E_{BS} under traffic load W_{18} = 50,000 application of 18-kip ESAL. Each point on the curve is obtained from the intersection point between total damage curve (in pink) and total damage threshold (in red) which is equal to 1.0 as shown in Figure 52 and Figure 53.

Some observations in Figure 54 and Figure 55 are summarized as follows:

1. To satisfy both of the design criteria for a given traffic design load, there is an "optimal" elastic modulus value and the associated design thickness for the base material which is the intersection point between the solid line and the dashed line. It is marked by a pink circle in Figure 54 and Figure 55.
2. When the elastic modulus of base material is less than the "optimal" value, the design thickness is predominated by the rutting failure mode; when the elastic modulus of base material is greater than the "optimal" value, the design thickness is dependent on the allowable serviceability loss.
3. When the design thickness is predominated by the rutting failure mode (the elastic modulus is less than the "optimal value"), the design thickness is very sensitive to the value of the base material elastic modulus due to a high rate of change for the dashed curves. In contrast, the design thickness is less sensitive to the change of the elastic modulus of base material.
4. For given design criteria, there are not large variations with respect to the "optimal" values for the elastic modulus of base material. For a high design requirement with ΔPSI =1 and RD= 1 inch, the "optimal" value for the elastic modulus of base material varies from 55,000 to 67,000 psi for design load in a range of 25,000 to 300,000 application of 18-kip ESAL. For a less restrictive design requirement with ΔPSI =3.5 and RD= 3 inches, the "optimal" value for the elastic modulus of base material falls in the relatively narrow range between 23,000 to 28,000 psi for design load of 25,000 to 300,000 applications of 18-kip ESAL.

6.3.2 Recommendation on Application of LOS on Gravel Roads

Some recommendations about applying LOS on gravel road structural design are provided based on the observations from the parametric study:

1. For gravel roads that require high performance, high quality granular materials are usually used to satisfy the design criteria. The LOS can be used as a binding agent and mixed with

granular materials to help mitigate the dust effect. A low mixing percentage of LOS should be used in this case to maintain the elastic modulus of the base layer. The CBR test for a mixing ratio of 30% LOS and 70% granular materials has been performed and it provided a CBR value of 80%. Although the CBR value mainly represents the load-carrying capacity of materials rather than the elastic modulus, high CBR values normally correspond to a higher elastic modulus based on some empirical equations that correlate the CBR and elastic modulus. Mixing a low percentage of LOS which is less than 30% is not anticipated to significantly reduce the elastic modulus of granular materials, and it can contribute to dust control.

2. For gravel roads which require a relatively lower performance, a higher amount of LOS may be applied into the mixture with granular materials. In this situation, the introduction of LOS serves two functions: (1) saves the cost of virgin materials; and (2) mitigates the dust effect. From gravel road structural design point of view, an ideal mixing ratio of LOS and granular materials should result in the "optimum" values of the elastic modulus as shown in Figure 55 for the mixture. For example, assuming the elastic modulus of base material equals to 50,000 psi for the design scenario in Figure 55. Assuming a 50% LOS and 50% base materials mixing reduces the elastic modulus by half based on the CBR test value of the 50/50 mixture. If the design load is 50,000 applications of 18-kip ESAL, the design thicknesses for the pure base material and the 50/50 mixture are 7.5 inches and 9 inches, respectively. The amount of the granular material per unit volume is reduced by 50% while the design thickness is only increased by 20%. Additional save on the cost may be achieved by converting a portion of the 50/50 mixture layer thickness to an equivalent sub-base thickness comprised by 30/70 mixtures.

It should be emphasized that the recommendations provided above are mainly based on the structural thickness design of gravel roads, other factors should also be taken into account when applying LOS in the gravel road design. However, it provides a preliminary recommendation of LOS use in gravel road design and the benefits of using LOS.

Chapter 7
Summary and Future Research

7.1 Summary

Gravel roads, which are the major type of low volumes roads take up approximately 60% of the public road network in Canada. It requires lower construction skill levels and cost, and can provide simple transportation needs in remote and rural areas. Unlike paved roads, distresses on surface gravel are more rapidly revealed and gravel roads generally require more frequent maintenance.

Kearl Oil Sand site is located in Alberta, Canada. The roads within the site serve various purposes, such as site access, personnel transportation. As a result of increasing traffic loadings, the surface of the roadways can deteriorate rapidly. The failure is experienced in the top 150-200 mm of the surface course. In addition, the oil sands mining leads to significant volumes stockpiling of LOS material, which cannot be processed economically but still has low grade bitumen content in it.

This research investigates the feasibility of LOS on gravel roads by incorporating LOS into granular materials as base and surface courses materials. A field site survey, a suite of laboratory tests, as well as base and sub-base thickness design provided multiple resources for assessing the feasibility of incorporating LOS into gravel road design.

The main findings from this research are summarized below:

- Stabilization operations can be required to restore performance and remediate various surface distresses, including dust, loose gravel, potholes at Kearl Lake site.
- LOS treated roads reduce dust significantly. The instances of their use on site showed some potential benefits which might be better realized with improved design, such as different mixing percentages other than 50/50, construction, and maintenance plans. Also, clumps of LOS were observed on the shoulders which indicate some instability of the material.
- The bitumen contents of LOS provided by Imperial Oil Inc. were found in a range of 3-4.5% by weight from extraction test, which can be defined as low graded oil sand.
- Sieving analysis showed using LOS alone and the mixture of 30G/70LOS are only suitable for sub-base course materials. Granular material used on site, 50G/50LOS and 70G/30LOS LOS-granular mixtures are applicable for both base and sub-base materials. The test results

indicated that a higher percentage of granular material provides a higher strength. However, none of the tested materials are suitable for surfacing material due to lack of fine content.

- The results from proctor test showed the mixture of 70G/30LOS has the highest maximum dry density (MDD) and the lowest optimum moisture content (OMC), which leads to a higher shear strength, lower permeability and compressibility. It was observed that more LOS incorporated results in a decrease of MDD and an increase of OMC.
- The CBR test is to evaluate the potential strength of sub-base and base course materials. The value was determined from the samples with OMC obtained from Proctor test. The mixture of 70G/30LOS has the highest CBR and the mixture of 30G/70LOS has the lowest CBR value. Also, no potential of swelling has been noticed in any mixtures.
- For gravel roads that require high performance, LOS can be used as a binding agent and mixed with granular materials for mitigating the dust effect. A low mixing percentage of LOS should be used in this case to maintain the elastic modulus of the base layer. For gravel roads which require a relatively lower performance, a higher amount of LOS may be applied into the mixture with granular materials. In this situation, the introduction of LOS can save the cost of granular materials and mitigate the dust effect.

7.2 Future Research

The following are recommended area for future work:

- The findings indicate that LOS has potential use on gravel roads. However, the issue of instability of LOS which is a natural unmodified material should be further studied.
- Both LOS and granular materials lack fine aggregates. Additives may be introduced to the LOS-granular mixture to develop surface materials in the future study. For example, adding 1% asphalt binder to improve the performance and provide a tight, smooth surface.
- Resilient modulus of base material is used as design input in current practice. Tri-axial tests should be performed to evaluate the resilient modulus of the LOS-granular mixtures.
- This work is mainly focus on satisfying the strength requirements for LOS incorporated materials. Other properties should also be examined including permeability, durability, shear strength etc.

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Appendix A

Moisture Content Laboratory Test

Material			
Parameter	Sample	LOS	Granular
	Container ID	BLL-3	ES-4
M1	Container mass (g)	297.0	122.2
M2	Original sample + Container mass (g)	2397	3432.1
M3	Dried sample + Container mass (g)	2258.0	3209.014
M4	Evaporable Water (g)	139.026	223.0865
M5	Dried sample (g)	1961.0	3086.814
M6	Moisture Content (%)	7.09	7.23

Mass of Evaporable Water = M2-M3

Mass of Dried Sample = M3-M1

Moisture Content = 100*(M4/M5)

Appendix B
LOS Extraction Test

Extraction Test - LOS						
Sample No.	1	2	3	4	5	6
Paper filter (g)	18.9	19.6	19.3	19.4	19.2	19.5
Mass of oven dried paper filter-Mf(g)	18.4	18.8	18.6	19.1	18.5	18.6
Mass of cup -Mc(g)	230.8	277.4	278.0	257.6	285.3	278.4
Mass of LOS sample (g)	1564.2	1728.3	1675.2	1800.1	1624.1	1855.9
Mass of oven dried paper filter(after extraction)-Mfe (g)	19.4	19.5	19.4	19.4	19.2	19.0
Mass of empty bowl (g)-Mb	120.7	152.4	365.8	408.4	154.8	181.0
Mass of empty bowl+extracted aggregate (after oven dry)- Mba (g)	1545.4	1707.2	1884.0	2036.8	1628.0	1859.0
Mass of cup (after extraction) -Mce (g)	233.2	289.2	288.4	263.1	292.9	289.3
Mass of cup (after extraction&after oven dry) (g)	230.9	277.4	278.2	257.8	285.5	278.3
Moisture content- Z(%)	5.1	7.0	7.0	7.1	5.3	6.1
W1 (g)	1564.2	1728.3	1675.2	1800.1	1624.1	1855.9
W2 (g)	1488.9	1615.3	1565.7	1680.4	1542.4	1748.5
W3 (g)	1.0	0.7	0.8	0.3	0.7	0.4
W4 (g)	1424.7	1554.8	1518.2	1628.4	1473.2	1678.0
W5 (g)	0.1	0.0	0.2	0.2	0.2	-0.1
W6 (g)	1425.8	1555.5	1519.2	1628.9	1474.1	1678.3
W7 (g)	63.1	59.8	46.4	51.5	68.4	70.2
W8 (%)	4.24	3.70	2.97	3.06	4.43	4.02

W1	Mass of LOS Sample	
W2	Mass of LOS Dried Sample	W2=100*W1/(100+Z)
W3	Mass of Fines in Filter	W3=Mf-Mfe
W4	Mass of Extracted Aggregate after Extraction	W4=Mba-Mb
W5	Mass of Fines in the Extract	W5=Mce-Mc
W6	Mass of Dried Aggregate	W6=W3+W4+W5
W7	Mass of Asphalt Binder	W7=W2-W6
W8	Asphalt Binder Content	W8=100*W7/W2

Appendix C

Gradation Test

1. LOS (100%)

LOS (Sample 1)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
19	0.0	0.0	0.0	100.0
12.5	104.2	1.2	1.2	98.8
9.5	977.4	11.4	12.6	87.4
4.75	1824.0	21.3	33.9	66.1
2.36	511.0	6.0	39.9	60.1
1.18	1291.5	15.1	55.0	45.0
0.6	847.5	9.9	64.9	35.1
0.3	541.4	6.3	71.2	28.8
0.15	1665.6	19.5	90.7	9.3
0.075	559.0	6.5	97.2	2.8
<0.075	238.6	2.8	100.0	0
Weight of dry sample (g)	8560.2			

LOS (Sample 2)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
19	0.0	0.0	0.0	100.0
12.5	77.1	0.9	0.9	99.1
9.5	454.4	5.2	6.1	93.9
4.75	1496.5	17.2	23.3	76.7
2.36	709.3	8.1	31.4	68.6
1.18	2129.4	24.4	55.8	44.2
0.6	837.3	9.6	65.4	34.6
0.3	246.8	2.8	68.2	31.8
0.15	1817.6	20.8	89.1	10.9
0.075	654.9	7.5	96.6	3.4
<0.075	298.4	3.4	100.0	0
Weight of dry sample (g)	8721.7			

LOS (Sample 3)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
19	0.0	0.0	0.0	100.0
12.5	99.3	1.1	1.1	98.9
9.5	613.6	6.8	7.9	92.1
4.75	2021.3	22.4	30.3	69.7
2.36	297.8	3.3	33.6	66.4
1.18	1913.0	21.2	54.8	45.2
0.6	1218.2	13.5	68.3	31.7
0.3	-63.2	-0.7	67.6	32.4
0.15	1958.1	21.7	89.3	10.7
0.075	667.7	7.4	96.7	3.3
<0.075	297.8	3.3	100.0	0

Weight of dry sample (g) 9023.5

2. Granular (100%)

Granular (Sample 1)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
25	0.0	0.0	0.0	100.0
19	1802.9	7.8	7.8	92.2
12.5	4329.4	18.7	26.5	73.5
9.5	2487.2	10.8	37.3	62.7
4.75	3416.4	14.8	52.1	47.9
2.36	1972.6	8.5	60.6	39.4
1.18	1400.9	6.1	66.7	33.3
0.6	1748.7	7.6	74.2	25.8
0.3	2868.4	12.4	86.6	13.4
0.15	1434.2	6.2	92.8	7.2
0.075	533.7	2.3	95.2	4.8
<0.075	1119.7	4.8	100.0	0.0

Weight of dry sample (g) 23114.2

Granular (Sample 2)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
25	0.0	0.0	0.0	100.0
19	1436.7	6.6	6.6	93.4
12.5	5072.0	23.3	29.9	70.1
9.5	1852.2	8.5	38.4	61.6
4.75	3546.3	16.3	54.7	45.3
2.36	1044.9	4.8	59.5	40.5
1.18	1691.2	7.8	67.3	32.7
0.6	1573.8	7.2	74.5	25.5
0.3	2177.0	10.0	84.5	15.5
0.15	1567.3	7.2	91.7	8.3
0.075	776.1	3.6	95.3	4.7
<0.075	1030.7	4.7	100.0	0.0

Weight of dry sample (g) 21768.2

Granular (Sample 3)				
Sieve opening size (mm)	Weight of sample retained (g)	Percentage of wt. retained (%)	Cumulative percentage of wt. retained (%)	Percentage of wt. passing (%)
25	0.0	0.0	0.0	100.0
19	1709.4	6.8	6.8	93.2
12.5	4550.0	18.1	24.9	75.1
9.5	3971.8	15.8	40.7	59.3
4.75	2941.1	11.7	52.4	47.6
2.36	2664.6	10.6	63.0	37.0
1.18	276.5	1.1	64.1	35.9
0.6	3494.2	13.9	78.0	22.0
0.3	2488.7	9.9	87.9	12.1
0.15	1256.9	5.0	92.9	7.1
0.075	980.4	3.9	96.8	3.2
<0.075	804.4	3.2	100.0	0.0

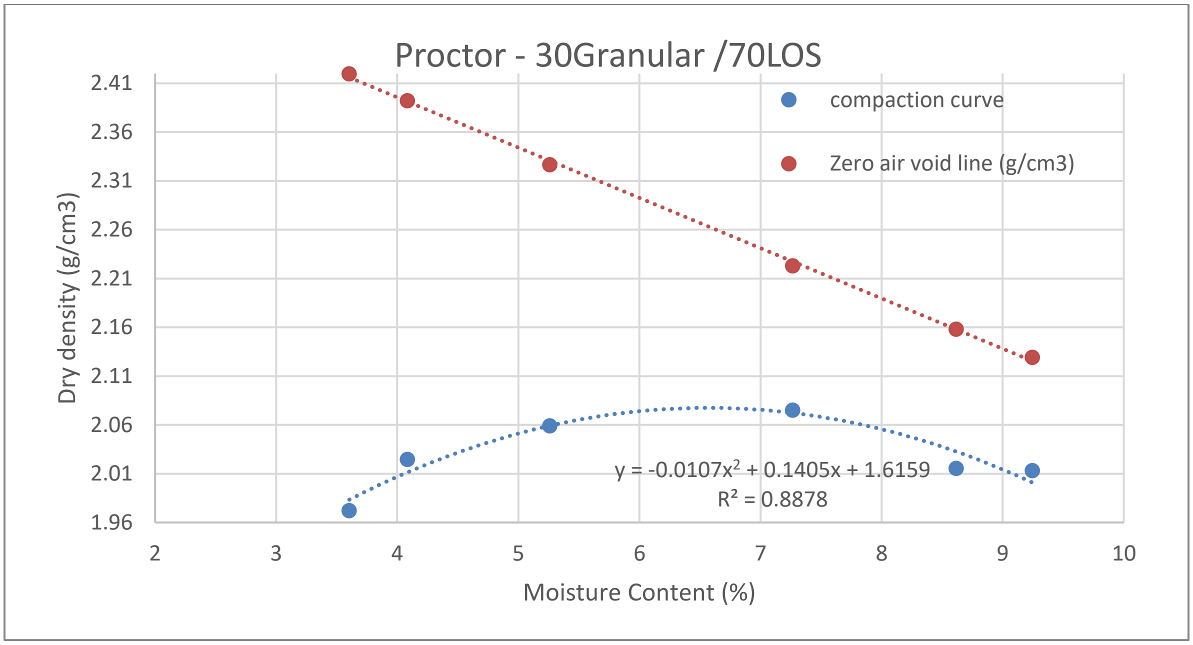
Weight of dry sample (g) 25137.9

Appendix D

Proctor Test

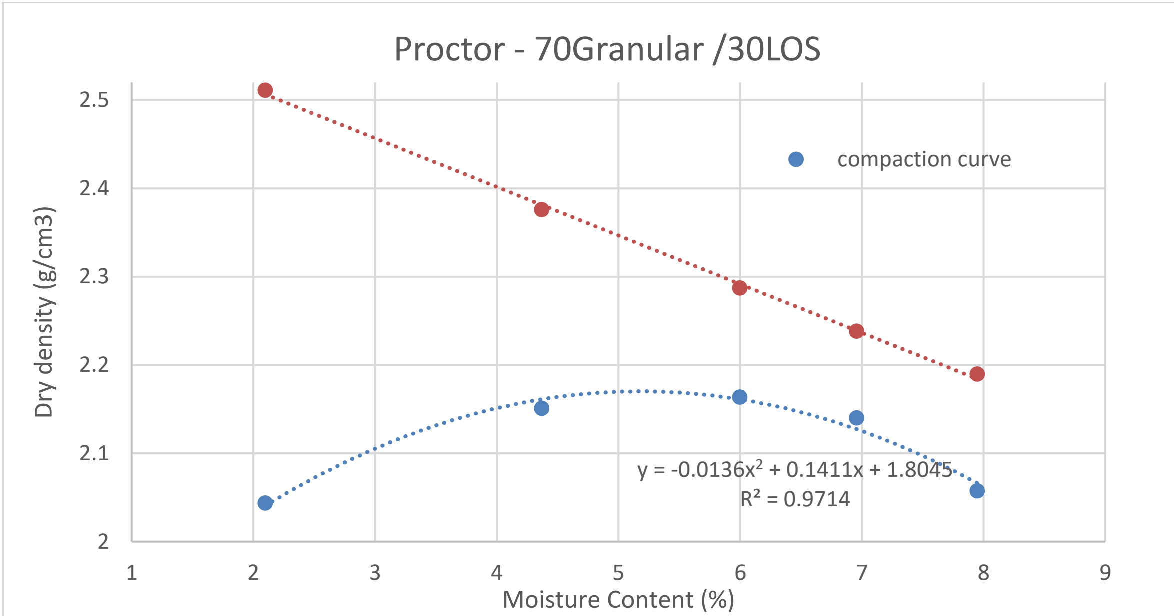
1. 30G/70LOS

Proctor Test - 30Granular /70LOS						
Wet density						
Water percent	0%	1%	2%	4%	5%	6%
Trial No.	1	2	3	4		5
Water added (g)	0.0	22.6	44.4	94.7	113.8	140.0
Weight of Mold + Wet Soil (g)	6683.9	6744.6	6801.8	6857.2	6824.5	6831.9
Weight of Mold (g)	4745.7	4745.7	4745.7	4745.7	4747.8	4745.7
Weight of Wet Soil (g)	1938.2	1998.9	2056.1	2111.5	2076.7	2086.2
Volume (cm3)	948.6	948.6	948.6	948.6	948.6	948.6
Wet density (g/cm3)	2.0	2.1	2.2	2.2	2.2	2.2
Moisture content						
Container ID	ES-2	4.75	37.5	BLL-1	19	EH-4
Weight of wet soil+container (g)	568.8	714.4	626.9	750.9	2209.5	846.0
Weight of dry soil+container (g)	554.4	693.6	601.6	712.3	2046.3	796.1
Weight of container (g)	154.8	184.1	120.6	181.0	152.3	256.3
Weight of water (g)	14.4	20.8	25.3	38.6	163.2	49.9
Weight of dry soil (g)	399.6	509.5	481.0	531.3	1894.0	539.8
Moisture content	3.60	4.08	5.26	7.27	8.62	9.24
Dry Density (g/cm3)	1.97	2.02	2.06	2.08	2.02	2.01
Zero air void line (g/cm3)	2.42	2.39	2.33	2.22	2.16	2.13
Proportions for a batch						
No.	1	2	3	4	5	6
Mass (g)	2283.4	2261.4	2221.8	2368.4	2428.3	2332.9



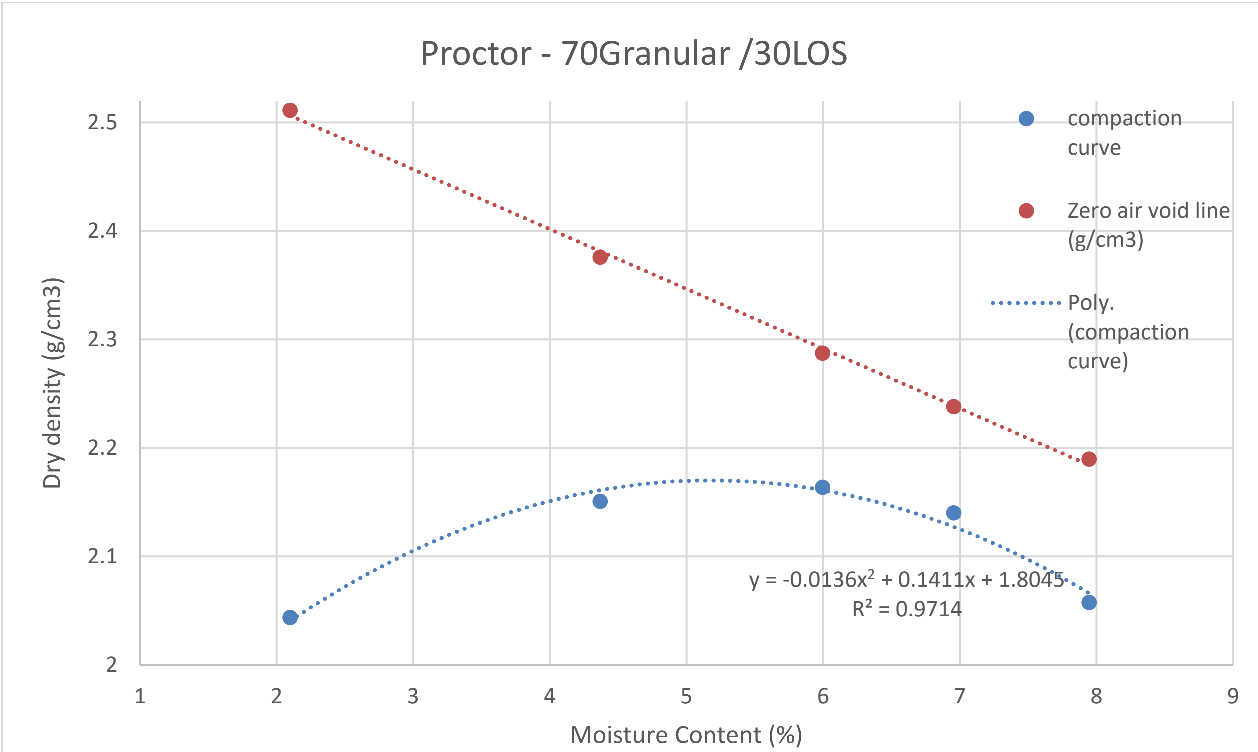
2. 50G/50LOS

Proctor Test - 50Granular /50LOS						
Wet density						
Water percent	0.50%	2%	3.5%	4%	5%	6%
Trial No.	1	2	3	4	5	6
Water added (g)	10.6	41.7	81.3	87.8	112.8	182.5
Weight of Mold + Wet Soil (g)	6750.1	6833.6	6899.0	6903.7	6886.4	6858.4
Weight of Mold (g)	4748.7	4748.7	4747.9	4748.7	4747.6	4748.7
Weight of Wet Soil (g)	2001.4	2084.9	2151.1	2155.0	2138.8	2109.7
Volume (cm3)	948.6	948.6	948.6	948.6	948.6	948.6
Wet density (g/cm3)	2.1	2.2	2.3	2.3	2.3	2.2
Moisture content						
Container ID	19	EN-1	ES-2	2.36	UC-1	B
Weight of wet soil+container (g)	903.3	1038.4	2276.2	1107.5	2246.7	1145.2
Weight of dry soil+container (g)	882.9	1004.8	2164.5	1052.5	2110.5	1072.8
Weight of container (g)	152.2	182.5	154.8	152.4	124.0	121.7
Weight of water (g)	20.4	33.6	111.7	55.0	136.2	72.4
Weight of dry soil (g)	730.7	822.3	2009.7	900.1	1986.5	951.1
Moisture content	2.79	4.09	5.56	6.11	6.86	7.61
Dry Density (g/cm3)	2.05	2.11	2.15	2.14	2.11	2.07
Zero air void line (g/cm3)	2.47	2.39	2.31	2.28	2.24	2.21
Proportions for a batch						
No.	1	2	3	4	5	6
Mass (g)	2281.7	2265.9	2476.6	2347.5	2407.8	3163.9



3. 70G/30LOS

Proctor Test - 70Granular/30LOS					
Wet density					
Water percent	1%	3%	5%	6%	7%
Trial No.	1	2	3	4	5
Water added (g)	23.6	74.0	115.9	140.0	133.0
Weight of Mold + Wet Soil (g)	6726.9	6877.1	6922.9	6918.8	6854.4
Weight of Mold (g)	4747.6	4747.6	4747.3	4747.3	4747.5
Weight of Wet Soil (g)	1979.3	2129.5	2175.6	2171.5	2106.9
Volume (cm3)	948.6	948.6	948.6	948.6	948.6
Wet density (g/cm3)	2.1	2.2	2.3	2.3	2.2
Moisture content					
Container ID	B	ES-2	ES-7	EH-2	4.75
Weight of wet soil+container (g)	2099.6	2281.6	2344.4	2274.2	2190.0
Weight of dry soil+container (g)	2059.0	2192.6	2222.0	2134.0	2042.3
Weight of container (g)	123.5	155.0	180.8	118.8	184.0
Weight of water (g)	40.6	89.0	122.4	140.2	147.7
Weight of dry soil (g)	1935.5	2037.6	2041.2	2015.2	1858.3
Moisture content	2.10	4.37	6.00	6.96	7.95
Dry Density (g/cm3)	2.04	2.15	2.16	2.14	2.06
Zero air void line (g/cm3)	2.51	2.38	2.29	2.24	2.19
Proportions for a batch					
No.	1	2	3	4	5
Mass (g)	2480.7	2620.2	2498.7	2450.4	2086.4



Proctor - 70Granular /30LOS
Dry density (g/cm3)
2.5
2.4
2.3
2.2
2.1
2
1
2
3
4
5
6
7
8
9
Moisture Content (%)
compaction curve
Zero air void line (g/cm3)
Poly. (compaction curve)
y = -0.0136x² + 0.1411x + 1.8045
R² = 0.9714

Appendix E

CBR Test

1. Example of Water Calculation

	Records	Results	Units
From previous test and preparation	Optimum water content (from proctor)-OMC	6.67	%
	Moisture content dry granular&LOS-MC	4.40	%
	Weight of dry granular&LOS for CBR (sample)-M_w	5.90	kg
From calculation	Weight of dry gravel+dry LOS for CBR (from calculation)-M_d	5.65	kg
	Water in the sample (from calculation)	0.25	kg
	Optimum water to be added to dry gravel+wet LOS (from calculation)	0.38	kg
	Water to be added to the sample	128.39	g

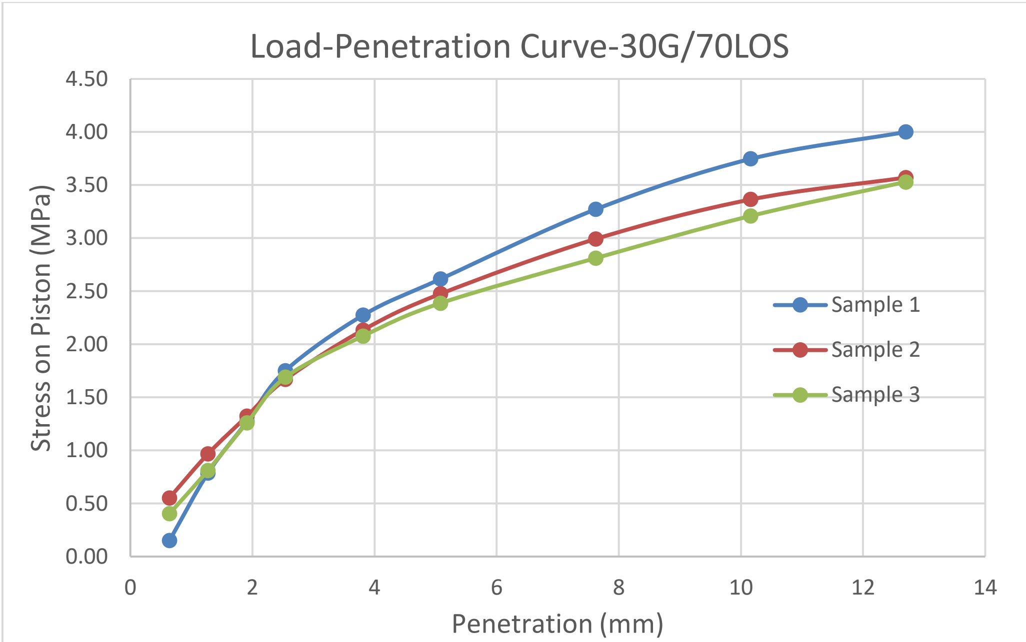
Optimum Water = OMC*M_d/100

Water in the Sample= MC*M_d/100

Water to be Added= Optimum Water-Water in the Sample

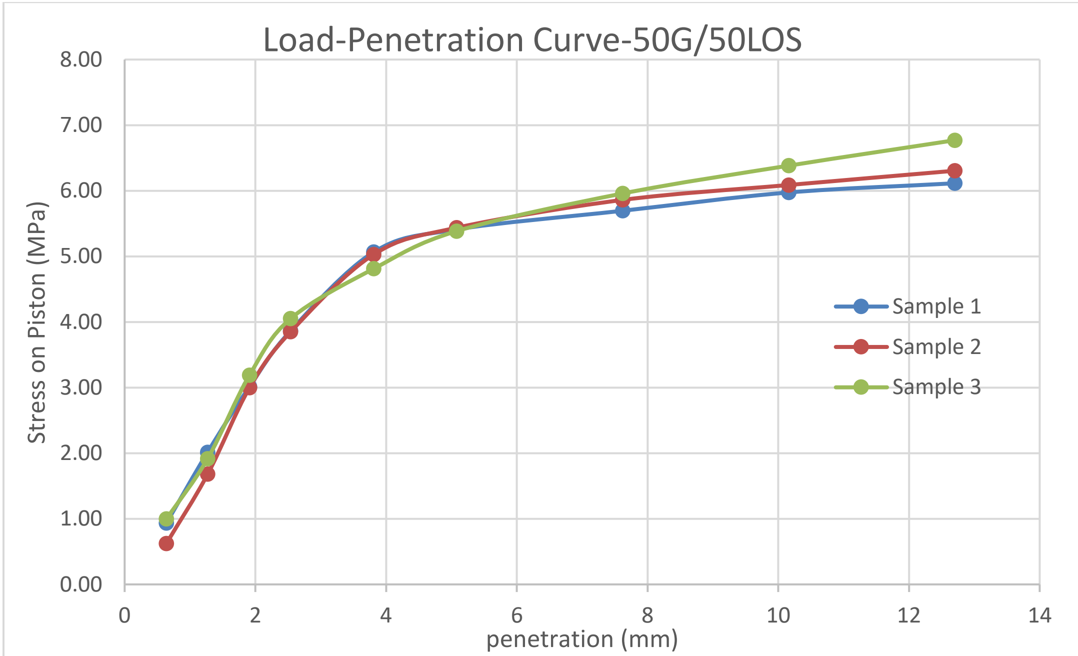
2. 30G/70LOS

AASHTO	30Granular/70LOS					
Penetration(mm)	Load (KN)			Stress (Mpa)		
	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
0.64	1.07	0.29	0.78	0.55	0.15	0.40
1.27	1.87	1.52	1.57	0.97	0.79	0.81
1.91	2.56	2.46	2.44	1.32	1.27	1.26
2.54	3.23	3.39	3.27	1.67	1.75	1.69
3.81	4.13	4.4	4.02	2.13	2.27	2.08
5.08	4.79	5.06	4.62	2.48	2.61	2.39
7.62	5.79	6.33	5.44	2.99	3.27	2.81
10.16	6.51	7.25	6.21	3.36	3.75	3.21
12.7	6.91	7.74	6.83	3.57	4.00	3.53
CBR (2.54)	24.192	25.390	24.492			
CBR (5.08)	24.034	25.388	23.181			



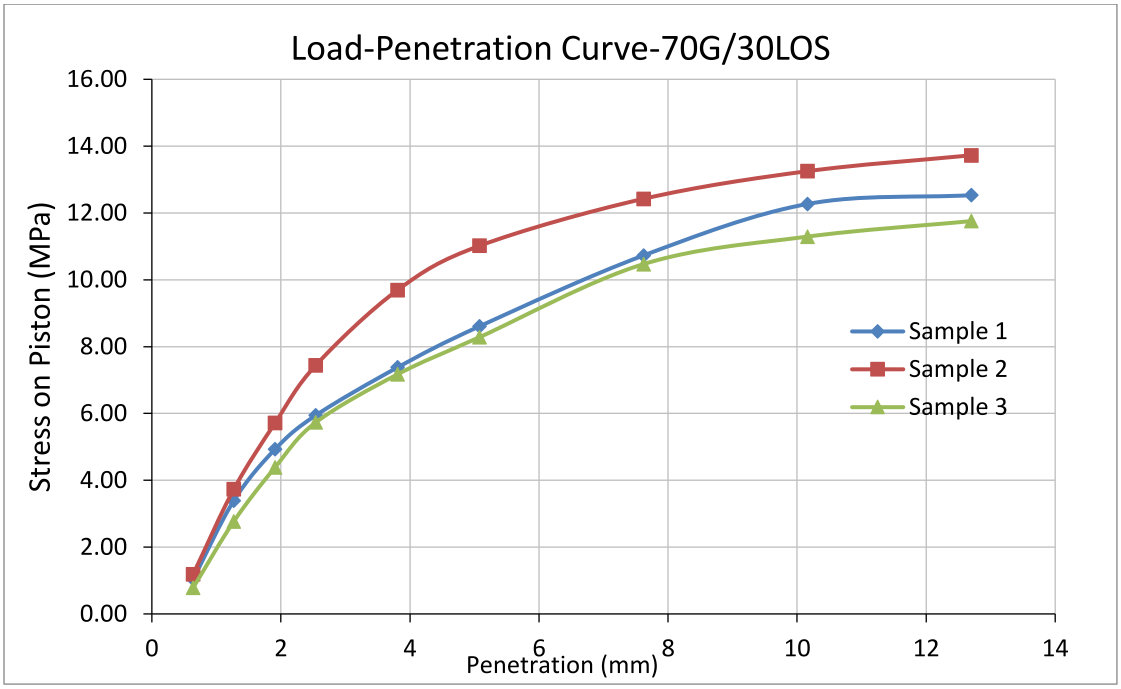
3. 50G/50LOS

AASHTO	50Granular/50LOS					
Penetration(mm)	Load (KN)			Stress (Mpa)		
	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
0.64	1.81	1.21	1.93	0.94	0.63	1.00
1.27	3.9	3.25	3.71	2.02	1.68	1.92
1.91	5.83	5.8	6.17	3.01	3.00	3.19
2.54	7.48	7.45	7.84	3.87	3.85	4.05
3.81	9.8	9.73	9.31	5.06	5.03	4.81
5.08	10.47	10.52	10.42	5.41	5.44	5.39
7.62	11.02	11.34	11.53	5.70	5.86	5.96
10.16	11.56	11.78	12.35	5.97	6.09	6.38
12.7	11.83	12.2	13.1	6.11	6.30	6.77
CBR (2.54)	56.024	55.799	58.720			
CBR (5.08)	52.533	52.783	52.282			



4. 70G/30LOS

AASHTO	70Granular/30LOS					
Penetration(mm)	Load (KN)			Stress (Mpa)		
	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
0.64	2.01	2.3	1.52	1.04	1.19	0.79
1.27	6.56	7.22	5.36	3.39	3.73	2.77
1.91	9.54	11.06	8.49	4.93	5.72	4.39
2.54	11.5	14.4	11.09	5.94	7.44	5.73
3.81	14.28	18.76	13.88	7.38	9.70	7.17
5.08	16.66	21.32	16.03	8.61	11.02	8.28
7.62	20.76	24.04	20.26	10.73	12.42	10.47
10.16	23.74	25.64	21.85	12.27	13.25	11.29
12.7	24.25	26.56	22.75	12.53	13.73	11.76
CBR (2.54)	86.133	107.853	83.062			
CBR (5.08)	83.590	106.972	80.429			



5. Swelling Percentage

Initial length (mm)	5
4 days length (mm)	5
swell percentage (%)	0

Swell percentage = (Change in length in mm during soaking/116.43) *100

Appendix F

MATLAB Code for Gravel Road Design

Main code:

```
clear;
clc;
%% Environment, Roadbed, Traffic
Region =4;     % Choose climatic region 1 to 6
Q_RB = 2;      % Quality of roadbed material: very good =1, good = 2, fair = 3, poor = 4, very poor = 5
E_BS = 25000;   % Resilient modulus of base material
W_18 = 35000;    % Total traffic

% Select Trial Base Thickness
D_BS_1 = 10;    % initial thickness
d_D_BS = 0.2;   % increment
D_BS_2 =14;     % maximum thickness
D_BS = D_BS_1:d_D_BS:D_BS_2;
N_trial = (D_BS_2-D_BS_1)/d_D_BS +1;      % number of trial

% Select allowable serviceability loess and allowable rutting depth
Dt_PSI = 2;   % serviceability loss (psi)
RD = 2;        % allowable rut depth (in.)
GL = 1;        %allowable aggregate loss (inch)

%% Trial calculation
T_damage = zeros(2,N_trial);
for i = 1:N_trial
   D_bs = D_BS(i);
trial = TRIAL(Region, Q_RB, E_BS, W_18, D_bs, Dt_PSI, RD);
T_damage(:,i) = [sum(trial(:, 5),1); sum(trial(:, 7),1)];
end

%% Determine base thickness
d_bs_s = interp1(T_damage(1,:), D_BS, 1);
d_bs_r = interp1(T_damage(2,:), D_BS, 1);
d_bs = max(d_bs_s, d_bs_s);
```

Trail function:

```
function Trial = TRIAL(Region, Q_RB, E_BS, W_18, D_bs, Dt_PSI, RD)

switch Dt_PSI
   case 1
    PSI = 1;
   case 2
```

```
    PSI = 2;
  case 2.5
    PSI = 3;
  case 3
    PSI = 4;
  case 3.5
   PSI = 5;
  otherwise
end

% Assume seasonal resilient moduli and base elastic modulus

Season_length = [0 0 7.5 4.5;    1 0.5 7 3.5;    2.5 1.5 4 4;    0 0 4 8;    1 0.5 3 7.5;    3 1.5 3 4.5];

M_R = [20000   2500   8000   20000; % Very good
20000  2000   6000   10000;          % Good
20000  2000   4500   6500;           % Fair
20000  1500   3300   4900;           % Poor
20000  1500   2500   4000;           % Very poor
20000  5000  10000  20000];          % User defined
% Traffic for each season

W18_P = Season_length(Region,:)/12*W_18;   % Projected 18-kip ESAL

% Determine allowable 18-kip EASL traffic for serviceability criteria

k = 0.1104*E_BS*10^(-4)+0.782;
b = -4.94137;
Y1 = k*D_bs + b;

Y0 = -0.4136*M_R(Q_RB,:)/10^3+10.5;
Y2 = Y0 + 0.375/0.625*(Y0-Y1);

CC = [0.34  13.90   0.40    14.00   0.46    14.01   0.52    13.98   0.56    14.04;
0.43    13.52   0.53    13.45   0.56    13.57   0.64    13.48   0.67    13.54;
0.56    12.98   0.64    13.01   0.71    12.92   0.77    12.92   0.79    13.01;
0.72    12.28   0.75    12.51   0.86    12.30   0.90    12.33   0.93    12.42;
0.87    11.65   0.88    11.92   0.99    11.67   1.02    11.80   1.05    11.86;
1.01    10.95   1.01    11.39   1.13    11.05   1.13    11.30   1.17    11.33;
1.15    10.22   1.13    10.77   1.27    10.34   1.23    10.77   1.29    10.74;
1.29    9.49    1.26    10.09   1.39    9.71    1.34    10.24   1.40    10.18;
1.40    8.85    1.39    9.43    1.51    9.09    1.44    9.71    1.53    9.50;
1.52    8.22    1.49    8.87    1.63    8.44    1.55    9.09    1.66    8.78;
1.63    7.55    1.61    8.19    1.74    7.78    1.65    8.56    1.78    8.10;
1.72    6.95    1.71    7.60    1.83    7.22    1.76    7.91    1.89    7.44;
1.81    6.35    1.81    6.97    1.93    6.57    1.84    7.41    1.99    6.76;
1.90    5.75    1.91    6.23    2.02    5.92    1.94    6.79    2.09    6.11;
1.99    5.05    2.02    5.45    2.11    5.26    2.03    6.17    2.17    5.48;
```

```
2.09   4.41   2.13   4.70   2.22   4.52   2.13   5.48   2.25   4.98;
2.19   3.59   2.22   3.99   2.31   3.80   2.24   4.77   2.32   4.46;
2.29   2.76   2.33   3.15   2.41   3.08   2.33   4.02   2.38   3.93;
2.44   1.52   2.45   2.21   2.49   2.37   2.44   3.15   2.48   3.18;
2.59   0.19   2.59   0.97   2.59   1.56   2.59   2.00   2.59   2.34];

x = interp1(CC(:, 2*PSI), CC(:,2*PSI-1), Y2, 'linear','extrap');
W18_S = 10.^x;
W18_S(W18_S<2) = 2;
W18_S(W18_S>400) = 400;

% Determine allowable 18-kip EASL traffic for rutting criteria
 W18_R = 0.1044*RD^2.575*log10(D_bs)^5.155*(E_BS/1800)^3.434*(M_R(Q_RB,:)/1800).^1.048;
% Reference "Existing Methods for the Structural Design of Aggregate Road Surfaces on Forest Roads" %
TRANSPORTATION RESEARCH RECORD 1291. MARGOT T.Y. et al.

% Determine seasonal damage (serviceability and rutting criteria)
Trial = zeros(4,7);

Trial(:,1) = M_R(Q_RB,:)'; %Roadbed resilient modulus (psi)
Trial(:,2) = E_BS;          % Base elastic modulus
Trial(:,3) = W18_P';        % Projected 18-kip ESAL
Trial(:,4) = W18_S'*1000; % Allowable 18-kip ESAL for PSI
Trial(:,5) = Trial(:,3)./Trial(:,4); % Seasonal damage by PSI
Trial(:,6) = W18_R';      % Allowable 18-kip ESAL for rutting
Trial(:,7) = Trial(:,3)./Trial(:,6); % Seasonal damage by PSI
%Trial
end
```

Convert Base to equivalent sub-base:

```
%% Convert base to equivalent sub-base thickness
D_BS_d = 13.5;

D_BS_f = 6; % final base thickness
E_SB = 15000; %Reslient modulus of sub-base
E_BS = 25000;

Coef = [-0.0134 0.9021  8.7581;
-0.0141 1.0301  4.358;
-0.016  1.2251  -1.1806;
-0.0269 1.6406  -8.7051;
-0.0286 1.9678  -17.617];    % y = a*x^2 + b*x + c

Y3_V = Coef*[D_BS_f^2; D_BS_f; 1];
Y3 = interp1([5 4 3 2 1]'*5000, Y3_V, E_SB);
```

```
Y4 = 15.43*log10(D_BS_d-D_BS_f) - 6.9195;
Y5 = Y4-(0.498-0.339)*(Y3-Y4)/0.339;

Y6 = 10^(-2.26/10^6*E_BS+0.9794);
Y7 = (1-0.774)*(Y6-Y5)/(0.774-0.498)+Y6;

if Y7 < 4.794
   D_SB = 10^(-0.0324*Y7+1.4526);
elseif Y7 >= 4.794 && Y7 < 10.213
   D_SB = 10^(-0.0558*Y7+1.5692);
else
   D_SB = 10^(-0.0549*Y7+1.56);
end
```