University of Southern Queensland

Faculty of Engineering and Surveying

EFFECTIVE ROAD PAVEMENT REHABILITATION FOR LOCAL GOVERNMENT ROADS WITHIN THE SUNSHINE COAST REGION

A dissertation submitted by

Thomas Sanders

in fulfilment of the requirements of

Courses ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

Submitted: October, 2014

Abstract

The Sunshine Coast is located in Southeast Queensland and has one of the largest local government road networks in Australia. The region has been developed on soft estuarine deposits with highly expansive or collapsible soils. The Sunshine Coast Council is continually looking for more effective pavement rehabilitation treatment options to manage the poor subgrade behaviour of the region. Unbound pavements and subgrade replacement are traditionally the dominant pavement rehabilitation methods used within the region.

This dissertation critically evaluates the effectiveness of Sunshine Coast pavement rehabilitation treatments through the analysis of road condition survey data and falling weight deflectometer testing. Initially, seven (7) sites were subjected to surface deflection testing. The surface deflection of pavements under an applied load provided a good indication into the structural integrity of the pavement. The pavement strength of these sites was assessed via plotting measured pavement deflections at various chainages against measured rut depths. Incorporating laser road condition survey data such as roughness and rutting provided a robust dataset to understand pavement conditions. Eight hundred and sixty-six (866) road segments which have been constructed or rehabilitated within the last ten (10) years were tested to assess the long term effectiveness of various pavement types within the region.

Council has been proactive in its approach to pavement rehabilitation, trialling new technologies and searching for cost saving initiatives where appropriate. Council practices are generally sound and in accordance with the latest Austroads and Department of Transport and Main Roads standards and specifications, aligning with current world best practice for pavement design and rehabilitation.

The effectiveness of pavement rehabilitation treatments are case-specific, however, Sunshine Coast practices could be improved by considering sustainable rehabilitation methods including stabilisation, plant mixed foamed bitumen and further use of geosynthetics. Further recommendations include aligning the Sunshine Coast Council Planning Scheme more accurately with Austroads and Department of Transport and Main Roads documentation, accompanied with internal practices for specific subgrade conditions.

University of Southern Queensland

Faculty of Health, Engineering and Sciences

ENG4111/ENG4112 Research Project

Limitations of Use

University of Southern Queensland

Faculty of Health, Engineering and Sciences

ENG4111/ENG4112 Research Project

Certification of Dissertation

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Thomas Sanders

Student Number: 0050097567

Acknowledgements

The author would like to acknowledge the following people for their assistance in completing this dissertation:

- Cameron Shields – Assets Officer, Sunshine Coast Council who provided assistance in the collection of testing and historical data. Cameron also provided guidance to relevant standards and pavement evaluation methods, and outlined Sunshine Coast Council's current road pavement rehabilitation practices.

- Tim Letchford – Operations and Maintenance Manager and Rob Steggall, Works Superintendant at the Sunshine Coast Council who shared their experiences, knowledge and data.

- Professor Ron Ayers: Project Supervisor for his constructive feedback.

- Maryann Sanders - My wife for her personal support throughout the course of this degree.

Thomas Sanders

Student Number: 0050097567

University of Southern Queensland

October 2014

Table of Contents

List of Tables

List of Figures

1.0 Introduction

1.1 Overview

Roads are an integral part of our community and provide the network to support our economy. Since the early 1900s and the formation of a Main Roads board there has been an emphasis on providing a cohesive road network, which would form the backbone of our nation, state and local communities. During this time, road authorities have been faced with the challenge of overcoming financial and technical deficiencies which prevent a cohesive network. Throughout this time road authorities have been required to increase funding to accommodate rapid population and traffic volume growth.

The Sunshine Coast region is not exempt from these challenges, in fact experiencing higher than average population growth and development, placing significant pressure on an ageing network. Since 2001 the population has grown 28.2% and added 70,000 people over this time, with a conservative population growth projection of 38.6% by 2031. However, the Sunshine Coast region only has a population density of 102.7 people per square kilometre, significantly less than regions with comparable road networks such as the Gold Coast (284.2 people per square kilometre) and Greater Brisbane (135.6 people per square kilometre), compounding road infrastructure funding challenges faced within the region.

The approximate value of the road network within the Sunshine Coast region is $1.5 billion, with an average annual construction and maintenance expenditure of $25 million. The Sunshine Coast region consists of 2,650 km of sealed roads with a population of 272,500, predicted to increase substantially within the next two decades. Stretching primarily along the coastal strip from north of Noosa to Caloundra South and as far west as Kenilworth and the Mary Valley. The Sunshine Coast represents a key area for commercial and residential growth over the next 20 years and is tipped to provide the location for many industries to establish and expand; generating further population growth and a higher demand on the regions road infrastructure network. Figure 1 represents the Sunshine Coast region.

Figure 1: Map of the Sunshine Coast Region

To understand the content of this dissertation the components of a road and their function must be understood. A road consists of three major components as shown in Figure 2. These components are:

- The subgrade or the existing ground material;
- The pavement or the structural layer. The depth of this layer varies depending on the strength of the subgrade material (typically 150mm to over 600mm); and
- The wearing running surface i.e. the bitumen or asphalt surface, which provides the waterproof and skid resistance layer.

Theoretically, providing the pavement material is not exposed to water and the design load is not increased dramatically over time, the road pavement will last in excess of 60 – 80 years. To prevent water from saturating the pavement material from rainfall and runoff, the primary treatment is the application of a sealing layer using bituminous products.

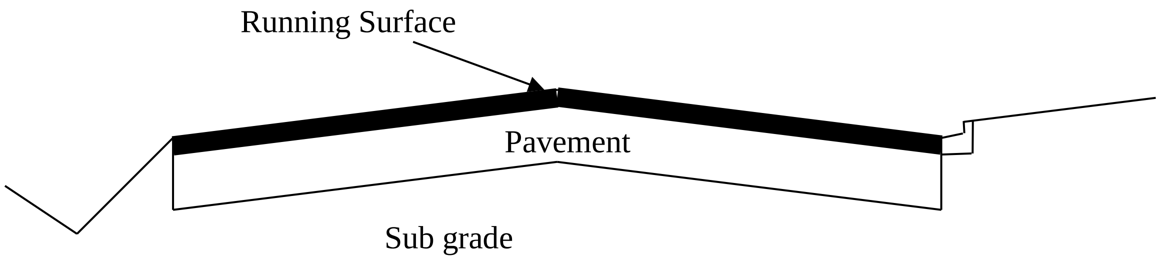


Figure 2: Typical Road Pavement Cross Section

Just as the key components of a road perform different functions, they also have different lifecycles. The bituminous road surface will age and become brittle over time, and under certain conditions will crack. If left untreated, these cracks will allow water to penetrate into the pavement material, which reduces its strength. If the pavement is left in a saturated condition for long enough it will fail and require removal. The challenge for road authorities is to identify which treatments are the most appropriate; at the correct time in the life cycle of the road.

There are various treatments that will be applied to a road over its life cycle. These can include rejuvenation, reseal, rehabilitation and at the end of the pavement's useful life, reconstruction. These treatments increase in cost in accordance to their complexity. The components of the road lifecycle discussed above, relate to the capital expenditure associated with a road. The additional factor to consider in the lifecycle of the road is the ongoing operational and maintenance costs. This expenditure is related to these activities associated with ensuring the road components safely achieve their proposed useful life; and includes such activities as: pothole repairs, pavement repairs and drainage repairs. The required amount of maintenance funding increases significantly the longer roads are left to deteriorate. The challenge is to prevent roads deteriorating to a point where rehabilitation or reconstruction is necessary, and developing the most efficient and cost effective pavement rehabilitation solutions.

1.2 The Sunshine Coast Local Government Network

The Sunshine Coast region has one of the largest road networks in Australia, valued at approximately $1.5 billion, and how this network is managed greatly affects the community. Through a combination of past development, recent wet summers after many years of dry seasons and increased traffic loadings a number of roads on the Sunshine Coast are approaching the end of their useful lives. This has been evident through the increasing number of potholes and pavement failures occurring on an increasing number of roads.

As the Sunshine Coast continues to grow and develop, the assets from the development in the 1980's and earlier are approaching the end of their useful life. This will result in large spikes of rehabilitation and reconstruction needs. With a total sealed road network length of 2,650km or 19,000,000m^2, a replacement value in excess of $1.5 billion and an annual depreciation of $31 million, the Sunshine Coast's road network is one of the largest networks in Australia; and still growing. The majority of the Sunshine Coast sealed road network is made up of minor roads i.e. carry less than 2000 vehicles per day. This dissertation will focus on the sealed road network only; categorised by Figure 3 below

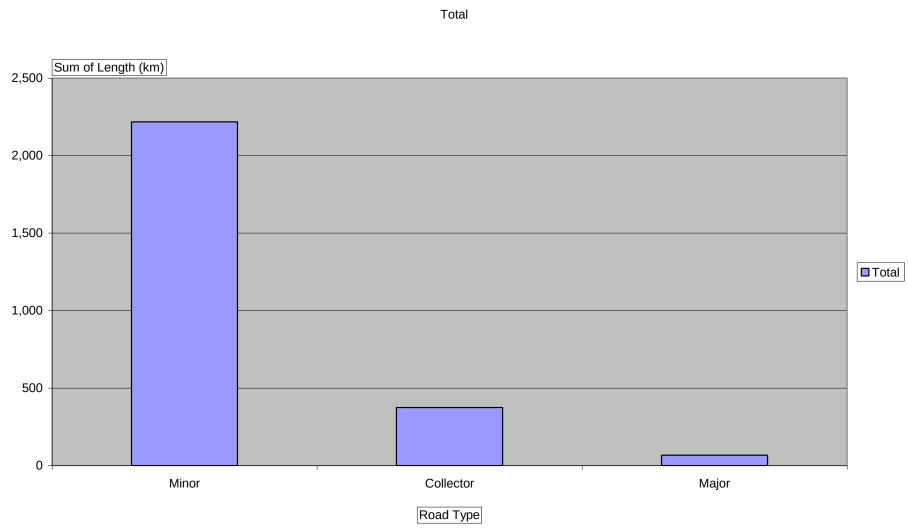


Figure 3: Sunshine Coast Road Network

As previously outlined and seen in the above figure, the Sunshine Coast region consists of a sealed network of approximately 2,600km, the fourth largest sealed network in Queensland, after Gold Coast, Brisbane City and the Department of Transport and Main Roads. The Sunshine Coast sealed road network consists of 1,350km (or 9,200,00m^2) of bitumen sealed surfacing and 1,300 km (or 9,700,000m^2) of asphalt surfacing. Based on a first principles assessment and adopted useful lives for pavement, bitumen surfacing and asphalt surfacing of 75, 15 and 20 years respectively and based on current treatment costs the average annual funding required for bitumen seals and asphalt overlays is $12.5M. The average annual funding required for rehabilitation and reconstruction is $16.5M.

It is noted in areas where growth is still occurring pavements are not achieving their predicted useful lives, especially the principal network roads. This is due to increased traffic loadings on roads that were never designed or constructed to take the large traffic volumes experienced today. This trend is also true for the Sunshine Coast, especially in the older centres, where pavement life is closer to forty or fifty years. Combining the true pavement life with an age analysis of the network shown in Figure 4, a potential spike in road pavements reaching the end of their useful lives is imminent, demonstrated by the increase in potholes and pavement failures evident within the network.

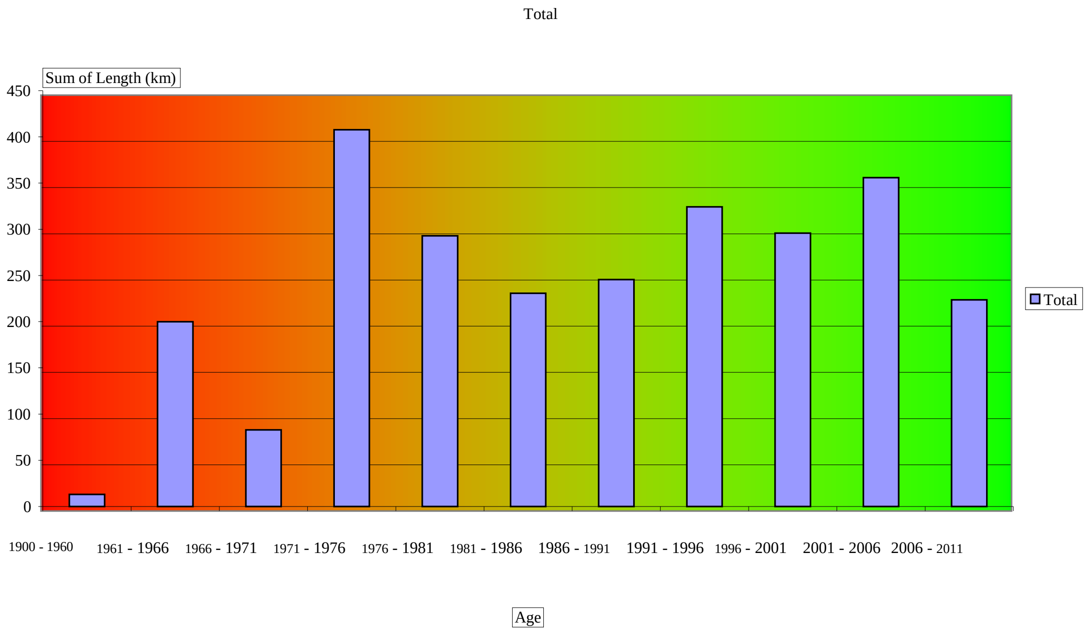


Figure 4: Pavement Age Profile - Sunshine Coast Road Network

Given the size of the network, current available investment levels, existing pavement age, recent wet seasons and increasing growth, it reiterates the importance of sustainable and effective pavement rehabilitation treatments into the future.

1.3 The Problem

Information sourced from the Sunshine Coast Council suggests that approximately 125km of roads (or 4.8% of the network) is considered to be in poor, very poor or failed condition. The network condition is presented in Figure 5.

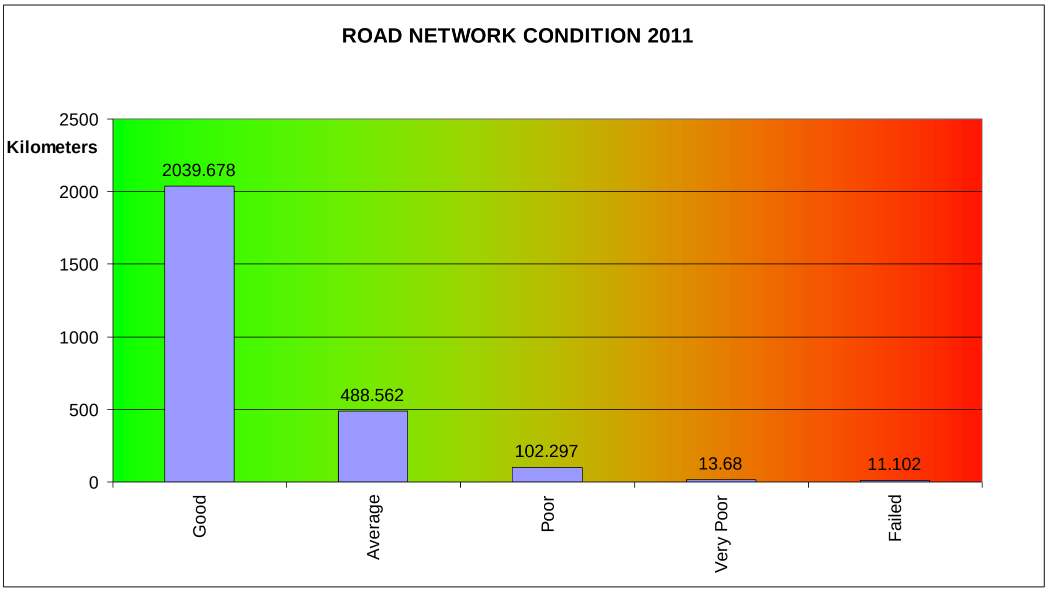


Figure 5: Current Road Network Condition

Between 2000 and 2009 the Sunshine Coast experienced less than average rainfall. This has been followed by some extremely wet years between 2010 and 2012, which approached or exceeded the wettest on record at a number of collection sites, the likes of which have not been experienced since 1999, 1988/89 and 1975, represented in Figure 6 below.

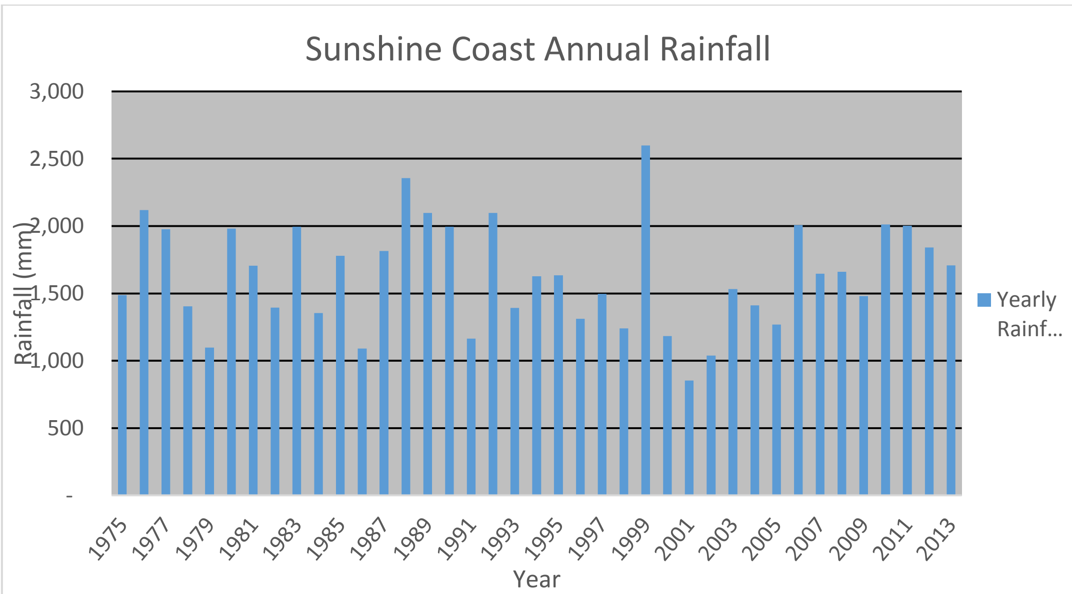


Figure 6: Sunshine Coast Annual Rainfall - Records sourced Bureau of Meteorology

The prolonged period of low rainfall preceding the recent wet seasons has extended the useful lives of many Sunshine Coast road pavements, as the ingress of water into the road pavements was not as prevalent. The increased rainfall in 2010/2011 resulted in many roads within the region becoming saturated, resulting in the increase of potholes and pavement failures. Additional impacts of the rain events are that a number of roads have now deteriorated to a stage where full rehabilitation or renewal is required as the pavement is compromised to a level where a reseal or asphalt overlay is not considered a viable treatment, meaning that the pavement will fail before the overlay reaches the end of its useful life.

Anecdotally, the January 2011 rain events, which were preceded by a very wet 2010, resulted in a rapid deterioration of the road pavement material as a result of a combination of: water entering the pavement through cracked sealed surfaces, water entering the pavement material through elevated water tables and in some locations inundation or flooding. This resulted in weakening of the subgrade and pavement structures of these roads and ultimately leading to surface cracking and pavement failure.

Other causes of rapid deterioration of the road pavements within the region are associated with development of the Sunshine Coast, where roads are experiencing higher than expected traffic loadings associated with new development. This is not only restricted to older streets in ageing suburbs which are now experiencing infill development and the associated construction traffic, but also extends to the principal routes into some of the newer larger estates. Some of the principle routes into these development areas appear to be suffering prematurely from the increased construction traffic associated with the future stages and house construction. The other great unknown to face the Sunshine Coast Council is the unknown quantity and quality of the proposed developments, in particular the Caloundra South development. This single development may incorporate a road network in excess of 50km. While the impact of this will be minimal initially with only minor maintenance required, the longer term maintenance associated with the road and pavement network will present another large spike to be contended with by future rate payers.

To understand the future needs of the road network it is important to understand how the road network was developed. The Sunshine Coast experienced rapid growth in the 1990's and then again in the early 2000's. The historical data also indicates another growth spike in the 1970's and it is these roads that are approaching the end of their useful lives, requiring rehabilitation or reconstruction in the near future. Such spikes in past development increases the amount of road resealing and renewal required, which if not addressed, compounds as more roads deteriorate to a point where rehabilitation or reconstruction are the only available options. As with any road entity budget constraints typically determine the achievable service levels, which are constantly under review in conjunction with investigation of future technologies to maximise the length of network treated with the funds available.

2.0 Literature Review

2.1 Introduction

A Literature review has been completed to establish the requirements of road rehabilitation, and the considerations for design and construction of rehabilitated road pavements. This review considers previous research undertaken on similar topics. This review also provides an overview of the geological history of the Sunshine Coast and the formation of expansive and alluvial clays. Sources of information have been used to outline current material testing procedures, common pavement failure types, recognised subgrade treatment options and to investigate the alternative methods used for pavement rehabilitation both in Australia and Internationally.

To successfully determine effective pavement rehabilitation options for the Sunshine Coast region, literature was reviewed under the following category.

- Geological Properties of the Sunshine Coast
- Coastal Alluvial Sediments
- Pavement Failure Types
- Pavement Evaluation
- Current Test Methods used for Pavement Design
- Moisture in road pavements
- Subgrade Treatment Options
- Alternative Rehabilitation Design Options
- Construction Practices

Information from this research provides a comparison of current Sunshine Coast Council road rehabilitation design and construction practices to world best practices.

3.2 Geological Properties of the Sunshine Coast

The geology of the Sunshine Coast is consistent with much of Southeast Queensland; as it results from a complex and often violent geological history, spanning more than 300 million years. Even the last 800,000 years has seen sea levels fluctuate dramatically, resulting in major changes to the shoreline and coastal environment. These ancient events have determined the present rock formations, minerals, soils, topography, vegetation and present land use in the district.

The oldest rocks exposed on the Sunshine Coast reveal origins dating back to the active growth of the eastern side of the Australian continent, from about 375 to 210 million years ago (Willmott, 2007). Since the volcanic episode of mid-Tertiary times, the region has been geologically stable. In the late Triassic to early Jurassic period between 210 and 180 million years ago, the continental margin essentially stabilised and aged into a number of broad depressions, which began to be filled by sediments eroded from old mountains. With continued sagging, great thickness of sediment accumulated in these basins and gradually hardened. Subsequent sands, silts and muds hardened into sandstone, siltstone and mudstone of the Landsborough sandstone.

Willmott (2007) suggests that the Sunshine Coast has seen the gradual erosion of valleys in areas once covered by basalt, and the lowering of land surface to expose the volcanic plugs since volcanic episode of mid Tertiary times. Soft Alluvial sediments were then deposited along stream valleys, and sands and muds have accumulated along the coastline. After the sea level rose at the completion of the last ice age, most of the flat areas behind the present coastline formed due to sediments and muds of the old bay areas deposited approximately 120,000 years ago are still subject to water logging (Willmott, 2007).

During erosion of the edges of the Buderim plateaux, large volumes of loose rock and soil debris have accumulated on the scarps, on the benches, and on extended aprons covering the older rocks beneath the basalt. These are the very places where groundwater springs are likely. Dark grey of black prairie soils, chocolate soils and black earths are usual on the benches and aprons; many of these contain large quantities of expansive clay minerals such as montmorillonite, which cause the soil to swell on wetting and crack on drying. The swelling is accompanied by a significant decrease in strength of the soil material.

Willmott (2007) advises that although fluctuating groundwater pressures have occurred periodically in wet seasons for thousands of years, and are part of the natural balance, there is evidence that groundwater levels and pressures rise significantly when natural forest cover is removed, mainly through the loss of transpiration by the trees. Therefore, higher peak pressures have developed during intense rainfall than was previously the case.

The present geological formation within the Sunshine Cost is shown in Figures 7, 8 and 9**.**

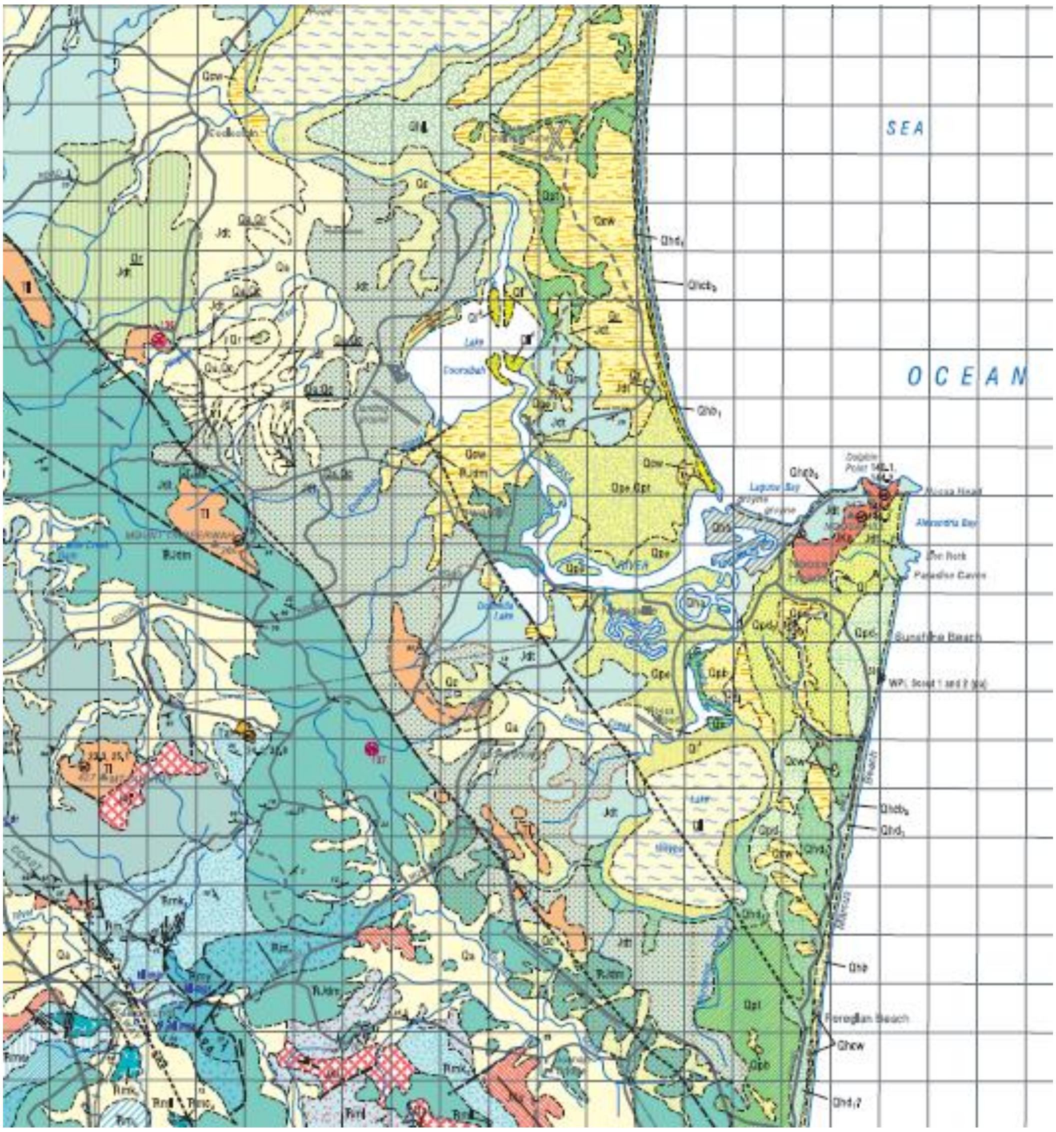


Figure 7: Northern Region Geological Properties - Department of Mines and Energy, 1999

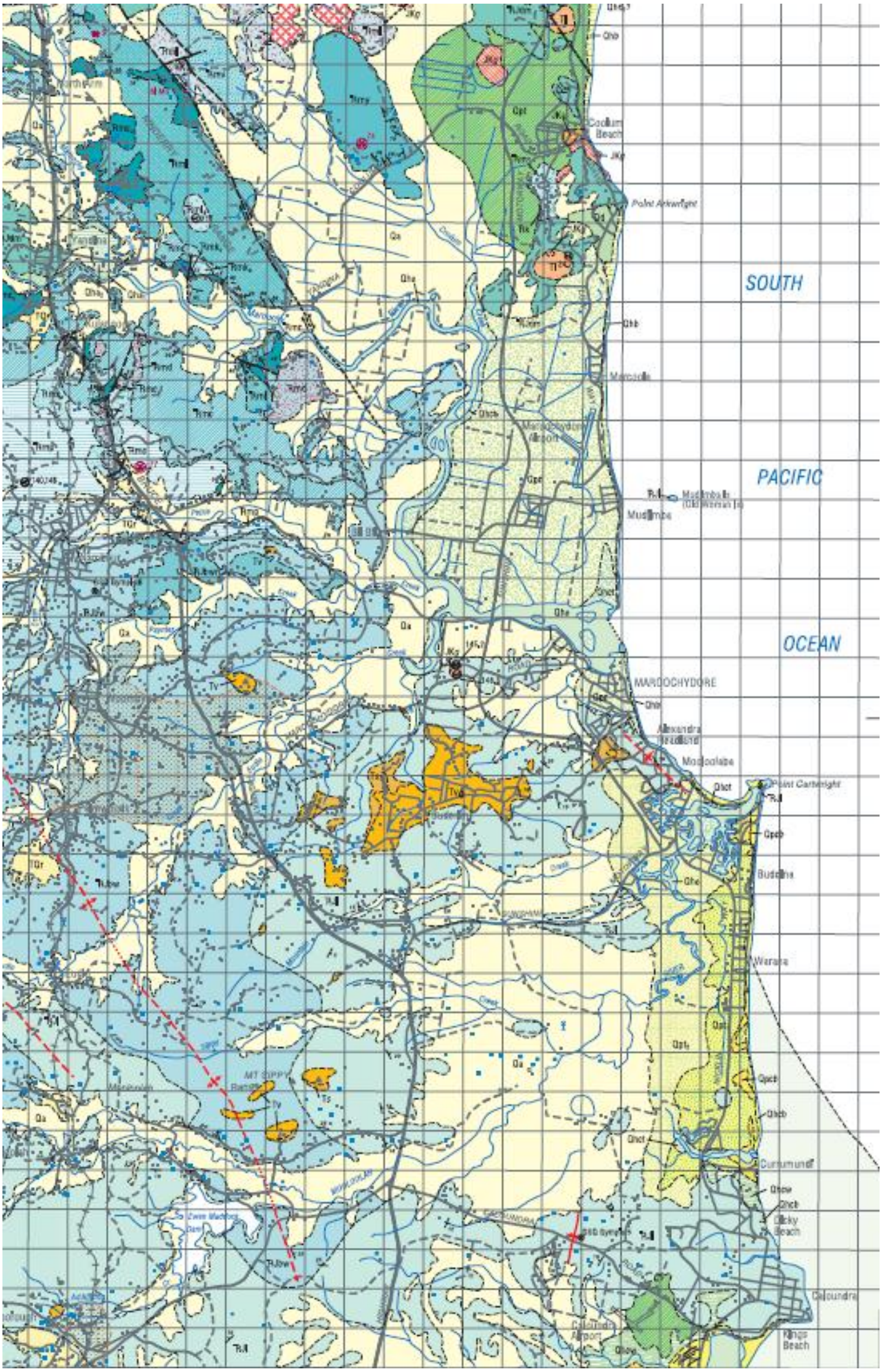


Figure 8: Southern Region Geological Properties - Department of Mines and Energy, 1999

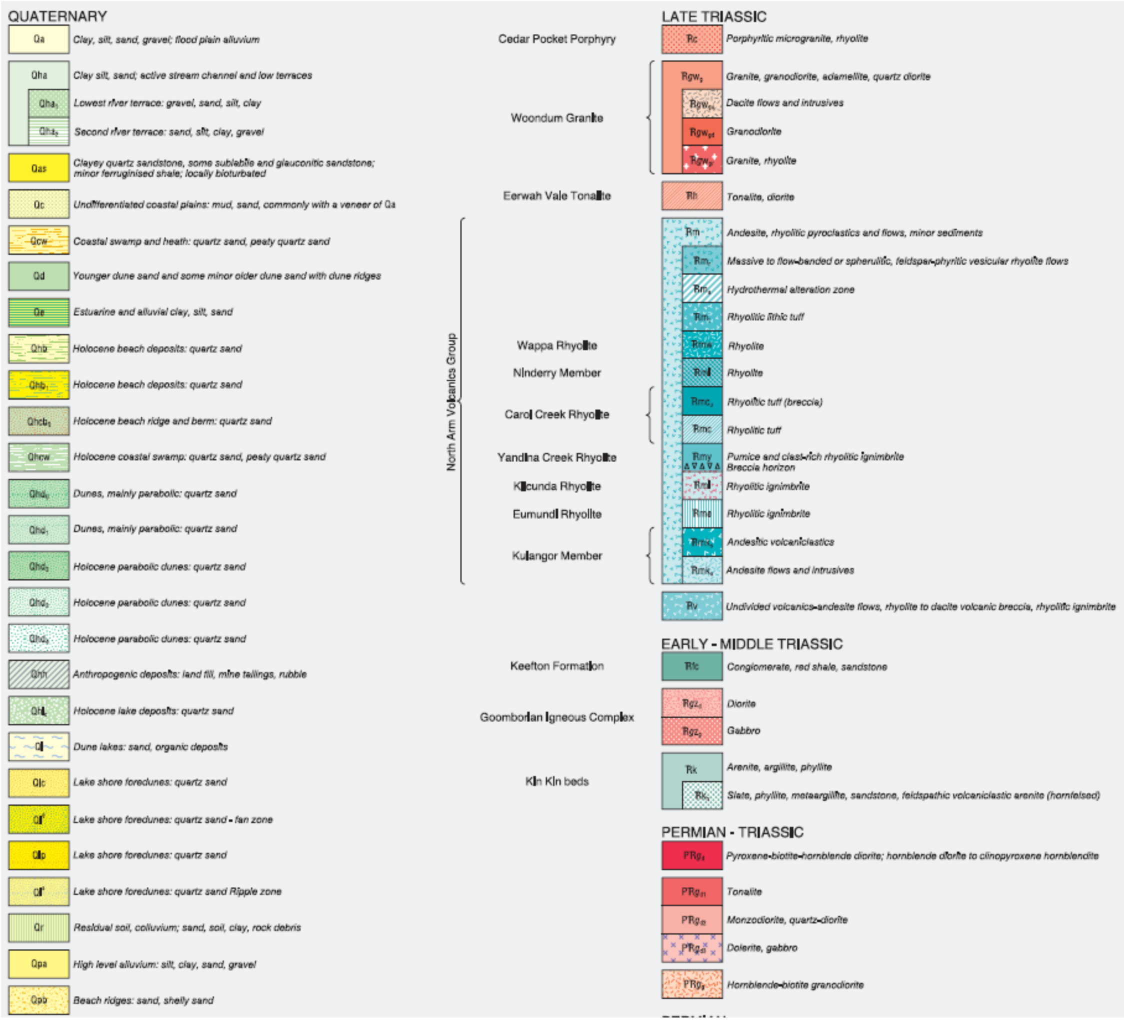


Figure 9: Geological Properties Legend –Department of Mines and Energy, 1999

The Sunshine Coast region is experiencing rapid growth and the increasing pressure of closer settlement is leading to further development within the region. Areas previously deemed unfavourable for construction are now being developed, presenting challenges for both construction and maintenance of the required infrastructure. Many of these sites are being constructed on soft estuarine deposits with highly expansive or collapsible soils.

3.3 Expansive and Collapsible Soils

3.3.1 Expansive Soils

Expansive soils are common throughout Australia and evident in the dark grey to black prairie soils, chocolate soils and black earths such as montmorillonite, located within the Sunshine Coast region. Expansive soils are defined as soils that change in volume in relation to variable water content. Commonly referred to as the shrink and swell behaviour. The more water they absorb the more their volume increases, for the most expansive clays expansion of 10% is not uncommon (Chen et al. et al., 1988).

The amount by which the ground can shrink and/or swell is determined by the water content in the near-surface zone; significant activity usually occurs to about 3m depth, unless this zone is extended by the presence of tree roots (Driscoll and Chown, 2001). Fine-grained clay-rich soils can absorb large quantities of water, swelling after rainfall and alternatively becoming very hard when dry, resulting in cracking of the surface. Holtz and Kovacs state that the swelling and shrinkage process is not fully reversible. The process of shrinkage causes cracks, which on re-wetting, do not close-up and also promote further water ingress, consequently, further expansion.

The expansiveness of the soil is influenced by a variety of factors including seasonal climatic conditions, or local environmental changes such as leaking stormwater pipes or water utilities, changes to surface drainage (development including road construction, concreting works), clearing and removal of vegetation, decreasing the absorption of water from the soil.

Expansive soil problems typically occur due to water content changes in the upper few metres, with deep seated heave being rare (Nelson and Miller, 1992). Climatic and environmental factors significantly influence the water content in the upper layers which are termed seasonal fluctuations or active depth. The active depth is the depth to which water content has increased due to the introduction of water from external sources (Jones and Jefferson, 2012). It is important to determine the active depth during site investigation.

Jones and Jefferson (2012) suggest the structures most susceptible to damage caused by expansive soils are usually lightweight in construction. Houses, pavements and shallow services are vulnerable to damage because they are less able to suppress differential movements than heaver multi-story structures.

Chen et al. (1998) undertook a series of case study examples of foundations and problems that arise when dealing with expansive soils. Factors which affect road pavements on expansive soils were outline as:

- Changes in water content
  - High water tables
  - Poor drainage under pavement layers
  - Water ingress from external sources
- Poor Construction practices
- Lack of appreciation of soil profile
  - Underlying geology contains inclined bedding of bedrock causing swell to be both vertical and horizontal
  - Uncontrolled fill placement
  - Areas of extensive depth of expansive soils.

Pavements are particularly vulnerable to expansive soil damage with estimates suggesting that approximately half of the overall costs from expansive soils are associated with pavements (Chen et al., 1988). The vulnerability of road pavements is due to their relatively light weight, extensive area and repetitive uneven loading. Pavement design can be treated similarly to foundation design. However, different approaches are required as it is impractical to make pavements stiff enough to avoid differential movements and can be more economical to treat subgrade soils. A number of approaches should be considered:

1. Choose an alternative route and avoid expansive soil;
2. Remove and replace expansive soil with a non-expansive alternative
3. Design for low strength and allow regular maintenance
4. Physically alter expansive soils through disturbance and re-compaction
5. Stabilisation through chemical additives, such as lime
6. Control water content changes although very difficult over the life of a pavement. Techniques include: pre-wetting, membranes, deep drains, slurry injection

(Jones, 2012)

Expansive soils have the potential to undergo large volumetric changes in response to variable water content. As mentioned, this can be caused by water ingress through the surface of pavements, externally contributed from neighbouring utility conduits, leaking stormwater and sewage systems; and can be affected by the reduction of adjacent vegetation. Expansive soils present significant challenges for pavement and foundation construction throughout the world, it is necessary to understand expansive soils to successfully engineer structures in an effective way to account for its potentially damaging behaviour.

3.3.2 Collapsible Soils

Subgrade materials comprised of soils that change volume upon wetting have caused distress to pavements since the beginning of professional practice and have cost many millions of dollars in roadway repairs (Houston, 1988). The alluvial sediments present on the Sunshine Coast are considered collapsible soils. Numerous soil types can fall in the general category of collapsible soils, including Aeolian deposits, alluvial deposits, colluvial deposits, residual deposits, and volcanic tuff (Howayek et al., 2011). Collapsible soils are characterized by very distinct geotechnical properties that include high void ratio, low initial bulk density and water content, great dry strength and stiffness, high percentage of fine grained particles and zero or slight plasticity. In most cases they contain over 60% of fines and have a porosity of 50% to 60%, liquid limit of about 25 and plastic limit ranging from 0 to 10 (Howayek et al., 2011). Collapsible soils are unsaturated soils that exhibit large decreases in strength as moisture contents approach saturation, resulting in collapse of the soil skeleton and large decreases in soil volume. Volume changes may or may not be the result of the application of additional loading. The amount of volume change that occurs depends on the soil type, structure, the initial soil density, the imposed stress state, and the degree and extent of wetting (Houston, 1988). Figure 10 represents a schematic view of key characteristics of collapsible soils.

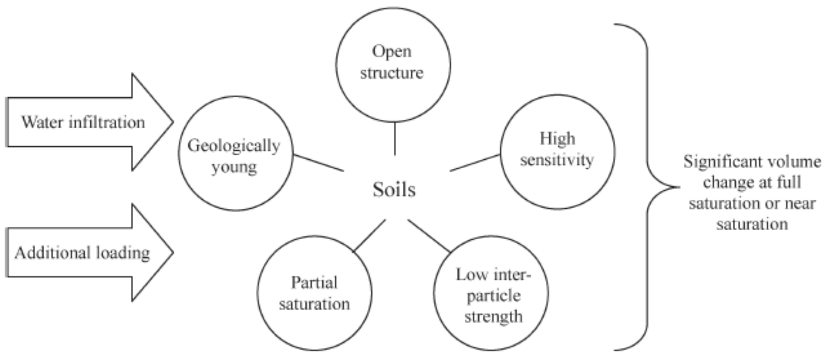


Figure 10: Schematic view of key characteristics of collapsible soils (Howayek et al., 2011)

Many collapsible soils may be residual soils that are products of weathering of parent rocks. The weathering process produces soils with a large range of particle size distribution. Soluble and colloidal materials are leached out be weathering, resulting in large void ratios and consequently unstable structures. Collapsible soil deposits are also common results of flash floods and mud flows. These deposits dry out and are poorly consolidated. As the soil dries by evaporation, capillary tension causes the remaining water to withdraw into the soil grain interfaces, bringing with it soluble slats, clay, and silt particles. As the soil continue to dry, these salts, clays and silts come out of solution, and “tack-weld” the larger grains together (Houston, 1988). Houston (1988) also suggests this leads to a soil structure that has a high apparent strength at its low, natural water content. However, collapse of the structure occurs upon wetting as the soils become unstable at any stress level which exceeds that at which the soil had been previously wetted. Therefore, in some locations when water exceeds natural content, collapse can occur at relatively low levels of stress. Additional traffic loading adds to the collapse potential. The critical component which triggers collapse however, is water.

Collapsible subgrade soils can have a seriously detrimental effect on pavement performance. This is affected by the differential settlement across road sections. Differential collapse settlement across roadway sections comes from two major factors: non-homogenous subgrades that encompass materials with different degree of collapse potential, and non-uniform distribution of wetting in subgrades materials. Often the latter can be originated by upward ‘‘pumping’’ of the water as a result of traffic loading (Howayek et al., 2011). Howayek et al. suggests that differential settlements cause rough and bumpy surfaces which reduce serviceability, raise the frequency and the cost of pavement rehabilitation.

In most cases, various projects will have unique design considerations, economic restraints, and differential risk factors which need to accounted for. Houston (1988) outlines the best design outcome relative to the subgrade soils may consist of the following techniques:

- In-situ treatments with additives such as lime, cement or fly-ash;
- Seepage barriers and/or drainage systems;
- Computing of the serviceability loss and a modification of the design to “accept” the anticipated expansion

Some techniques for identifying collapsible soil problems include, qualitative index tests conducted on disturbed samples, wetting tests on relatively un-disturbed samples and in-situ measurement techniques. Most methods for identifying collapsible soils are only qualitative in nature, providing no information on the magnitude of the collapse strain potential (Houston, 1988). Qualitative methods include various functions of dry density, moisture content, void ratio, specific gravity, and Atterberg Limits.

Houston (1988) suggested in-situ methods had positive results in some cases, as researchers believed that sample disturbance was greatly reduced, and that a more quantitative measure of the collapse potential was achieved. In-situ test methods suffered due to the unknown extent and degree of wetting during field testing. The zone of material influenced was generally inconclusive, therefore, actual strains induced by the addition of water were not well known. Therefore, research suggests the most reliable method for identifying collapse potential of a soil was to obtain the best quality undisturbed sample possible and subject this sample to laboratory wetting. Houston (1988) found that the results of a simple oedometer test indicated the collapse potential and at the same time gave a direct measure of the amount of collapse strain potential that may occur in the field. The greatest source of error is predicting the extent of wetting that might occur in the field.

It is recommended that to best estimate the amount of settlement expected in the field, in-situ wetting must be estimated and soil samples must be subjected to wetting tests in the laboratory. Settlement is then estimated using the strains observed in these tests. Houston (1988) recommends that if collapse settlements are expected to be quite large, mitigation measures may be taken. Several mitigation measures were extensively studied in a large-scale field test conducted by the New Mexico State Highway Department. The methods evaluated included:

- Sub-excavation
- Flooding the area with water
- Ponding combined with reversed sand drains
- Vibrofloatation; and
- Dynamic compaction.

(Lovelace, Bennet and Lueck, 1982)

Rollins (1994) undertook an investigation into the effectiveness of treatment methods for collapsible soils. The evaluation was undertaken using six full-scale load tests performed on 1.5m square footings. Treatments included pre-wetting with water, partial replacement with compacted fill and various pre-wetting procedures with chemical additives and dynamic compaction under dry and wet conditions. Soil improvement was evaluated using double oedometer testing on "undisturbed" samples along with cone penetration test and pressure meter tests.

The conclusions were:

- Dynamic compaction treatment provided effective means of reducing settlement
- Pre-wetting in combination with dynamic compaction increased compaction efficiency and uniformity
- The use of partial excavation and replacement methods prevent settlement for small volumes of water but continued percolation would eventually lead to excessive settlement
- Pre-wetting with water was the easiest and least costly treatment, however, it must be accompanied by preloading, surcharging or over excavation to be effective
- Creep settlement was significant of all treatment methods; and
- Accurate estimations of the performance of collapsible soils were difficult due to the problems associated with obtaining undisturbed samples and variability of alluvial soils.

When there is a high potential for the soil to collapse, further economic comparisons should be undertaken. Considerations include the cost of repairing future pavement failures versus the cost of undertaking initial mitigation measures. Economic constraints determine which mitigation measures may be suitable. Houston (1988) suggests some versions of pre-wetting techniques will usually provide the maximum site improvement per dollar spend on mitigation.

3.4 Current Test Methods

The selection of a pavement rehabilitation strategy depends to a large extent on the evaluation of the pavement structural capacity condition and roughness (Uzan and Lytton, 1989). Testing of road pavements and subgrades aim to provide an understanding of the in-situ road pavement. Testing is undertaken to determine the cause of failure and to determine a suitable rehabilitation treatment. Testing is comprised of both destructive and non-destructive methods.

3.4.1 Non-Destructive Testing

Non-destructive testing is gaining more and more popularity among pavement engineers (Tung and Uzan, 2012). Non-destructive testing includes surface deflection testing and the use of Ground Penetrating Radar (GPR) methods. Surface deflection testing is the most common form of non-destructive testing used on road pavements. It is measured by means of Falling Weight Deflectometer (FWD) testing. FWD testing measures the structural response of a pavement when exposed to a defined load, providing an estimated pavement modulus and remaining life. Research into FWD testing has shown that results can be directly related to seasonal moisture factors and should be considered at the time of testing. In order to accurately determine the elastic modulus of materials, apart from the deflection data, the pavement profile of the tested structure is required (Tung and Uzan, 2012).

When using deflection testing, there may be a need to convert deflections using one method to the equivalent deflections using another method. Guidance on his matter is provided in *Pavement Strength in Network Analysis of Sealed Granular Roads: Basis for Austroads Guidelines* (Austroads, 2003a).

The pavement profile can be obtained from various sources including local knowledge of the construction history, as-constructed drawings and geotechnical investigations. However, geotechnical investigations consist of undertaking borehole investigations and is a destructive testing procedure. Depending on the quantity of boreholes conducted, a true representation of the pavement profile may not be obtained. Considering the requirement to examine the existing pavement profile in a non-destructive and consistent way, the use of Ground Penetrating Radar (GPR) techniques were introduced. Tung and Uzan (2012) advised that GPR technology is used to determine a continuous pavement profile and material characteristics.

GPR is a highly versatile non-destructive method which provides a range of condition and construction pavement information (Tung and Uzan, 2012). GPR records the time taken by emitted radio frequencies to travel between the electrical boundaries of the pavement layers. The travel time is used to determine the depth of material interfaces within the pavement structure, which in turn is used to calculate the thickness of individual pavement layers (Tung and Uzan, 2012).

GPR testing enables back-analysis of Falling Weight Deflectometer testing to determine the elastic modulus of the different materials in a non-destructive manner. While non-destructive testing can provide reasonable results, the importance of invasive pavement testing methods cannot be overlooked. Combining GPR testing with a targeted borehole investigation will increase confidence within the pavement profile results. Once validated, GPR technology can be used to reduce the quantity required and minimise the possibility of boreholes being undertaken in unsuitable locations.

Additional non-destructive test methods include visual pavement condition surveys undertaken by human audit or more recently via laser condition survey. This form of testing predominantly examines roughness and rutting data for future pavement life cycle modelling. This is used predominantly to predict resurfacing treatments and timeliness for optimal outcomes.

Non-destructive testing is widely becoming the preferred testing method among pavement engineers and road asset managers. Road transport is gaining popularity and the increased amount of traffic is highlighting the importance of continual improvement and maintenance of road networks. To ensure this, non-destructive testing methods provide a rapid and cost effective alternative for monitoring and testing or road networks. Pavement rehabilitation designs require an accurate and comprehensive assessment to determine the cause of failure. Destructive testing forms an important part of assessing these pavements and are required to determine an effective treatment.

3.4.2 Destructive Testing

Mooney et al. (2000) has stated that destructive testing is often necessary to determine the true cause of pavement failure, due to the limitations of non-destructive testing. Destructive testing must often be done using either trenching or coring to obtain samples. Subsurface profiles may be taken to see deformation of different layers, and to check that recorded layer thickness profiles are correct (Chen et al., 2003).

According to Mooney et al. (2000), trenching also provides a visual view of pavement layers, and an assessment can be made of the wetness of each layer, and any moisture at interfaces between them. Standardised testing methods available for use in Queensland are listed in the *Materials Testing Manual* (Queensland Department of Main Roads, 2002a). These tests are mostly empirical testing methods.

The general types of tests currently used include California Bearing Ratio (CBR), Dynamic Cone Penetrometer (DCP), Hydrometer testing and determination of Atterberg Limits.

3.4.2.1 California Bearing Ratio

The California Bearing Ratio (CBR) test is an empirical test used to determine the pavement subgrade strength. California Bearing Ratio is defined as the ratio of force required to cause a circular plunger of 1,932mm^2 area to penetrate the material to a specified distance expressed as a percentage of standard force (Queensland Department of Main Roads, 2002). Samples are either tested under soaked or unsoaked conditions. The method allows for the determination of CBR Maximum Dry Density (MDD) and CBR Optimum Moisture Content (OMC) as well as the optimal determination of swell and post penetration moisture content (Queensland Department of Main Roads, 2002). Moisture content can be varied to represent climatic conditions.

3.4.2.2 Dynamic Cone Penetrometer

Dynamic Cone Penetrometer (DCP) testing provides an in-situ strength measurement of materials, this method provides an indication of the subgrade resistance to penetration in its natural undisturbed state. DCP testing indicates the ability of a material to withstand loading before penetration into the surface occurs. If the DCP cone penetrates easily into the soil, it indicates that the material is low strength, further compaction and additional pavement layers may be required. DCP testing is conducted by driving a penetrometer into the subgrade by dropping a 9.07 kg weight onto a 16mm diameter vertical shaft and measuring the penetration depth against the blows, providing an in-situ CBR value.

3.4.2.3 Hydrometer Test

The Hydrometer testing method involves measuring the percentage of sand, silt and clay in the inorganic fraction of soil. To determine the grain size distribution for particles greater than 75um, sieving is used. For particles smaller than 75um, Hydrometer testing is used. Hydrometer testing uses Stoke’s equation (for the velocity of a free falling sphere in suspension) to determine grain size distribution. The sieve is placed in suspension and by the use of Stoke’s equation the equivalent particle size and percent of soil in suspension are computed.

Hydrometer testing is usually discontinued when the percentage of clay sized particles has been determined (Walters, 2008). To provide additional information on potential soil behaviour, further classification tests are undertaken. The most common type of further testing to further understand the mechanic behaviour of clay soils is the Atterberg Test method.

3.4.2.4 Atterberg Limits

Albert Atterberg proposed the limits (liquid limit LL, plastic limit PL and shrinkage limit SL) of consistency in an effort to classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability (Casagrande, 1958). The Atterberg limits are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state the consistency and behaviour of the soil is different and thus so are its engineering properties. Atterberg limits are used to distinguish between silt and clay, and it they can be used distinguish between various types of silts and clays. The behaviour of Atterberg limits with respect to moisture content are shown in Figure 11.

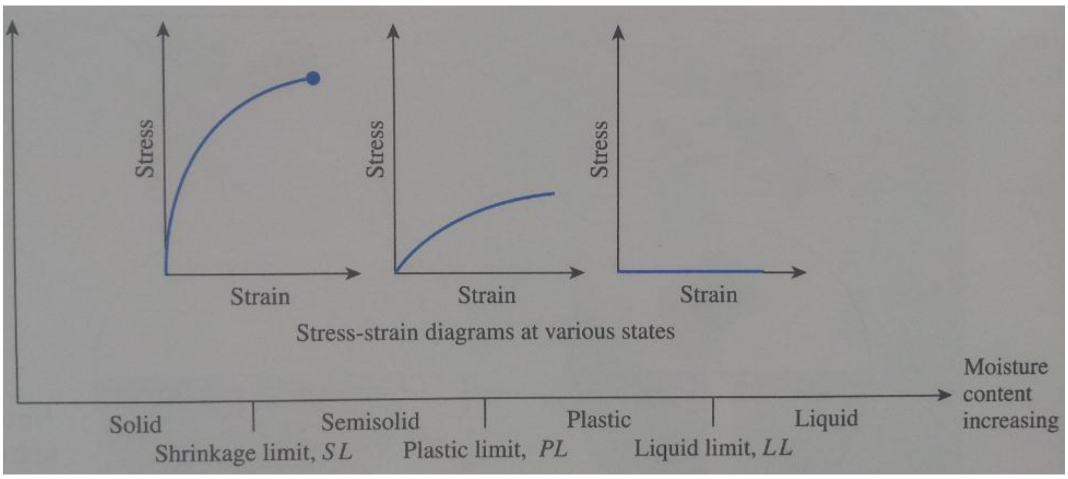


Figure 11: Atterberg Limits (Das, 2010)

Shrinkage limit:

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. The shrinkage limit is much less commonly used than the liquid limit and the plastic limit (State of New York Department of Transportation, 2007).

Shrinkage limit can be determined as:

$$SL = w_i - \Delta w_s$$

Where,

w_i is the initial moisture content

Δw_s is the change in moisture content

The shrinkage limit can be estimated by considering the volume and weight of the solids:

$$SL = m = \frac{\rho_w V}{w_s} - \frac{1}{G_s}$$

Where,

ρ_w is the density of water

G_s is the specific gravity

Plastic limit:

The State of New York Department of Transportation (2007) states that the plastic limit (PL) is the water content where soil starts to exhibit plastic behaviour. A thread of soil is at its plastic limit when it is rolled to a diameter of 3mm or begins to crumble. To improve consistency, a 3 mm diameter rod is often used to gauge the thickness of the thread when conducting the test.

Liquid Limit:

Liquid limit (LL or w_L) is defined as the arbitrary limit of water content at which the soil is just about to pass from the plastic state into the liquid state. At this limit, the soil possess a small value of shear strength, losing its ability to flow as a liquid. In other words, the liquid limit is the minimum moisture content at which the soil tends to flow as a liquid (State of New York Department of Transportation, 2007).

Plasticity Index:

The Plasticity Index (PI) is the range of water content within which the soil exhibits plastic properties; that is, it is the difference between the liquid and plastic limits.

$$PI = (LL - PL)$$

The PI is important in classifying fine-grained soils (Das, 2010). For proper evaluation of the plasticity properties of a soil, it has been found desirable to use both the liquid limit and the plasticity index values.

Shrinkage Index:

The shrinkage index (SI) is defined as the difference between the plastic and shrinkage of a soil, furthermore it is the range of water content within which a soils is in a semisolid state of consistency. Shrinkage index provides an indication of the change in volume expected in a given soil as its moisture content varies.

$$SI = (W_p - W_s)$$

Consistency Index:

The consistency index (CI) is defined as the ratio of the difference between the liquid limit and the natural water content to the plasticity index of a soil:

$$CI = (LL - w)/PI$$

Where,

w is the natural water content of the soil (undisturbed condition in the natural ground)

$If\ CI = 0, w = LL$;

$CI = 1, w = PL$;

$CI > 1$, the soil is in semi-solid state and is stiff;

$CI < 0$, the natural water content is greater than the LL, and the soil behaves like a liquid.

Liquidity Index:

The liquidity index (LI) is the ratio of the difference between the natural water content and the plastic limit to the plasticity index:

$$LI = \frac{w - PL}{PI}$$

$If\ LI = 0, w = PL;$

$LI = 1, w = LL;$

$LI > 1$, the soil is in liquid state;

$LI\ <\ 0$, the soil is in semi-solid state and is stiff.

Plasticity Chart

Casagrande (1958) studied the relationship of the plasticity index to the liquid limit of a wide variety of natural soils. On the basis of these test results, he proposed a plasticity chart as shown in Figure 12. The 'A-line' separates the in organic clays from the inorganic silts and the 'U-line' defines the upper limit of plastic clays (Das, 2010).

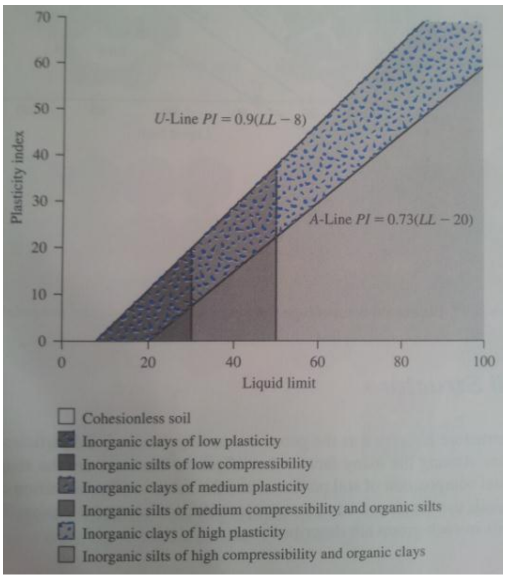


Figure 12: Plasticity Chart (Das, 2010)

The information provided in the plasticity chart is of great value and is the basis for the classification of fine-grained soils in the United Soil Classification System (Das, 2010).

3.5 Pavement Maintenance

Annual pavement works expenditure is around A $3 Billion, or nearly half of the total annual road expenditure with a significant percentage of these costs allocated to road pavement maintenance (Austroads, 2002a). Austroads states that roads are designed to varying standards and built from natural or processed materials to meet the needs of the communities they serve. Like all other structures they are subject to deterioration which commences as construction is completed. If the standard for which the pavement was designed is to be upheld, maintenance is required immediately after construction is complete. Most flexible pavements are expected to need some form of rehabilitation after approximately 20 years of trafficking (Walters, 2008). After which time they are typically suffering from forms of fatigue of deformation, and have unsuitable ride quality. This effects road user costs and safety.

Ideally, maintenance would ensure that the road always functions as efficiently as when first constructed, but in planning and maintenance, due regard must be paid to limitations of available labour, plant and funds (Austroads, 2008). Therefore, maintenance programs are modified to best control the rate of deterioration and ensure that the minimum service levels of the appropriate road authority are maintained.

3.5.1 Maintenance Strategies

The main objective for road authorities is to maintain their assets at an appropriate level of service (LOS) and structural integrity at the lowest possible cost (agency and user costs) without creating any significant adverse impacts on the environment, user safety and community activities (Austroads, 2008). Austroads also suggests that road maintenance activities relate to the repair of defects and attention to the road structure and associated facilities to ensure preservation of the asset and safety of its users. Maintenance is generally divided into routine maintenance, preventative maintenance and rehabilitation.

- Routine maintenance – includes the activities which address minor defects on the carriage way and structures. These works are usually unplanned and undertaken with minimal equipment and materials;
- Preventative maintenance – includes works that are intended to reduce further deterioration through timely surface interventions. Optimal preventative maintenance intervention times are often suggested by pavement management systems in conjunction with visual inspections and local knowledge;
- Pavement Rehabilitation – includes works that target roads whose ride quality has deteriorated below the acceptable levels of service. These works may also be undertaken due to insufficient structural capacity to cope with current or future traffic volumes.

3.5.2 Pavement Defects

This section of the report summarises typical defects and repair types which are performed by road authorities. It is essential to undertake efficient and effective pavement defect repairs to maintain the surface in a trafficable condition for the safety of road users and to reduce further deterioration and delay the requirement for pavement rehabilitation. It is important to understand defect types and their causes when considering future pavement rehabilitation designs.

Routine maintenance of road pavements can be considered under the following categories:

- Rutting
- Depressions

- Roughness
- Corrugations
- Cracking
- Shoulder Failure
- Potholes
- Shoving

Rutting

Rutting is the formation of longitudinal depressions of the wheel paths, most often due to consolidation or movement of material in either the base, subgrade or asphalt. It can be cause by a variety of means such as:

- The pavement is performing in accordance with the original design assumptions;
- The design traffic has been exceeded;
- The effective subgrade strength is less than the design strength adopted in the original design; or
- The in situ condition of the subgrade is different from the design condition adopted (e.g. moisture content is higher); or
- The pavement has suffered from one or more overloads.

(TMR, 2012)

The pavement is viewed to be experiencing severe failure and reaching the end of its design life when the pavement exhibits a rut of 25mm depth at the surface (Austroads, 2007a). The pavement is not considered a failure until the 25mm threshold is reached.

In addition to its effect on serviceability, deformation in base layers may lead to a reduction in the effective pavement thickness and, if left untreated, to the premature development of deformation in the subgrade (TMR, 2012). TMR (2012) suggests that this deformation may progress to shoving if the rutting becomes so severe that surface cracking occurs and allows water penetration into the underlying layers and subgrade.

Depressions

Depressions usually occur in road pavement surfaces when fill or backfill material has been inadequately compacted, commonly encountered at utility trenches and bridge abutments. Depressions caused by inadequate compaction of the fill may continue to increase in size and depth through consolidation which may require deep seated correction (Austroads, 2009).

Roughness

Pavement roughness is the measure of surface irregularities with wavelengths between 0.5 meters and 50 meters in the longitudinal profiles of either or both wheel paths in the traffic lane (Austroads, 2007a). It is one of the most reported measurements as it directly contributes to road user comfort and operating costs. It increases wear and tear on vehicle parts and the handling of the vehicle.

Pavement roughness can also be used as an indicator for pavement distress. It can often indicate surface distress of pavement materials or subgrade strength, or a combination of both. Currently most road authorities measure roughness in terms of the International Roughness Index (IRI). For network analysis IRI is generally obtained by measuring the road profile and processing this profile through an algorithm that simulates how a reference vehicle would responds to the roughness and summing the suspension travel (Gillespie, 2014). Austroads have endorsed the use of IRI for the representation of roughness in Australia.

Corrugations

Corrugations are transverse undulations in the road pavement structure, generally found on unsealed roads and rural bitumen seal surfaces but can occur in asphalt surfaces.

They are most commonly caused due to inadequate material quality, resulting in the inability to withstand traffic loading. Defective work practices such as irregular compaction can cause corrugations along with insufficient bonding between wearing surface and base materials. Figure 13 shows an example of typical corrugations in a road surface.

Figure 13: Defect Type: Corrugations (Austroads, 2009)

Cracking

Cracking as a road condition parameter is the measure of cracks appearing on the road surface. A crack is an unplanned break in the pavement surface (Austroads, 2010). Cracking of a road pavement can be in a variety of different classifications (Austroads 2009):

- Block Cracking – interconnected cracks forming a series of blocks approximately rectangular in shape, typically distributed over a large area of pavement;

  Causes:

  - Reflection of subsurface joints;

- Shrinkage or fatigue or underlying pavement (generally cemented);
- Inadequate slab thickness; and
- Ageing and hardening of asphalt surfacing.

Treatments:

- Crack filling;
- SAMI seals;
- Geotextiles;
- Milling and overlay; and
- In situ asphalt recycling.

- Crocodile Cracking – interconnecting cracks forming a series of polygons, resembling a crocodile skin. Crocodile cracking generally suggests that the asphalt surfacing has reached the end of its serviceable life.

  Causes:

  - Fatigue;
  - Inadequate pavement thickness;
  - Moisture in pavement;
  - Inadequate pavement quality; and
  - Lack of compaction in asphalt or cementitious layers.

  Treatments:

  - SAMI seals;
  - Milling and overlay;
  - In situ asphalt recycling;
  - Drainage improvements;
  - In situ stabilisation;
  - Heavy patching; and
  - Reconstruction / rehabilitation.
- Longitudinal Cracking – runs longitudinally along the pavement, is often the first type of cracking initiated in a wheel path or rut.

  Causes:

  - Reflection of shrinkage cracks in underlying materials;

- Poorly constructed joint (e.g. widening);
- Volume change of expansive clays;
- Differential settlement; and
- Reflection of cracks in underlying cemented base.

Treatments:

- Drainage improvements;
- Sealing shoulders;
- Crack filling;
- Milling and overlay;
- Heavy patching; and
- Reconstruction / rehabilitation.

- Transverse Cracking – unconnected crack running across the pavement:

  Causes:

  - Reflection of shrinkage crack or joint underlying surface;
  - Construction joint or crack in asphalt surfacing;
  - Structural failure of cement concrete base;
  - Shrinkage of slab during curing;
  - Settlement associated with utility trenching or a structure; and
  - Intrusion of tree roots into the pavement structure.

  Treatments:

  - Crack sealing;
  - SAMI seals;
  - Milling and overlay; and
  - In situ asphalt recycling.

These various classifications of cracking defects are shown in Figure 14.

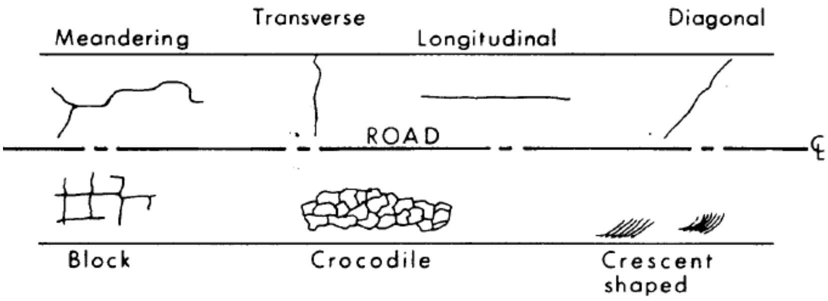


Figure 14: Classification of Cracking Defects (Austroads, 2009)

Shoulder Failure

Shoulder failure occurs along the unsupported edge of the pavement profile, where the unsealed shoulder is lower than the level of the adjacent surface. Failure is often due to weakened pavement material due to a number of factors including:

- Inadequate road alignment, encouraging traffic on the shoulders;
- Omission of shoulder reinstatement after overlay;
- Moisture ingress from poorly maintained drainage; and
- Growth of vegetation at the edge of the seal.

Treatments include re-sheeting, sealing, stabilisation or local pavement widening, depending on the cause of failure. Timely maintenance is required to minimise the damage to the trafficable pavement structure.

Potholes and Patching

The Queensland Department of Main Roads (2012) suggests that potholes provide a dramatic indication of pavement failure. Failure can be structural in nature, related to the surfacing or a combination of both. Alternatively, patches are an indication of pavement or subgrade failures and can provide an insight into what issues are likely in the future. Potholes can be described as steep-sided or bowl-shaped cavities extending into the layers below the wearing course (Austroads, 2009). Likely causes of potholing are:

- Loss of wearing surface material;
- Load accelerated deteriorating;
- Moisture ingress into the road pavement; and
- Poor quality construction materials.

Rectification works are usually undertaken as routine maintenance by road authorities. Road pavements which continually develop potholes require further treatment such as resealing or asphalt overlaying.

Patches are repaired sections of pavements which represent a loss of serviceability or structural capacity. Reconstructed patches are generally permanent and are usually square or rectangular in shape. Patches may contribute to increased road roughness and further distress (Austroads, 2009). Additional joins in the pavement surfaced cause by patching provides areas of weakness, promotes water ingress and can cause differential settlement. Road authorities commonly suggest crack sealing the edges of patching work to limit these defects. Common causes of pavement failures which require patching include:

- Surface deficiencies (rutting, cracking, ravelling, shoulder failure and stripping);
- Pavement deficiencies;
- Subgrade failure;
- Inadequate compaction; and
- Change in subgrade conditions (e.g. rise in moisture content).

Patching generally does not require any further treatment other than crack sealing the edges. If further action is required within a short period of time it suggests the possible rehabilitation of the pavement. Consideration should be given to the reasons for the patching and a resurfacing appropriate to that type of defect (Austroads, 2009).

Shoving

Shoving is the bulging and horizontal deformation of the road surfacing, usually occurring in areas of high shear stress. Deformations that are usually shallow are not likely to be confused with larger depressions or pavement distress resulting from weaknesses in the pavement or the subgrade (Austroads, 2009). Austroads (2009) suggests common causes of shoving are:

- Lack of containment at the pavement edge combined with selling of moisture-susceptible pavement material;
- Inadequate pavement thickness:
- Poor quality construction materials;
- Inadequate compaction of asphalt wearing surface or base material;
- Localised softening of asphalt binder due to fuel/oil spillage;
- Excess bitumen binder content in asphalt;
- Lack of adhesion between pavement layers; and
- Moisture in pavement and/or subgrade.

Treatments include:

- Milling and replacement with adequate material;
- In situ asphalt recycling;
- Drainage improvements;
- Heavy patching;
- In situ stabilisation;
- Asphalt overlay; and
- Rehabilitation.

Shoving is typically represented by defects similar to those shown in Figure 15.

Figure 15: Typical Shoving Defect (Austroads, 2009)

3.5.3 Moisture in Road Pavements

Providing adequate drainage to a pavement system has been considered an important design consideration to ensure satisfactory performance of the pavement, particularly from the perspective of life cycle cost and serviceability (Agarwal, Rokade and Shrivastava, 2012). Excessive water content in the pavement structure can cause early distress and accelerate structural failure of the pavement. Lytton, Pufahl and Michalak (1993) states that water related damage can cause one or more of the following deteriorations:

- Reduction of subgrade and base/sub base strength;
- Differential swelling in expansive subgrade soils;
- Stripping of asphalt on flexible pavements; and
- Movement of fine particles into base or sub base course materials resulting in a reduction of the hydraulic conductivity.

Moisture can infiltrate pavement structure in a number of ways which will inevitable cause deterioration of the pavement structure. The moisture content has a major effect on the strength of unbound materials and subgrades which are heavily dependent on moisture content. Austroads (2009) implies that a knowledge of the sources of moisture ingress and the methods in which they enter the pavement structure is essential for adequate pavement and subsoil drainage design.

Moisture changes in pavements usually result from one or more of the following sources (Austroads, 2009):

- Seepage from verges, medians or higher ground;
- Capillary action or fluctuations in the height of the water table;
- Infiltration of water through the surface of the road pavement and shoulders;
- An abrupt, significant decrease in the relative permeability of the successive layers in the pavement causing saturation of the materials in the vicinity of the permeability reversal;
- The transfer of moisture, as a result of moisture content or temperature differences within or beneath the pavement; and
- The transfer of moisture due to osmotic pressure in the vicinity of the root structures of large vegetation.

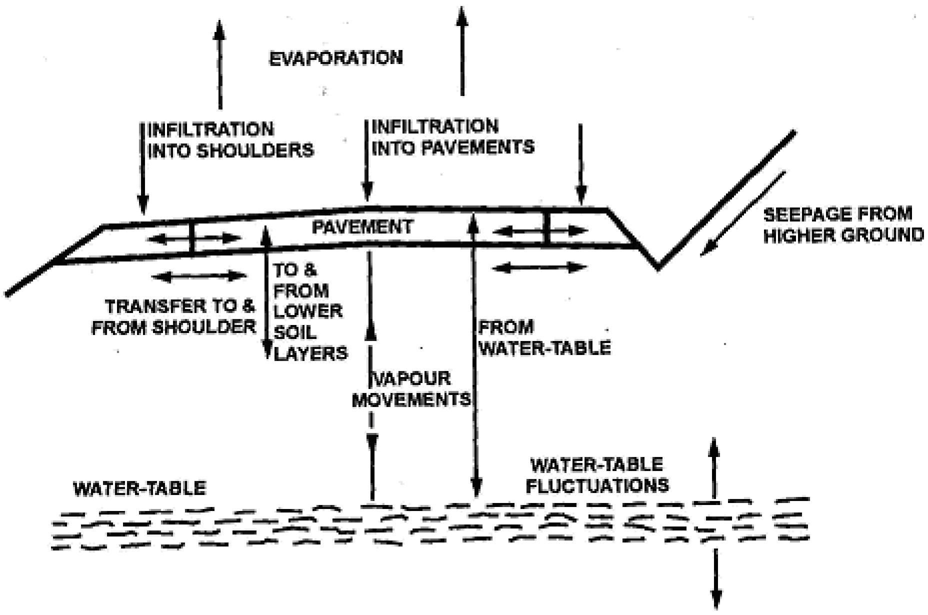


Figure 16: Sources of Moisture in Pavements (Waters, 2002)

Most of the above moisture infiltration sources, evident in Figure 16 can be controlled by three broad types of drainage systems:

- Surface drainage;
- Subsoil drainage; and
- Drainage blankets.

Surface drainage consists of crossfall, elevation and table drains, preventing moisture ingress into the pavement structure. Elevating the pavement from its surrounding materials, with a sloping surface to minimise water infiltration due to rainfall, is the most practiced form of surface drainage. Current practice is to achieve a minimum 2.5% crossfall on road pavement surfaces.

Austroads (2009) suggests that due to the possibility of water infiltrating a pavement structure from many sources, subsoil drains may be required to intercept, collect and then discharge water from beneath the pavement. Subsoil drainage systems are generally installed to either intercept water before it reaches the pavement structure or to remove water from the existing pavement structure. It is common to install subsoil drains in pavements prior to undertaking rehabilitation works, in an attempt to improve subgrade conditions and minimise unsuitable material. Drainage blankets consists of an introduced free-draining material to intercept subterranean water sources.

Excess moisture and particularly high degrees of saturation result in significant pore pressures within the material (Walters, 2008). This may produce premature failure of the pavement due to shear/bearing failure, rutting or lifting of wearing course due to positive pore pressures. Moisture in road pavements is often the primary cause of premature failure.

3.5.4 Subgrade Treatment Options

The subgrade is a portion of natural soil which the pavement or sub base is built upon. Subgrade support is critical in the design of a pavement structure. The quality of the subgrade will determine the pavement design and effect the useful life of the pavement. Subgrade performance depends on three basic characteristics, suggested by Ceylan, Schaefer and White, Schaefer and White (2008):

- Strength – it is essential that the subgrade be able to support loads transmitted from the pavement structure. The load-bear capacity is often affected by degree of compaction, moisture content and soil type. A CBR of 10 or greater is considered essential to support heavy repetitive loads without excessive deformation;
- Moisture Content – Moisture affects a number of properties including load bearing capacity, shrinkage and swelling. Moisture infiltration is possible in many ways as previously mentioned. Excessively wet subgrades will deform under loading; and
- Shrinkage and/or swelling – this occurs depending on their moisture content and generally leads to cracking of the pavement constructed over them.

Research has shown that with a subgrade strength of less than a CBR of 10, the sub base material will deflect under traffic loadings in the same manner as the subgrade (Ceylan, 2008). Basic knowledge of subgrade soils and their basic engineering properties is essential for pavement design. Achieving a high quality subgrade requires proper practices and quality control testing, however, the pavement design requirements and the level of engineering control should be consistent with the relative importance, scope and financial constraints of the project.

Soft subgrade and moisture sensitive soils such as collapsible, and expansive soils present construction challenges as well as life cycle pavement performance challenges. Ceylan, Schaefer and White, Schaefer and White (2008) stresses the important of proper treatment of problematic soils are important to ensure a long-lasting pavement structure that does not require excessive maintenance. Five techniques can be used to improve the strength and reduce climatic variation of pavement foundations on performance (Ceylan, Schaefer and White, Schaefer and White, 2008):

Stabilisation

Stabilisation is a subgrade treatment option considered for soils that are highly susceptible to volume and strength changes due to moisture variations and the subjected stress state. Subgrade soils can be treated with various chemical materials to improve the strength and stiffness characteristics of the soil. The stabilisation of soils is usually undertaken for the following reasons:

- To provide a construction foundation to dry very wet soils and enable compaction of upper pavement layers. This process generally excludes the stabilised soil as a structural layer in the design process; and
- To strengthen weak soils and minimise volume change potential of highly expansive or collapsible soils. This process usually forms part of the pavement design structure.

Additives used to control swelling and improve strength characteristics of unsuitable materials include lime, fly ash, cement and bitumen. *Queensland Department of Main Roads* (2012) explains the appropriate stabilising agent is a decision largely based on the material to be stabilised or modified. Table 1 provides a guide indicating suitability of stabilising agents for different soils.

Stabilising agent (or primary stabilising agent)	Material with particle size distribution with:					
	More than 25% passing the 75µm sieve and:			Less than 25% passing the 75µm sieve and:		
	PI ≤ 10	10< PI <20	PI ≥ 20	PI ≤ 6 and PI x % passing 75µm ≤ 60	PI ≤ 10	PI ≥ 10
Cement and cementitious blends	Usually suitable	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually unsuitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful		Usually suitable	Usually suitable	Usually unsuitable
Bitumen and cement blends	Usually suitable	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually unsuitable	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
*Polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually unsuitable
*Miscellaneous chemicals		Usually suitable	Usually suitable		Doubtful	Usually suitable

*Should be taken as a broad guideline only. Refer to trade literature for further information.

Note: The above forms of stabilisation may be used in combination (e.g. lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation).

Table 1: Suitability of Stabilising Agents for use with different soils (TMR, 2012)

Lime stabilisation improves the characteristic strength and chemical compositions of some soils. Ceylan, Schaefer and White, Schaefer and White (2008) explains that the strength of fine-grained soils can be improved significantly with lime stabilisation, while the strength of course grained soils is usually moderately improved. Lime stabilisation is most effective with highly expansive soils, such as the highly plastic montmorillonite. Lime treatment of subgrades is intended to facilitate construction loads and it is suggested that no reduction in the required pavement thickness should be made.

Cement stabilisation is the use of Portland cement for improving the engineering properties of low plasticity clays, sandy soils and granular materials. Cement stabilisation sufficiently increases the strength and stiffness of materials; and an increase in cement content generally increases the quality of the mixture. Higher cement content will invariably cause higher incidences of shrinkage cracking caused by the change in moisture content within the treated material. Ramanujam and Jones (2007) explain that the main disadvantage of subgrade cement stabilisation is the high stiffness created and a tendency for the overlying pavement to crack. Over recent years road makers have moved to an alternative slow setting cement that contains additives in order to improve workability, however, this has proven to cause greater stiffness than the original cement stabilization process leading to increased cracking problems.

Ceylan, Schaefer and White (2008) suggest that fly ash can be used in the stabilisation of clay soils as a substitute to lime and cement or in combination with lime and cement. As with lime and cement, the use of fly ash reduces the shrink-swell properties of the soils, generally used to dry soils for compaction. Considered for slay soils that are above optimum moisture content.

Bituminous stabilisation may be undertaken by foamed bitumen or bitumen emulsion stabilisation. Secondary stabilisation agents, usually cement or lime are added to increase the stiffness and strength of the material. Austroads (2009) defines foamed bitumen is a mixture of air, water and hot bitumen. Injection a small quantity of cold water into the hot bitumen produces expansion of the bitumen, forming foam. Bitumen in its foamed state increase particle bonding due to its large surface area. Austroads (2009) outlines bitumen emulsions as dispersions of fine droplets of bitumen in water, generally 60% bitumen and 40% water with a small portion of emulsifier. Setting and curing of emulsions involves the removal of water (breaking), leaving solid bitumen. Bitumen binders improve the bonding and cohesion between soil particles and usually improve the wet strength and water absorption resistance of the in situ materials.

Pre-wetting

Rogers and Rollings (1994) explains that pre-wetting has been routinely used to stabilise collapsible soils prior to construction in the past, however, it is only useful where the induced loads are small and recommends pre-wetting without preloading is not generally sufficient to prevent future foundation stress. Pre-wetting promotes the soil to settle under the existing overburden pressure and without preloading additional settlement may occur. Petry and Little (2002) state pre-wetting had become a proven method by the end of the 1970's and believe that ponding water on a foundation reduces the future swell initial, often assisted by moisture barrier installation. Pre-wetting of clay soils provided significant problems during construction. Saturated soils continually demonstrated the inability to support construction equipment and loading. Pre-wetting is usually not considered a viable option and the creation of a working platform through stabilisation or replacement is preferred.

Replacement

Das (2010) explains preliminary considerations for construction on expansive soils is the replacement of in situ materials with less expansive material. This is commonly practiced on the Sunshine Coast, however, with increasing traffic loadings, the required depth of pavement materials on poor subgrade materials is increasing. This presents possible conflicts with pre-existing utilities and infrastructure such as electricity, telecommunications, water, sewerage and gas. Where possible, replacement of unsuitable subgrade materials is still practiced. In recent times, replacement has been used in conjunction with various forms of geosynthetics to minimise the required excavation depth, minimise materials required and to avoid potential conflicts with underground services and infrastructure.

Geosynthetics

Ceylan, Schaefer and White (2008) explains that geosynthetics are a class of geomaterials that are used to improve soil conditions for a number of applications. Das (2006) believes that geosynthetics (including geofabrics, geotextiles, geomembranes and the like) play a role in separating materials, reinforcing, filtering, draining and/or providing a moisture barrier. The term "Geosynthetic" is used to cover a wide range of different materials including geotextiles, geogrids and geomembranes. Combinations of these materials in layered systems are usually called geocomposites (Ceylan, Schaefer and White, 2008). Significant savings can be made by replacing unsuitable materials with geosynthetics. Geosynthetics provide subgrade and pavement reinforcement by distributing the loadings on the pavements and providing lateral restraint.

A geotextile is defined by Ceylan, Schaefer and White (2008) as a permeable geosynthetic comprised solely of textiles. Geogrids consist of a regular grid of plastic with large apertures to provide interlocking potential of aggregates. Hence, the size of the aperture is dependent on the gradation of the material it is to be used with. Geogrids are manufactured using high density polymers. These polymers are then punched or weaved in one or two directions and the junctions between them are reinforced. Geomembranes are used to prevent fluid from penetrating the soil and as such consist of continuous sheets of low permeability materials (Ceylan, Schaefer and White, 2008). These materials are usually used for drainage purposes. Geocomposites are created by combining two or more geosynthetic products. Geocomposites are the most common form of geosynthetic used in road pavement construction on the Sunshine Coast. Applications for the various types of geosynthetic materials can be seen in Table 2 and Table 3.

Geosynthetic Materials	**Function**					
	Filtration	**Drainage**	**Separation**	**Reinforcement**	**Fluid Barrier**	**Protection**
Geotextile	x	x	x	x		x
Geogrid			x	x		
Geomembrane					x	
Geonet		x				
Geocomposites:						
Geosynthetic Clay liner					x	
Thin film Geotextile Composite					x	
Field coated Geotextile					x	

Table 2: Functions of Geosynthetics (Ceyan, 2008)

Function	Specific Use
Filtration	• Beneath aggregate subbase for paved and unpaved roads and airfields or railroad ballast
Drainage	• Drainage interceptor for horizontal flow • Drain beneath other geosynthetic systems
Separation (of dissimilar materials)	• Between subgrade and aggregate subbase in paved and unpaved roads and airfields • Between subgrade and ballast for railroads • Between old and new asphalt layers
Reinforcement (of weak materials)	• Over soft soils for unpaved roads, paved roads, airfield, railroads, construction foundations

Table 3: Transportation Uses of Geosynthetics (Ceyan, 2008)

In recent years, there has been a significant amount of research undertaken into the use of geosynthetics. Kwon and Tutumluer (2005) explain that the use of geosynthetics in unpaved roads and flexible pavement sections can lead to considerable improvements in pavement performance. A recent survey conducted among state highway agencies indicated that geosynthetics were more likely being used in the US for subgrade restraint rather than base reinforcement (Christopher, Berg and Perkins, 2001). Black and Holtz (1999) concluded their paper with a comment that subgrade sections beneath geotextiles become more consolidated with time than areas without the geotextiles. Research supports the use of geosynthetics for various purposes in road pavement construction and rehabilitation.

3.5.5 Pavement Rehabilitation Options

Pavement rehabilitation refers to the application of a treatment to an existing pavement experiencing distress, often due to fatigue. This section briefly describes the alternative rehabilitation treatments. Relationships between pavement defects and corresponding treatments are not presented in a prescriptive manner and engineering judgment is required when determining an appropriate treatment. The selection process is outlined by the *Queensland Department of Transport and Main Roads* (2012) is as follows:

1. Designer identifies the purpose of pavement rehabilitation;

2. Designer gathers available pertinent information and determines an appropriate approach;
3. Designer identifies existing pavement structure;
4. The designer evaluates all available information (historical and testing results) to determine condition;
5. The designer relates the condition obtained from the evaluation to the desired performance;
6. This range is narrowed by accounting for aspects such as project purpose, project constraints and relevant design and construction considerations;
7. Options are selected and designed;
8. Alternative rehabilitation strategies are compared, usually includes examining the whole of life costs of each option; and
9. Recommendations about which option should be selected.

Figure 17 represents this process graphically.

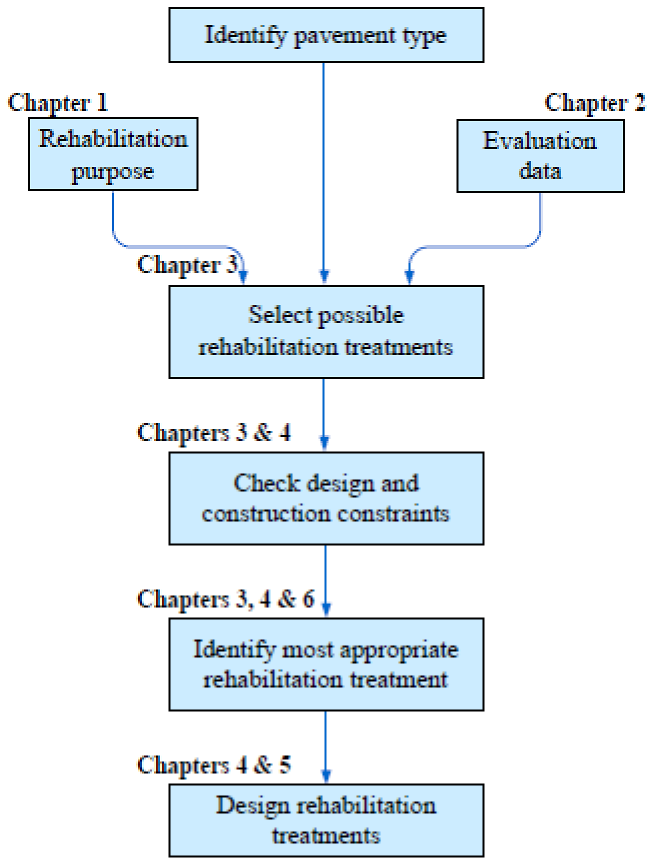


Figure 17: Selection of Rehabilitation Options (TMR, 2012)

The Pavement Design Manual (TMR, 2012) divides pavements into five basic types:

- Flexible pavements;
- Full depth asphalt pavements;
- Deep strength asphalt pavements;
- Flexible composite pavements; and
- Rigid pavements.

Pavement rehabilitation generally fall into the abovementioned categories, albeit varied to suit rehabilitation rather than new construction. Identifying a rehabilitation treatment is difficult and the type of failure needs to be investigated, under these circumstances evaluation and design tends to be site specific and more difficult.

Design and construction considerations of pavement rehabilitation works include the effect on the public, road geometry, drainage, pavement surfacing requirements, construction requirements, risk, availability of resources and financial implications. Austroads (2009) divides specific rehabilitation treatments into drainage systems, flexible pavement treatments and treatments for rigid pavements. Typical treatments include:

- Surface Treatments (asphalt overlays, bitumen seals, rejuvenation);
- Geotextile reinforced sprayed seals;
- Geogrids for reinforcement and reflective crack reduction;
- Milling and filling of irregular pavement surfaces and commonly replaced with asphalt;
- In situ asphalt recycling;
- Heavy Patching;
- Granular overlay options where existing infrastructure doesn't restrict level control;
- In situ stabilisation of granular pavements with chemical additives (lime, cement and bitumen);
- Crack/Joint sealing; and
- Full depth concrete patching.

Queensland Department of Transport and Main Roads (2012) and *Austroads* (2009) acknowledge that often more than one option is a viable solution. In such instances several options need to be considered. Generally this comparison considerers the following:

- Availability of resources and industry experience;
- Financial considerations or constraints;
- Technical aspects of each design option;
- Consequential effects; and
- Economic comparison.

Road authorities have a responsibility to thoroughly investigate the above options and consider the alternatives in a holistic approach, considering whole of life costs, environmental impacts, effect on road users and financial constraints, not only for individual projects, generally for an entire road network.

3.6 Conclusion

In conclusion, this research was undertaken using online resources including journal articles; published reports, dissertations and; Australian and International design standards. A review of available geological literature for the Sunshine Coast was vital in understanding the current subgrade challenges presented within the region. This review also included an investigation into current testing practices to gain a better understanding of soil properties and how they affect pavement design methods. World best pavement maintenance and rehabilitation practices were researched to outline current technologies and practices available. This information will provide direction on how to complete this research project.

4.0 Research Design and Methodology

4.1 Aim and Objectives

This project seeks to critically evaluate current pavement rehabilitation practices used within the Sunshine Coast region and to propose alternative practices. This will be achieved by researching current Sunshine Coast Council, Australian and International pavement design methods and practices. Research includes the geological and environmental history of the Sunshine Coast region and analyses the effectiveness of current pavement rehabilitation methods within the Sunshine Coast. Results from this research will be used to propose improvements to Sunshine Coast pavement rehabilitation practices.

This project aims to focus on the pavement rehabilitation options available, while considering the constructability of each design, whole of life costs and basic asset management principals applicable to maintaining a road network of this size. This will be achieved by incorporating findings by investigating pavement failure mechanisms, test methods, design, rehabilitation constructability and associated costs.

4.2 Consequential Effects/Implications/Ethics

4.2.1 Sustainability and Environmental Effects

Engineers Australia (2014) has produced a sustainability charter which outlines the need for sustainable development that meets the needs of the present without compromising the ability of future generations to meet their own needs. The objectives of this charter are presented below with relevant commentary.

- Development should enhance individual and collective well-being while maintaining the viability of the planet.

There are minimal environmental impacts of this project. Some soil sampling was undertaken as part of this project, however, the basis of this study is historical data already collected and available for investigation. Any samples previously collected were taken from pavements where pavement rehabilitation works were programmed. FWD and laser survey testing is considered non-destructive and relatively economical in comparison to destructive testing methods. The main effect on the environment throughout these tests is emissions from the vehicles used to undertake the testing. All testing methods used are an integral part in establishing a suitable and holistic pavement rehabilitation design, attempting to maximise the sustainability of design options.

- Development should ensure equity within the present generation as well as for future generations.

  This project aims to minimise the effects of expansive and collapsible subgrade soils; and therefore reduce the level of ongoing maintenance of roads constructed on undesirable subgrades. Consequentially this will decrease the use of virgin materials and increase the sustainability of the environment. As part of this research, pavement rehabilitation methods which incorporate the use of in-situ materials have been investigated to promote less demand on the environment e.g. stabilisation, recycled asphalt etc. The use of geosynthetics also limit the need for additional excavation and in turn reduces the required pavement structure thickness due to lateral reinforcement and load distribution, minimise the effects of traffic loading on the subgrade. The utilisation of the outcomes of this project throughout the region could have varying consequences on sustainability.

- Development issues and problems should be solved holistically and proactively.

  The aim of this project is to consider the most effective pavement rehabilitation options for the Sunshine Coast region. It is intended to investigate the incorporation of in-situ materials where possible and understand asset management principles which fundamentally prioritise roads requiring treatment within the road authorities' network. This approach concentrates on effective service levels, effective treatments considering whole of life costs, environmental impacts, effect on road users and financial constraints, not only for individual projects, generally for an entire road network.

Engineers Australia (2014) reiterates the importance of sustainable development. It touches on the requirement for fundamental change in the way that resources are used and in the way decisions are made. This project will attempt to minimise resources throughout the study, however, apply a holistic approach to any recommendations made.

4.2.2 Safety

All project works were undertaken in accordance with the Queensland Work Health and Safety Act (2011), supported by the Guide to Safety in the Civil Construction Industry (2000). These legislative documents provide the required practices to meet their obligations and minimise their exposure to risk, that all personnel are required to comply with. Throughout this project the applicable safety measures included:

- Site specific induction and relevant Personal Protective Equipment (PPE) for activities;
- Applicable PPE:
    - Eye protection;
    - Hand protection;
    - High visibility safety garments;
    - Protective footwear;
    - Protective headwear; and
    - Sun protection.
- Plant operation – awareness of plant movements, wear the applicable PPE in proximity to specific plant items and remain aware of exclusion zones.

- Site visits will be undertaken for chemical stabilisation road works. Material Safety Data Sheets (MSDS) to be present on site and additional PPE may be required as per MSDS.

4.2.3 Ethical

The Code of Ethics as published by Engineers Australia clearly defines the values and principles to which members commit to practice and reinforce the accountability for the code. The Code of Ethics provides the framework for Engineers to exercise their judgement when practicing for the common good. Engineers Australia expects that members of the engineering team will behave in a manner which merits the trust and respect of the public and the communities impacted by engineering activities (Engineers Australia, 2014).

Values, obligation of and rules of the conduct code are below with relevant commentary:

1. Public wellbeing, health and safety and sustainability, achieved by: maintaining the needs of the present while maintaining the ability for the future, promoting efficient and effective use of resources and safeguarding the wellbeing, health and safety of the public.

   Improved understanding of unsuitable subgrade soils and development of alternative rehabilitation treatments will benefit the community in terms or pavement performance. Lower maintenance costs and minimising the use of virgin materials maintains the ability for the future. Minimising pavement defects increases the safety of road users and the general public as road works will be minimised and road roughness will be improved. Throughout this research the impacts of particular actions and future designs were assessed to select an appropriate solution; and encourage environmentally sound and sustainable projects. Furthermore, the aim of this research is to promote the development of methods with less demand on non-renewable resources. Incorporating options which include the use of in situ pavement materials will minimise waste and encourage recycling of materials.

2. Responsible leadership which consists of acting lawfully, upholding the reputation of the engineering profession, promote the value of the profession to the public and to communicate effectively with all stakeholders.

   Throughout this project, all activities were conducted in a manner which upheld the values and reputation of the engineering professions. All stakeholders were communicated with effectively, treated with respect and courtesy without discrimination. The author ensured all results and actions throughout the project were fair, honest and in the best interests of the community, client, employers and colleagues. All works took into account accepted codes, engineering and environmental standards. The author attempted to provide clear and timely communications and ensure all information provided is relevant and in a readily understood form. Risk assessments identified no issues or consequential effects of this project. Environmental consequences are negligible as the majority of the project involves research of historical data.

3. Personal and professional integrity includes: acting with respect, avoiding perceived or actual conflicts of interest and seeking to eliminate fraudulent activity.

   During this project the author attempted to apply skills and knowledge with honesty, good faith and without personal bias. The reported recommendations are made in an objective and accurate manner. All work was practiced in accordance with statutory requirements and the commonly accepted standards at the time. Undertaking this project improved the author's knowledge, skills and experience in their chosen profession. The author only undertook work within their competency. This includes the area of road pavement design, construction and maintenance. Professional advice from colleagues with further understanding of the relevant topic will be required.

This research project satisfies ethical requirements under Engineers Australia's (2014) Code of Ethics. All work performed as part of this research will be conducted in accordance with the code and the author will ensure these standards are upheld.

4.3 Methodology

The methodology for this research is listed below with relevant commentary:

1. Research current Sunshine Coast Council, Australian and International pavement rehabilitation design methods.

   This research was undertaken using online resources including journal articles, published reports, dissertations and standard publications. Academic libraries were used to source international rehabilitation design methods. Current Australian rehabilitation methods were investigated through the review of Austroads and Queensland Department of Transport and Main Roads standards and published reports. Preliminary interviews were conducted with Sunshine Coast Council staff and local geologists to gain further understanding of current practices within the Sunshine Coast.

2. Research geological and environmental history of the Sunshine Coast region.

   Initially, the internet was used to determine the availability of applicable resources. As previously mentioned above, interviews were conducted with local geologists and geotechnical engineers to gain an understanding of the local geology and problematic soils within the area. Further information was sourced by contacting the Geological Society of Australia, Queensland Division who provided relevant articles and suggested literature for review. The geological history provided an indication of the types of soils within the Sunshine Coast area. The results of this geological research ensured concentration on applicable options and construction practices, relevant to the challenges of the region.

3. Collect soil test information for subgrade conditions within the Sunshine Coast. Evaluate the subgrade materials and their properties.

   The collection of soil test information was undertaken primarily through Sunshine Coast Council records. Further geotechnical testing sites are predetermined by Council's capital works program and this additional data will form part of this research if related to the project. All tests were conducted in accordance with Australian Standard AS1289. Interviews with external laboratory technicians to confirm specific test methods were not required. All test records are accompanied with detailed descriptions of their location and test method used.

4. Analyse the effectiveness of current pavement rehabilitation practices within the Sunshine Coast through the use of laser survey data, falling weight deflectometer testing and 'As Constructed' data.

   Through the review of Council's Pavement Management System, a range of pavement rehabilitation projects completed within the last 10 years were selected and a visual assessment completed. Data available from a recently completed laser survey was used to determine roughness, rutting, cracking and depressions of these projects and benchmarking of results was undertaken. Furthermore, Falling Weight Deflectometer (FWD) testing has been conducted on pavement rehabilitation projects completed within the last three years. These results were compared against traffic loadings, treatments and known subgrade conditions to ascertain which treatments have been more effective, given their current deflection and appropriate back calculation results. Project 'As Constructed' data and financials will be used to assess the feasibility of the various treatments.

5. Critically evaluate the effectiveness and performance of current Sunshine Coast Council pavement rehabilitation design practices against world's best practices.

   Further research into current pavement performance across the world provided an indication into which pavement treatments and construction practices which are producing greater results for subgrade conditions similar to the Sunshine

Coast. Current Sunshine Coast Council practices were compared with International recommendations and results. Technologies and assessment procedures as outlined in step four above.

6. Propose improvements to Sunshine Coast Council pavement rehabilitation design practices.

   Results from steps four and five formed recommendations for possible improvements. This was undertaken based on a holistic approach considering financial, environmental, ethical, construction and basic asset management principles. This objective is the culmination of the research undertaken in the preceding steps and the underlying discourse for this project. These recommendations attempt to minimise the effect of problematic soils on Council's road network.

7. Present results and recommendations in the required oral and written formats.

   Results and recommendations were presented using the guidelines provided by the University of Southern Queensland; and presented in oral and written formats. Written formats will be submitted in the form of a project proposal, project specification, preliminary report and final dissertation. Oral presentation were conducted on campus at the University of Southern Queensland early October 2014. The aim of this project was to assist the Sunshine Coast Council with the management of their road network and provide information for their Asset Management Department for future consideration. Furthermore, this research was undertaken to satisfy the requirements of the University of Southern Queensland's Engineering and Surveying program, assisting the author in obtaining critical skills and promoting professional development within the profession.

4.4 Testing and Evaluation Procedures

To critically evaluate the effectiveness of recent Sunshine Coast Council pavement rehabilitation projects Falling Weight Deflectometer (FWD) testing was undertaken on seven (7) projects completed within the last three (3) years as shown in Table 4 below.

Road Name	Suburb	Hierarchy	Constructed Date	Treatments	Subgrade CBR	Subgrade Accuracy
BUDERIM Street	CURRIMUNDI	SUB ARTERIAL	2012	FULL DEPTH ASPHALT	1.5	LABORATORY TEST
GANNAWARRA Street	CURRIMUNDI	COLLECTOR	2013	GRANULAR	7.5	LABORATORY TEST
LYON Street	MOFFAT BEACH	INDUSTRIAL ACCESS	2013	GEOSYNTHETIC (Combi-grid) + GRANULAR	3.5	LABORATORY TEST
BUNYA Road	BRIDGES	COLLECTOR	2013	GRANULAR OVERLAY	8.0	LABORATORY TEST
POINT CARWRIGHT Drive	BUDDINA	TRUNK COLLECTOR	2013	FULL DEPTH ASPHALT	5.5	LABORATORY TEST
MARY Street	ALEX HEADS	ACCESS STREET	2013	GEOSYNTHETIC (Tensar) + GRANULAR	6.5	LABORATORY TEST
GLENVIEW Road	GLENVIEW	RURAL COLLECTOR	2012	GRANULAR	20.0	LABORATORY TEST

Table 4: Survey Sites - Sunshine Coast Council Projects

In addition to the FWD testing, Sunshine Coast Council commissioned a contractor to undertake a road condition survey of their whole network mid-2013. This data was used to investigate the types of failures and to understand the pavement conditions on the above listed projects as well as all council roads which were constructed within the last ten (10) years.

Incorporating laser profiling data such as roughness, rutting and texture depth provides a robust data set to understand pavement conditions.

4.4.1 Falling Weight Deflectometer (FWD)

The surface deflection of a flexible pavement under an applied load provides a good indication into the structural integrity of the pavement. It is also an important parameter used in the design of pavement rehabilitation treatments through the back analysis of existing pavements. It is used to estimate existing pavement layers and subgrade modulus.

The testing was undertaken by council's contractor Pavement Management Services, located primarily on the Sunshine Coast, QLD. Pavement Management Services currently operates two falling weight deflectometers from the Dynatest Engineering family. The units are air portable for movement to and from various locations. The equipment has completed various projects including testing of Cocos Island airport off the Western Australian Coast and container loading facilities at Port Botany (PMS, 2014).

Figure 18: Pavement Management Services - FWD Equipment

The falling weight deflectometer is the world standard dynamic plate bearing test for the non-destructive testing of the structural capacity of flexible pavement. The equipment uses up to nine (9) seismic geophones to measure the deflection of the road pavement under the application of an applied load from a predetermined height as seen in Figure 19 below. Each test was adjusted, where appropriate for the pavement temperature at the time of testing.

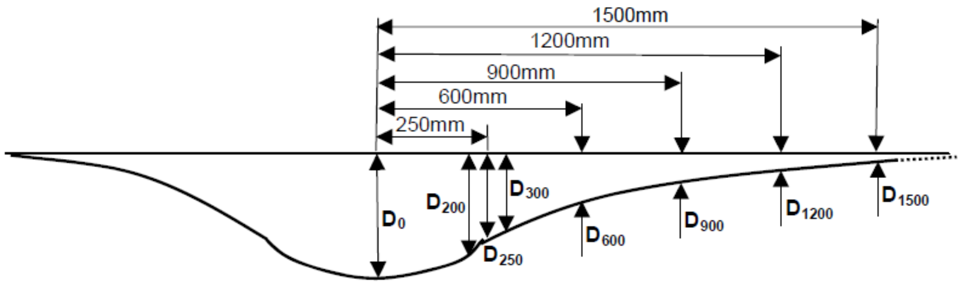


Figure 19: Typical FWD Displacement Measurements

For the purpose of this research project, testing was undertaken at 40m intervals at the above listed sites. The maximum deflection (D_0) and curvature (D_{200}) were used to analyse the performance of each pavement.

The Curvature Function (CF) gives an indication of pavement stiffness and therefore fatigue of the pavement. Results were compared with Austroads (2009) *Guide to Pavement Technology: Pavement Evaluation and Treatment Design* standards.

The correlation between the severity of rutting and maximum deflection assisted in determining the structural deficiencies within the different pavement structures, consequently suggesting which treatments are more effective.

4.4.2 Road Condition Survey – Laser Technology (RPS)

Incorporating laser profiling data such as roughness, rutting and texture depth provided a robust data set to understand pavement conditions. Radar Portal Services was the successful contractor and awarded the contract to undertake Council's most recent road condition survey.

Radar Portal Services used a system known as the Roadscout 3 pavement monitoring unit.

Figure 20: Radar Portal Services - Roadscout 3

The Roadscout 3 crack detection and crack mapping is achieved through:

1) **Full 4.0m lane width imaging:** The full lane width (and a bit more) is scanned in a single pass. Systems that scan only part of the lane (e.g. 2-2.5m scan width), potentially leave serious surface defects undetected.

2) **Consistent Lighting:** Artificial lighting ensures consistent image illumination; independent of sunlight. Artificial lighting is achieved over the full 4.0 scan width even in full sunlight, through the use of high brightness led lighting.

Manual and automatic crack detection is highly susceptible to differences in lighting conditions, as crack detection relies on clear shadows being formed by sunlight. Depending on the position and intensity of the sunlight, cracks can change from being either obvious to not detectable. This can lead to widely different assessments of the level of crack for the same section of road. The RoadScout system eliminates this issue through the use of artificial lighting over the full lane width. This is achieved without disturbing other road users.

3) **High resolution:** 1mm surface resolution is produced over the full 4.0m lane width. This allows early stage cracking to be easily detected: the key to cost effective pavement management. Current tests have shown that the system on average will detect more than twice as many surface detects in comparison to assessment through high resolution asset camera imagery (this however depends on the nature of the surface distress). Detecting surface defects early is extremely important as it allows the use of cheaper surface treatments, extending road life. The total savings over the life of the road that can be achieved through this type of monitoring and maintenance are normally orders of magnitude more than the cost of data collection.

4) **Data Quality can be verified:** The images collected by the system are stored, allowing more effective automation and also allows verification of the results. Data can be viewed at many levels from raw images, to crack mapping. The result is a very open system with results that can be proven.

5) **Linked to Laser Profiling System:** Surface defects displayed in the surface image can be linked to the laser profiling system outputs, allowing improved understanding of the surface defects detected. This is especially useful for bleeding and other bulk asphalt degradation defects.

6) **Unbroken Imagery:** One Camera. One Lens. This ensures crack double mapping does not occur and allows easy full defect mapping for project level work.

7) **Precisely positioned:** Accurately linked to a high precision positioning system allows data to be accurately positioned with respect to chainage or geo-spatially.

8) **Allows detection of cracks with 'pumping of fines':** The system can detect cracks < 1mm when associated with pumping of fines.

9) **Data useful for both network level and project level analysis:** The data quality is the same as manual produced field crack maps. Rapid data collection and rapid analysis significantly reducing the cost of such high level assessments, allowing it to be applied at network levels. When rehabilitating pavements, it is often not necessary to re-collect road condition data. Improved time assessment of surface defect changes also allows improved road rehabilitation designs, by better understanding the current road state and the time progression of the surface distress. The end result is improved road life and reduced total costs.

10) **Quantitative conversion to higher level assessments:** Automatic assessment of ROCON90 road rating from crack maps allows quantitative assessment of road cracking condition. As a result the outputs are less susceptible to operator differences and allows more effective time differencing of network level cracking.

(Radar Portal Services, 2014)

The RoadScout 3 is calibrated to Austroads standards and measures the following:

- Roughness (IRI, IRI3, NAASRA)
- Rutting
- Texture Depth
- Surface Defects (cracking)

The RoadScout 3 uses a completely different system to measure rutting, texture depth and roughness. The features of the system are:

1) A laser triangulation system that measures 2048 points over a 4.0m lane width.
2) Measures 1.5 million points per second.
3) 0.5mm height accuracy per point.
4) < 0.5mm accuracy when spatially integrated.
5) Current repeatability of around 0.4mm without lane alignment.
6) 50mm profile spacing in the direction of travel. 2mm spacing across the lane.
7) Data allows re-alignment post data collection.

Validation of the System

For rutting calibration the system was calibrated against a straight edge. With this method, a straight edge was placed in both wheel paths, and the maximum rut depth within the two rest points of the straight edge was determined. A mark was made on the road surface to allow exact chainage and transverse alignment of the straight edge. The area was then repeatedly scanned. Later, the locations were detected using the road markings, allowing a precise positioning of the straight edge for the rutting calculation.

Expected errors for the straight edge a reference device is in the order of around 1mm-2mm, due to the manual nature of the reference device.

The results for a number of sections are shown in Figures 21 and 22. The line of bet fit parameters and the co-efficient of correlation are given in the figure captions.

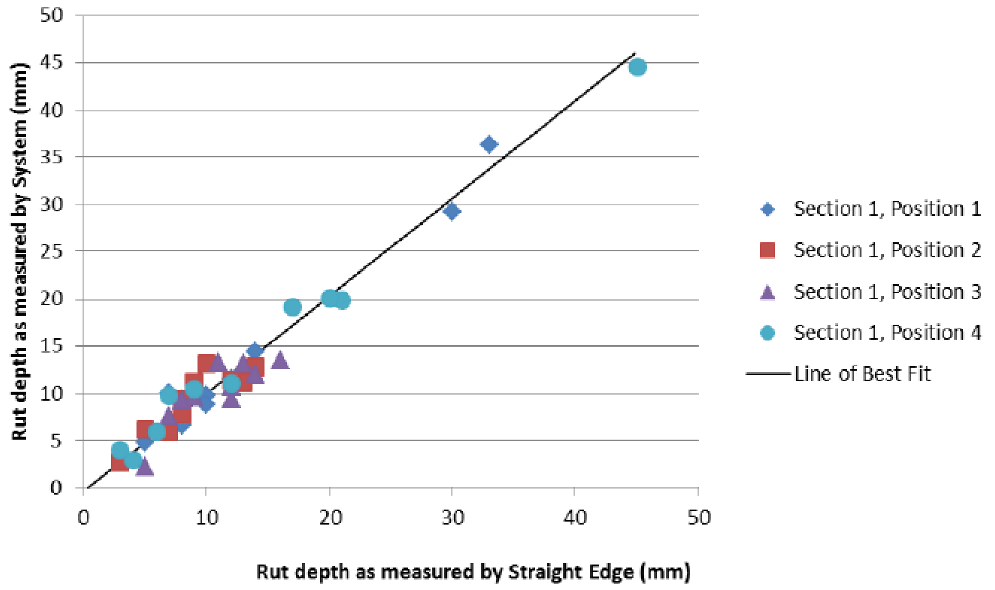


Figure 21: Rut depth comparison between a straight edge measurement and the RoadScout3 System measurement for a 500m section of road. A = 0.966, B = 0.37mm, R^2 = 0.963. Positions 1 to 4 refer to both wheel paths for each side of the road

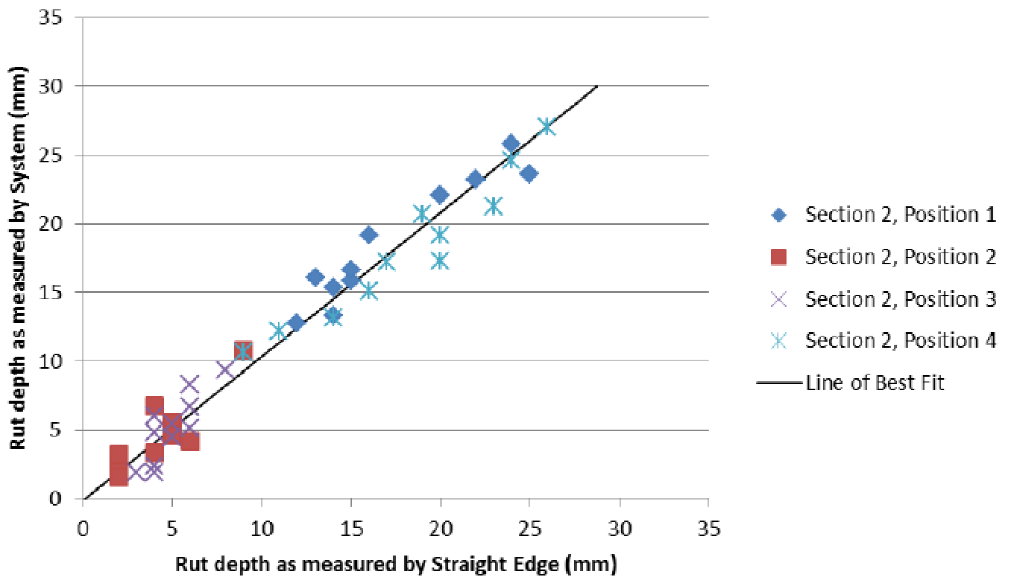
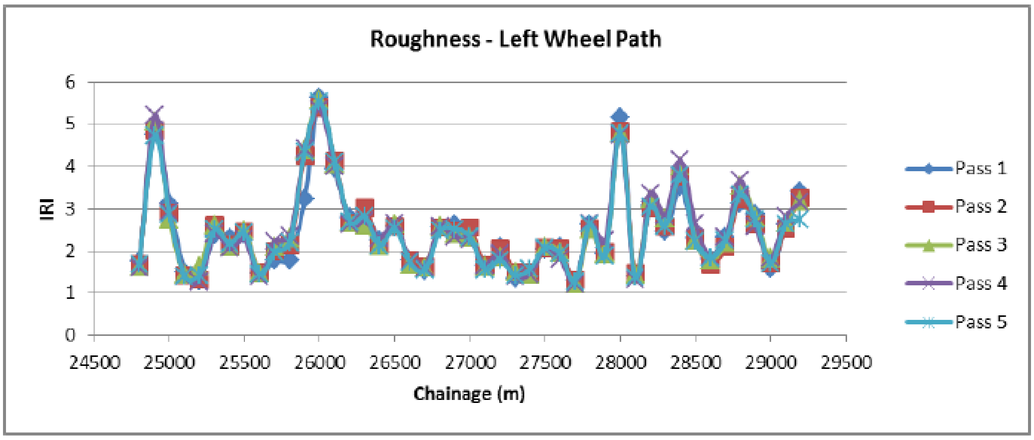


Figure 22: Rut depth comparison between a straight edge measurement and the RoadScount3 System measurement for a second 500m section of road. A = 0.955, B = 0.12mm, R^2 = 0.966

To pass the AustRoads Standards, it requires that it passes the following criteria: $0.90 \leq A \leq 1.10$, $-2.5 \leq B \leq 2.5$ mm, $r2 \geq 0.85$. As seen above the Roadscout3 system satisfied this criteria.

A section of road was repeatedly scanned with the Roadscout3 system to test the roughness calibration via the Loop Method. The results are shown below.



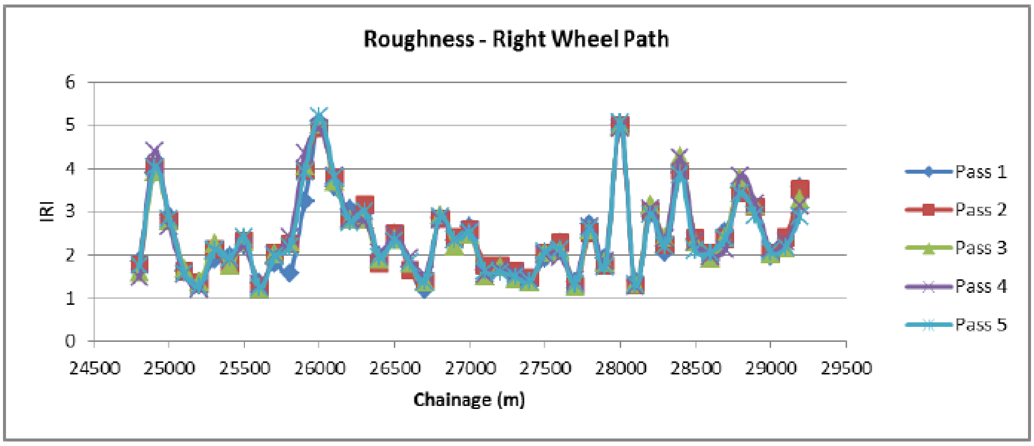


Figure 23: Roughness repeatability for the RoadScout3 system. The system achieved the required 0.95 R^2 coefficient of determination repeatability value (Radar Portal Services, 2014).

The RoadScout system was tested against both a MLP and DLP. The results for the MLP are shown in Figure 24.

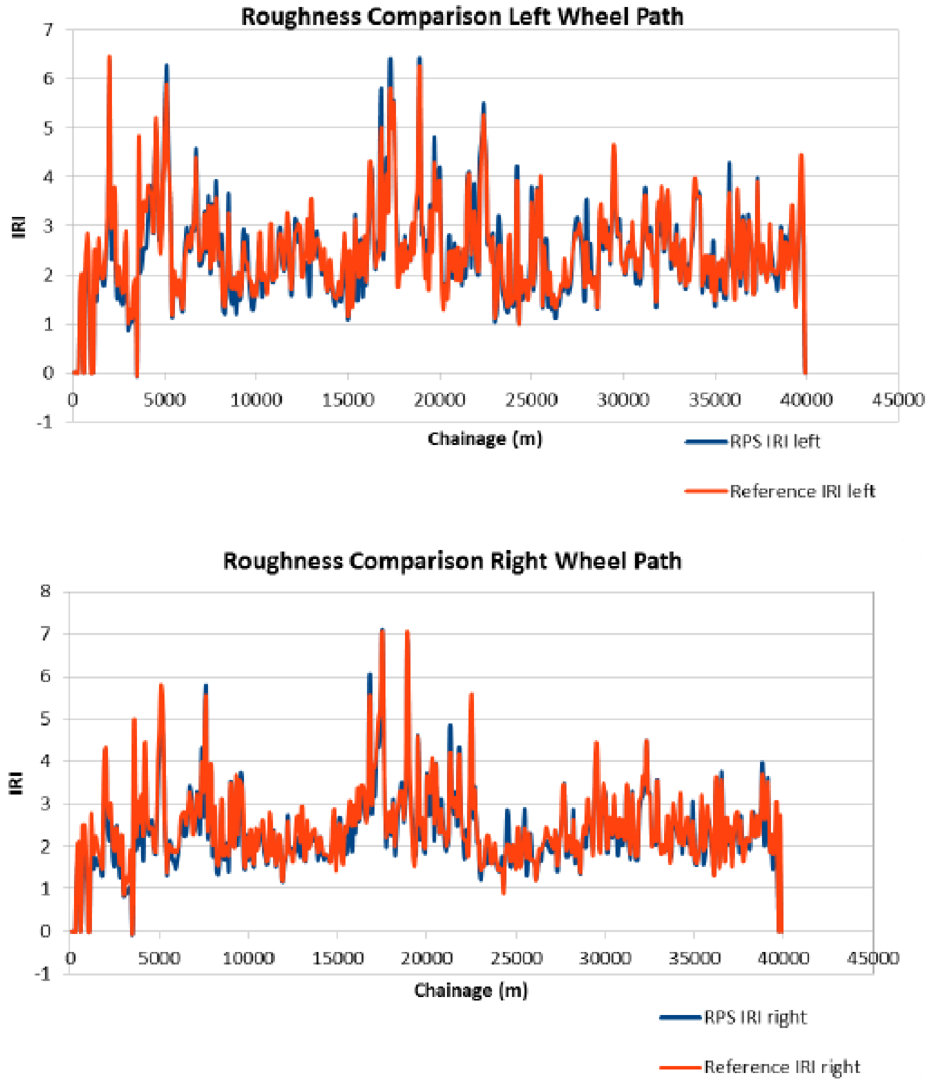


Figure 24: Validation of Roughness correlation against both MLP and DLP systems (Radar Portal Services, 2014).

Texture depth calibration was completed using the sand patch test method. Results are shown below.

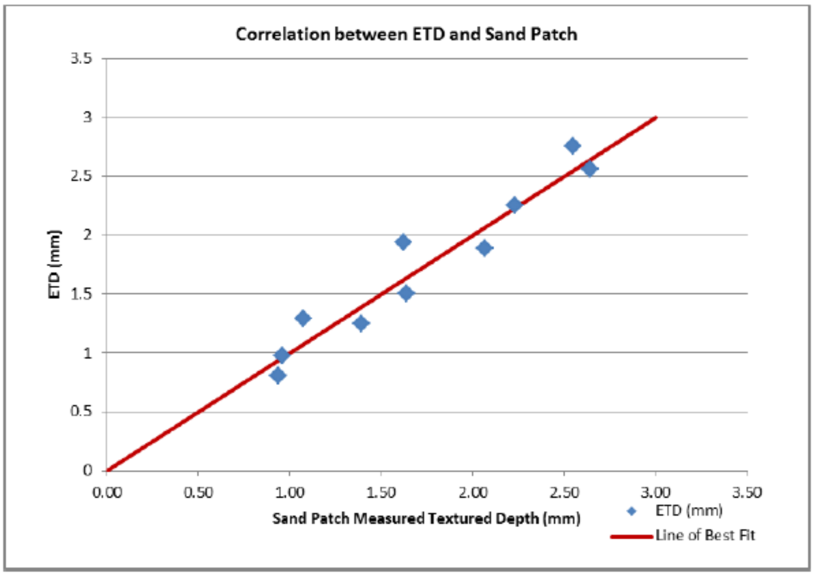


Figure 25: Correlation between RoadScout 3 and Sand Patch Test Method. A = 0, B = 1.0, R^2 = 0.94

Validation of the surface defect types was completed by visual inspection and as a result this data has been excluded from this research project. A correlation as low as 16% was achieved from the data supplied to visual inspection by Council staff. Issues with human categorisation of the surface defects are currently being reviewed. It is envisaged corrected results will be achieved after reviewing the data further. At this point in time Council is still waiting to receive this data. Inspections sheets from the surface defect validation process can be found in Appendix C and the correlation with road condition survey data can be seen below in Table 5.

Road	Road Block	Total Cracking - SCC (m)	Total Cracking - RPS (m)	Correlation
Henzell St, Dicky Beach	101	235	37	16%
Henzell St, Dicky Beach	102	34	0	0%
Henzell St, Dicky Beach	103	52	0	0%
Wilson Ave, Dicky Beach	101	83	49	59%
Wilson Ave, Dicky Beach	202	65	14	22%

Table 5: Correlation between SCC and RPS Surface Defect Inspections

4.4.3 Evaluation of Results

Evaluation of FWD Results

As an indicator of structural condition, deflections aid the selection of appropriate structural rehabilitation treatments if any is required, by identifying:

- The structural adequacy of the overall pavement;
- Homogeneous lengths of pavement which might be treated similarly;
- Areas of weak pavement, requiring specific treatment; and
- Areas for more detailed pavement investigation.

(Austroads, 2009)

Falling weight deflection testing can determine the structural adequacy of existing flexible pavements and their resistance to deformation. Austroads (2009) suggests deflection data can provide significant information on the condition of a pavement. For instance, some of the information testing can provide includes:

- Very high local deflections (> 1.5mm) may indicate a weak subgrade;
- High values of curvature function (CF) may indicate low stiffness in the upper pavement layers or cracking;
    - Granular pavements are expected to have CF values 25% - 35% of their maximum deflection;
    - Values > 35% represent low stiffness in granular base courses;
- High deflections near pavement edges may indicate poor local drainage; and
- Low but extremely variable deflection pattern may indicate an old, cracked, failing or poorly patched pavement.

Austroads (2009) also suggests plotting the severity of rutting against maximum deflection to assess whether rutting is related to the structural capacity of the pavement. This evaluation method has formed a significant part of this research project.

Evaluation of Road Condition Data

Roughness

Roughness measurements are usually taken as part of a routine or cyclic network testing program. Roughness is a condition parameter that characterises deviations from the intended longitudinal profile of a pavement. Measurement of roughness focuses on characteristic dimensions that affect vehicle dynamics and hence road user costs, ride quality and dynamic pavement loads (Austroads, 2007).

Austroads (2009) suggests that roughness values can be derived from either physical response of a vehicle to a road surface, otherwise known as NAASRA count or by inputting the longitudinal profile of a road surface and using a mathematical model of a hypothetical vehicle, commonly undertaken by laser sensors and specific software.

Two means of reporting and measuring roughness currently used in Australia are:

1. NAASRA roughness counts; and
2. International Roughness Index (IRI) – average results of the application of a computer model of a standard 'quarter-car' to the measured longitudinal road profile of each wheel path. The simulated travel speed of IRI is 80km/h.

(Austroads, 2007)

Table 6 outlines the maximum desirable roughness counts for varying road functions. Local roads have no defined limits as roughness levels depend on local conditions and traffic calming measures.

Road Function	Typical maximum desirable roughness for new construction or rehabilitation (length >500 m)	Indicative investigation levels for roughness (counts/km)	
		Isolated areas	Length >500 m
Freeways and other high-class facilities	to 40 counts/km	110	90
Highways and main roads (100 km/h)	to 50 counts/km	140[2]	110
Highways and main roads (less than 80 km/h)	to 50 counts/km	160	140
Other local roads (sealed)	No limits defined[3]	No limits defined[3]	No limits defined[3]

1. Roughness measures are in equivalent NAASRA counts/km.
2. Lower levels may be appropriate where total traffic or heavy vehicle volumes are high.
3. Roughness levels depend on local conditions and traffic calming measures.

Table 6: Maximum Desirable Roughness Counts (Austroads, 2009).

Since it is recognised by Austroads (2009) that roughness counts on local roads depend on local conditions and the inclusion of local traffic calming devices etc. Radar Portal Services provided an alternative form of reporting roughness for a local network outlined as a curve in the direction of travel assessment (IRI3).

The goal of this measure is to produce an indication of the deviation from a normal, as designed, road surface. To achieve this, a second order polynomial curve ($ax^2 + bx + c$) of least squares is fitted to the surface. The standard deviation of the difference between the actual surface and this ideal surface is then calculated. The length of the curve is a parameter, but for most suburban roads this typically should be 10 meters long.

This measurement is different to an IRI measurement (mm/m). IRI measurements typically don't work in a suburban street context, due to a number of reasons:

1) Suburban streets are designed for variable speed. On corners changes in camber are acceptable, because cars are normally traveling at a slower speed;
2) Road geometry associated with topography (which for a highway would be eliminated), is often left unchanged;
3) Speed control measures such as speed bumps or other local area traffic management systems (LATMs) are common;
4) Conventional IRI measurement systems required the unit to travel at speeds of > 20 km/h to reliably produce a measurement, thus much of the network is left unmeasured; and

5) Conventional IRI measurement systems produce erroneous results when the system changes angle (due to differences in the camber of the road), or when turning a corner. This is because the effect from gravity changes.

(Radar Portal Service, 2014)

The typical method is to remove sections where the vehicle is travel too slow, where there is LATMs and when the vehicle is traveling around a corner. While this does improve the IRI results, it does that at the expense of removing data or regions from the assessment.

IRI3 takes a different approach. Predominately it is less affected by the long low frequency defects then IRI. This means it can be used of smaller sections of road, at slow speeds. Also normal topography geometry does not significantly influence the measure.

The typical thresholds for IRI3 are:

Road Type	IRI3 Values (mm)
New Road	0-3mm
Good Condition	3-6mm
Moderate Condition	6-12mm
Poor/Dirt Road	12-20mm
Very Poor	>20mm

Table 7: IRI3 Thresholds (Radar Portal Services, 2014).

Results from the contractor's IRI3 method was compared with the standard International Roughness Index (IRI) with varying results.

Rutting

Rutting is a form of pavement deformation typically evident in flexible pavements, which is caused by the traversing of loaded wheels over its surface. It is evident as a longitudinal depression along wheel paths.

Ruts are usually measured using a standard 1200mm straight edge and a depth wedge, or more recently using laser sensors as used by Radar Portal Services. Austroads (2009) defines rutting as a measurement of the maximum vertical pavement displacement in the transverse profile through a wheel path or traffic lane. Measurement of the rut depth gives an indication of the surface and structural condition of the pavement and also provides an indicator of potential aquaplaning problems (Austroads, 2009).

Rut depth data can be used to determine:

- Deformation depth – wide ruts with no shoving may indicate deformation at subgrade level;
- Inadequate pavement strength – determine by plotting measured maximum pavement deflections at various chainages against measured rut depth. Austroads (2009) suggests the higher the correlation of rut depth and deflection the more likely rutting is due to inadequate pavement strength; and
- Densification of pavement under early traffic – if there is no correlation between rutting and deflection and no shoving evident.

Structurally, rut depths below 10mm are regarded as not significant, at 10mm, and under conditions of high vehicle speeds and water ponding, rutting is regarded as potentially significant. Rutting becomes a critical structural issue and safety problem around 20-25mm.

Austroads (2007) reporting parameters are as follows:

Rutting should be reported in terms of severity and extent for the left wheel path (and for the lane where available) for each reporting interval, as:

Severity: - mean rut depth (mm), to the nearest whole number

- standard deviation of rut depths (mm), to one decimal point

Extent: - the percentage of the length with maximum rut depths in 'bins' as follows:

rut ≤ 5 mm	5 mm < rut ≤ 10 mm	10 mm < rut ≤ 15 mm
15 mm < rut ≤ 20 mm	20 mm < rut ≤ 25 mm	25 mm < rut ≤ 30 mm
30 mm < rut ≤ 35 mm	35 mm < rut ≤ 40 mm	rut > 40 mm

to the nearest whole number.

Figure 26: Rutting Reporting Parameters (Austroads, 2007).

Evaluation of results for this research project were undertaken in accordance with the above Austroads (2007) reporting parameters.

4.5 Meeting Records

- 28/4/14 – Meeting with John Tucker formerly of Golders and Associates. An informal discussion on John's experience working on the Sunshine Coast, the geological history of the region, problems encountered on a variety of projects and recommendations for further research to assist with achieving a successful outcome for this project. Conclusion: The Sunshine Coast consists of widespread deposits of Landsborough Sandstone and coastal alluvial sediments. Areas near the coastline are generally comprised of collapsible soils with high fines content.
  John suggested water ingress to be the most damaging factor to our pavements, based on his experience constructing roads and extensive geological knowledge of the subgrade materials encountered within the region.

- 11/6/14, Meeting with Richard Murray of RPQ, Swanbank. A site visit was conducted to RPQ's Swanbank plant to learn more about the use of plant mixed foamed bitumen pavement material, and to inspect their mobile plant. RPQ's successful tender for NDRRA work surrounding Ipswich provided an opportunity to conduct site visits to witness the various construction stages of the use of plant mixed foamed bitumen pavement material. Conclusion: The use of plant mixed foamed bitumen pavement material could be an alternative method used on the Sunshine Coast Council, providing the opportunity to increase the recycling of existing pavement materials in future pavement rehabilitations.

- 23/06/14, Meeting with Cameron Shields, Assets Office at Sunshine Coast Council. Discussions included the current pavement rehabilitation methods used within the Sunshine Coast region, prioritisation of projects and considerations when selecting pavement options. Conclusion: Projects are assessed on a region wide approach based on a number of criteria including but not limited to:

  - Current condition;
  - Road hierarchy;
  - Safety / Risk to road users;
  - Financial considerations;
  - Environmental impacts; and
  - Corporate demand.

  Sunshine Coast Council's Asset Management Team utilises processes outlined in the Transport and Main Roads Pavement Rehabilitation manual for the typical pavement rehabilitation process including condition assessment, structural capacity analysis, rehabilitation design and economic analysis. Council is currently undertaking a review of its rehabilitation methods and reviewing the corporate definition of pavement rehabilitation projects in comparison to full road reconstruction to achieve a more holistic approach to maintaining the network.

- 24/6/14, Discussions with Tim Letchford, Operations and Maintenance Manager at Sunshine Coast Council. Discussions included historic rehabilitation methods for the Sunshine Coast region, in particular the former Maroochy Shire Council. Conclusion: The former Maroochy Shire Council undertook substantial pavement stabilisation works and minimal asphalt deep lift pavements.

Stabilising pavements increases the risk of shrinkage and block cracking however, treatment via the use of polymer seals is suggested.

- 17/7/14, Demonstration of road condition video software, Radar Portal Services. This formal presentation outlined some results from the road condition survey and demonstrated the software to extract results. Conclusion: The software and data created a great opportunity to perform the analysis of data collected, however given Information Technology (IT Services) constraints in storing such large files, this software would not be available for use prior to the completion of this project.

- 4/9/14, Geofabrics Presentation, Brisbane. A formal presentation to industry professionals on the history and various uses of Tensar Grid products. Conclusion: Potential to explore further use of the products and revisit the way it is currently used in pavement rehabilitations within the Sunshine Coast region. Sunshine Coast Council has had positive and negative results from the use of these products.

5.0 Site Survey of Local Sunshine Coast Roads

5.1 Mary Street – Alexandra Headland

In 2012 Mary St, Alexandra Headland was prioritised as a pavement rehabilitation for the 2012/2013 financial year. A visual inspection was undertaken by Council officers on 29 June 2012 following a period of heavy rain. The inspection revealed that the pavement demonstrated signs of significant structural damage including rutting and crocodile cracking. The pavement had been extensively patched with significant crocodile cracking in patches, indicating that the repairs were not successful. Photos below in Figure 27 shows typical pavement distress prior to pavement rehabilitation.

Figure 27: Mary St, Alexandra Headland - Site Photographs

There was evidence of poor drainage over most of the section with water appearing to saturate the pavement base course at many locations.

Two boreholes revealed granular pavement thicknesses of 180 and 165 mm and seal thicknesses of 40, and 35 mm i.e. total pavement thicknesses of 220 and 200 mm. Subgrade soils were generally medium to high plasticity sandy clays and clayey sands. Laboratory CBR values of 4% and 5% were recorded.

At the time of the site inspection there was significant evidence of poor drainage with water observed seeping through cracks in the surfacing and evidence of pumping of clay fines. A service road has been provided on the western side of Mary Street starting approximately 40m from the Janet Street intersection and exiting near the Buderim Avenue intersection. The service road is separated from Mary Street by a stone pitched retaining wall with the service road on the high side of the wall. A number of clay pipe drains were observed near the base of the wall. The drains were not flowing at the time of the site inspection and appeared to slope down from the wall.

Kerb and channel was provided on the western side of the service road. The visual inspection revealed some evidence of poor drainage and pavement failures in the service road. It is possible that some runoff water from the service road may be reaching the retaining wall foundation. Furthermore, inspection of the gully pits in Mary Street did not reveal the presence of any subsoil drains.

5.1.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- In-Situ Stabilisation;
- Granular Pavement; and
- Geogrids.

In addition to the above options, some consideration was also given to the construction of a full depth asphalt pavement.

The existing pavement was in poor condition with extensive crocodile cracking, failed patches and showing evidence of poor drainage. The existing granular pavement thickness varied from 200 to 220 mm which was considerably less than the thickness which was required for a granular pavement with the design traffic and subgrade support conditions which applied. Stabilisation and resealing of the existing pavement was not considered a viable option.

Reconstruction of the granular pavement comprised removal of existing material down to subgrade level and construction of a new granular pavement and was selected as the preferred option.

In this instance the use of geogrids was considered due to the weak subgrade. Geogrids are used where traditional treatments will result in excessive depths of pavements which may interfere with existing services. In this case it was determined that geogrids could allow deletion of the lower sub-base layer, minimising service clashes as a result of requiring a thinner pavement. The negative impact of the use of geogrids include the complications for future maintenance activities.

Deep lift asphalt was considered, however Council guidelines do not recommend its use for other than heavily traffic roads and also recommend caution where weak or deep subgrades are encountered, leading to possible bogging of paving machines.

The recommended design comprised boxing out and reconstruction of a new granular pavement, including the use of geogrids with asphalt surfacing. Rectification of pavement drainage was also undertaken.

5.1.2 Pavement Design and Construction Process

The design adopted comprised boxing out and reconstruction of a new pavement with the use of geogrids. The recommended design comprised a 40mm AC surface over a

250mm thick granular pavement on Tensar TX160 and Class B non-woven geofabrics or equivalent bonded.

Preliminary calculations indicated a total asphalt thickness of approximately 225mm for a pavement constructed directly on a CBR 4 subgrade. It was probable that a working platform may also have been required to facilitate construction and provide adequate performance. This option was not recommended due to these factors.

The design traffic loading was 5.2×10^5 ESAs. For design purposes the value was rounded up to 6×10^5 ESAs. This value was considered appropriate when compared with similar roads. A design CBR value of 4% was adopted based on subsurface investigation and laboratory testing.

Council's normal practice for urban street surfacing is to use a minimum of 30mm of DG10 mix. Because of the relatively steep grade of the project, a greater surfacing thickness was deemed appropriate. Therefore, a 40mm surfacing using DG10 was placed.

The following pavement configuration was adopted:

- Surfacing 40mm DG10
- Base course 125mm Type 2.1 (Min CBR 80%)
- Sub Base 100mm Type 2.3 (Min CBR 45%)

The pre-existing drainage system was also found to be inadequate and rectification was necessary for the reconstructed pavement to perform adequately. Subsoil drains were installed along both sides of the road, extending the full length of the site. Mitre drains were included on the steeper sections of the road to improve workability during construction and to intercept any seepage from deeper in the hillside. Subsoil drains were also installed below the wall to intercept any seepage at this point.

The program of works were as follows:

- Box out and place Class B non-woven geofabrics and Tensar TX160 geogrid;
- Reconstruct kerb and channel and subsoil drains;
- Reconstruct 250mm of thick granular pavement consisting of 150mm Type 2.1 base course and 100mm Type 2.3 base course material; and
- 10mm primer seal 'SURFIX PS' PME binder and 40mm DG10.

Shown in Figure 28 below.

Figure 28: Construction Processes of Mary St, Alexandra Headland

5.1.3 Investigation Results

Figure 29 below demonstrates minimal improvement in the maximum deflections from testing prior to the pavement rehabilitation in 2012 and post construction in 2014. This suggests insufficient pavement and/or weak subgrade. This could be due to moisture in the subgrade at individual test locations however, maximum deflections consistently exceed Sunshine Coast Council's intervention level for resurfacing of 1.0mm, averaging 1.09mm in both 2012 and 2014.

Austroads (2009) suggest that very high local deflections (more than 1.5mm) may indicate weak subgrade conditions. Considering significant subsoil drainage works undertaken as part of the pavement rehabilitation and site inspections prior to and post construction suggest groundwater may be a contributing to the subgrade performance.

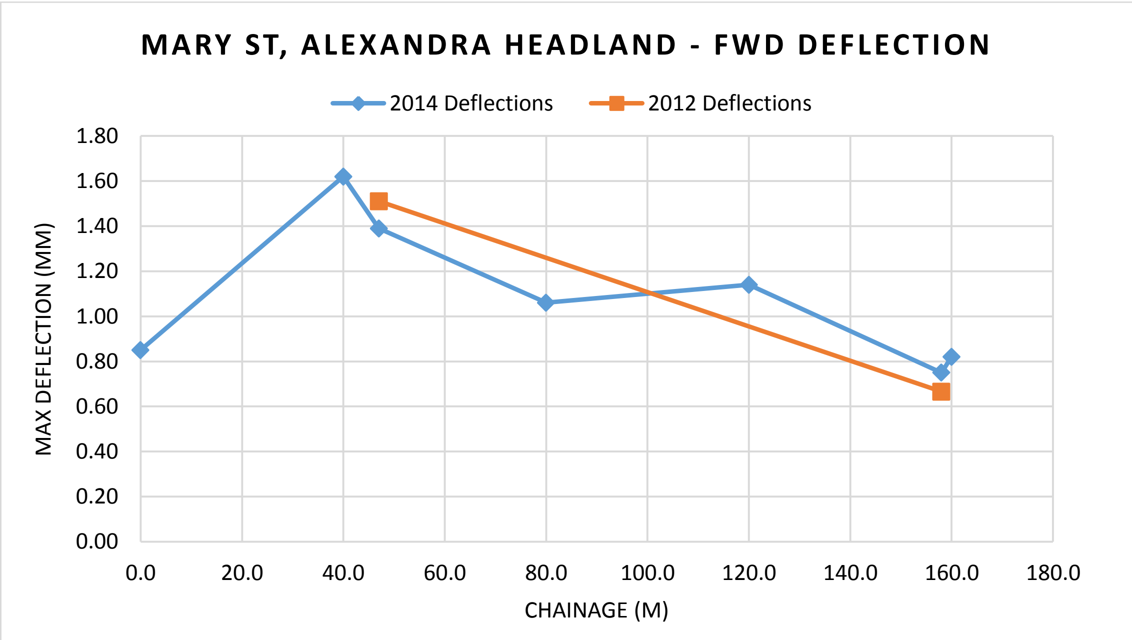


Figure 29: Mary St, Alexandra Headland - 2012 and 2104 Maximum Deflection Results

Austroads (2009) suggests high values of curvature function (CF) may indicate low stiffness in the upper pavement layers, or a pavement with cracked surfacing. For granular pavements the curvature function is likely to be 25% to 35% of the maximum deflection.

Mary Street, Alexandra Headland has an average relationship between CF values and maximum deflection of 31% for the length of the project, shown in Figure 30. Of particular interest are the five (5) locations where CF values exceeds 35% of the corresponding maximum deflection, ranging from 35% to 64%, demonstrated in Figure 30. Curvature function values at these locations range from 0.40mm to 0.96mm. Transport and Main Roads (2012) outlines pavements exhibiting CF values greater than 0.4mm may indicate a pavement that is lacking stiffness or a very thin pavement. Therefore, 25% of Mary Street is exhibiting characteristics of a pavement lacking stiffness.

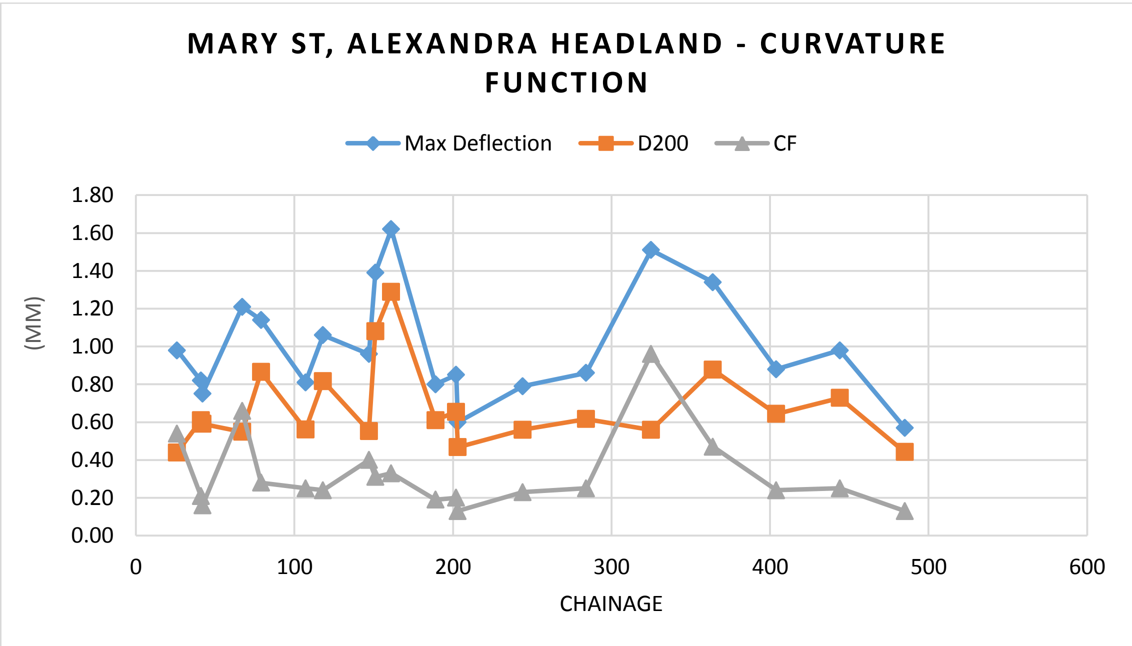


Figure 30: Mary St, Alexandra Headland - Curvature Function

Transport and Main Roads (2012) recommends plotting measured pavement deflections at various chainages against measured rut depths. The higher the correlation of rut depth and deflection the more likely the rutting is due to inadequate pavement strength. Results indicate an average rutting of 4.19mm and 3.49mm for the right and left outer wheel path respectively. Chainage 150 to 170 in Figure 31 represents the only clear section of high correlation between the maximum deflection and rutting in the right outer wheel path.

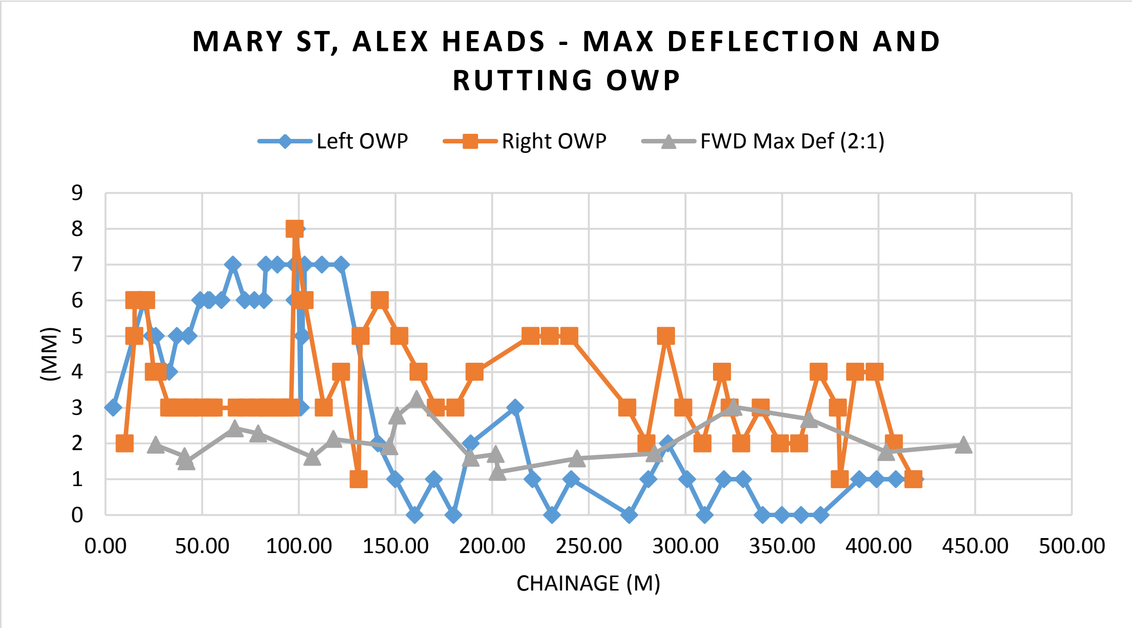


Figure 31: Mary St, Alexandra Headland - Rutting and Maximum Deflection

Reporting requirements of severity and extent of outer wheel path rutting is outlined in Section 4.4.3. In accordance with this, Mary Street displays rutting with severity ranges of 0 – 5 mm and 5 – 10mm extending for 66% and 34% of the project respectively, shown in Figure 31. Austroads (2007) reports that structurally, rutting less than 10mm is not regarded as significant. As there is no evidence of shoving along Mary Street, the results characterise a pavement where the pavement layers are too thin to protect the subgrade. A reduction in pavement thickness was undertaken due to the inclusion of Tensar Grid and further work is required on the use of Tensar Grid pavement rehabilitation options to improve understanding of the product capabilities.

5.2 Gannawarra Street, Currimundi

In 2011 Gannawarra St, Currimundi was prioritised as a pavement rehabilitation for the 2012/2013 financial year. A visual inspection revealed there was evidence of previous cement stabilisation through this area; evident on the ground with block cracking. The pavement had undergone significant heavy patching and due to the existing kerb and channel, and sections of narrowing there was minimal scope to raise the finished pavement levels. Photos below in Figure 32 show typical pavement distress prior to pavement rehabilitation.

Figure 32: Gannawarra St, Currimundi - Site Photographs

Five (5) boreholes revealed a generally good subgrade consisting predominantly of silty sand, and sand. Some sandy clays were located and all test pits indicated higher than desirable moisture content. In-situ CBR's were determined between 6.5% and 35% returning soaked CBR values of 7% to 20%.

The existing pavement was in poor condition with extensive block cracking, failed patches and evidence of poor drainage. The existing asphalt thicknesses varied between 20 to 40mm while the existing granular pavement thickness varied from 85 to 195 mm.

Rutting was observed throughout the length of the project. Ruts measured along the project were around 20mm (approximately 40% of the project) with the balance around 10-15mm, determined using a 1200 straight edge. According to the TMR Pavement Design Manual and methodology, Normal Design Standard is based on a 20mm rut in the subgrade at the end of the design life and Second Design Standard is based on a 30mm rut in the subgrade at the end of the design life. This is measured at subgrade level and generally presents as a lesser amount at the surface.

The design life of the existing pavement was determined by analysing deflection data collected in 2012, and it was concluded that the pavement had generally 1 to 4 years of residual life remaining. Further information accessed from Council's PMS data suggested that the pavement was last rehabilitated in 1996. This supported the residual life prediction. Gannawarra St, Currimundi was provided funding to be rehabilitated in the 2012/13 financial year.

5.2.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- Granular pavement;
- Deep lift asphalt;
- Cement treated base materials; and
- In-situ foamed bitumen stabilisation.

In addition to the above options, some consideration was also given to the use of geogrids.

Reconstruction of the granular pavement comprised removal of existing material down to subgrade level and construction of a new granular pavement, and was selected as the preferred option.

Deep lift asphalt was considered, however Council guidelines do not recommend its use other than for heavily traffic roads and also recommend caution where weak or deep subgrades are encountered leading to possible bogging of paving machines. There was sufficient base that some base gravel would remain as a working platform. However, this was not consistent throughout the project and this treatment did not allow for future rehabilitation once reaching the end of its useful life.

The visual inspection indicated the pavement had been previously cement stabilised, which precluded re-stabilisation of the pavement.

In-situ foamed bitumen stabilisation of the existing pavement was not considered a viable option. It was determined there was insufficient base to achieve a 20 year design life and back analysis revealed that only 11 to 12 years life could be expected.

The recommended design comprised boxing out and reconstruction of a new granular pavement with asphalt surfacing. Rectification of pavement drainage and kerb and channel reconstruction was also undertaken. The recommended pavement design included a granular pavement of 210mm to 310mm.

5.2.2 Pavement Design and Construction Process

The design adopted comprised boxing out and reconstruction of a new pavement, replacement of subsoil drainage and reconstruction of all kerb and channel. The recommended design comprised a 30mm AC surface over a 210 - 310mm thick granular pavement.

Preliminary calculations indicated a total asphalt thickness of 165 to 195mm. It was probable that a working platform may also have been required to facilitate construction and provide adequate performance as shown by Table 8. This option was not recommended due to these factors.

	LOCATION				
DEEP LIFT AC (THROUGH LANES)	**1**	**2**	**3**	**4**	**5**
ADOPTED CBR (%)	20	19	17	25	7
ASPHALT THICKNESS	**165**	**165**	**165**	**165**	**195**
EXIST PAVEMENT THICKNESS =	190	120	180	200	220
GRAVEL REMAINING AS WORKING PLATFORM (mm)	**25**	**-45**	**15**	**35**	**25**

Table 8: Deep Lift Asphalt Option - Gannawarra St, Currimundi

The design traffic loading was 9.7×10^5 ESAs. For design purposes the value was rounded up to 1×10^6 ESAs, based on 8% commercial vehicles and assumed growth of 3%. This value was considered appropriate when compared with similar roads.

The following pavement configuration was adopted:

- Surfacing 30mm BCC2
- Base course 110mm Type 2.1
- Upper Sub Base 100mm Type 2.3
- Lower Sub Base 100mm Type 2.5

The pre-existing drainage system was also found to be inadequate and rectification was necessary for the reconstructed pavement to perform adequately. Subsoil drains were installed along both sides of the road, extending the full length of the site. The program of works were as follows:

- Box out and remove kerb and channel
- Reconstruct kerb and channel and subsoil drains
- Reconstruct 310mm of thick granular pavement consisting of 110mm Type 2.1 base course, 100mm Type 2.3 and 100mm Type 2.5 base course material
- 10mm primer seal 'SURFIX PS' PME binder and 30mm BCC2

5.2.3 Investigation Results

Figure 33 demonstrates marginal improvement in the maximum deflections from testing prior to pavement rehabilitation works in 2012 and post construction in 2014. Results demonstrate a uniform pavement structure. Average maximum deflection results for 2012 and 2014 were 0.79mm and 0.52mm respectively.

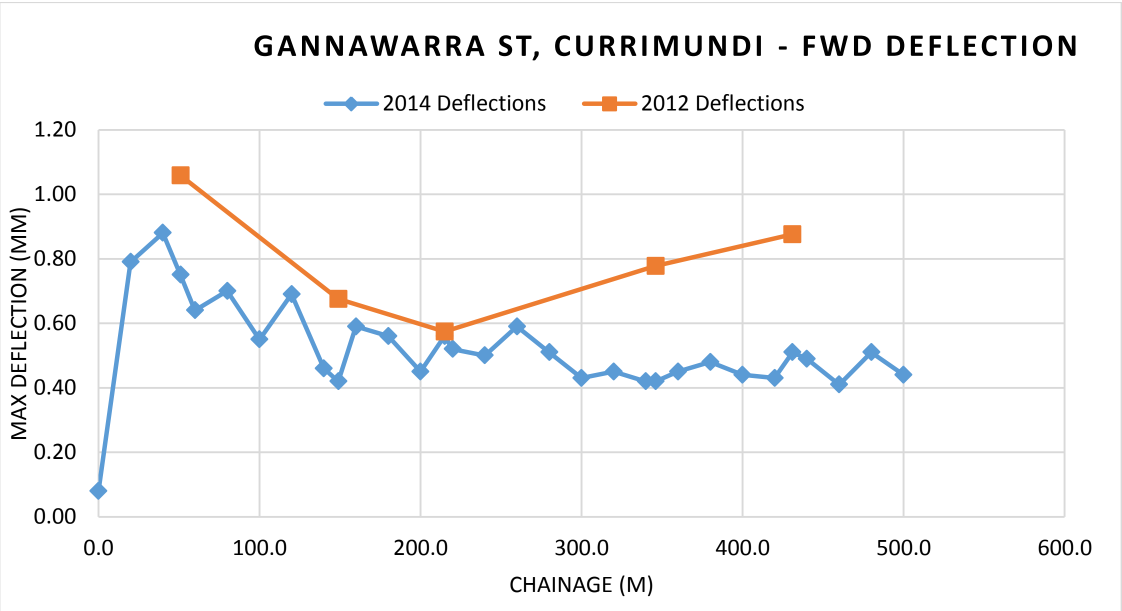


Figure 33: Gannawarra St, Currimundi - 2012 and 2014 Maximum Deflection

As outlined previously, Austroads (2009) reports that high values of curvature function (CF) may indicate low stiffness in the upper pavement layers. For granular pavements the CF values are likely to be 25% to 35% of the maximum deflection.

Gannawarra Street, Currimundi has an average relationship between CF values and maximum deflection of 26% for the length of the project, shown in Figure 34. TMR (2012) also suggests low values of CF (<0.2mm) indicate a stiff pavement and 90% of Gannawarra Street resulted in CF values below this with two (2) test locations marginally above at 0.21mm located at chainages 20m and 40m, resulting in values of 0.21mm.

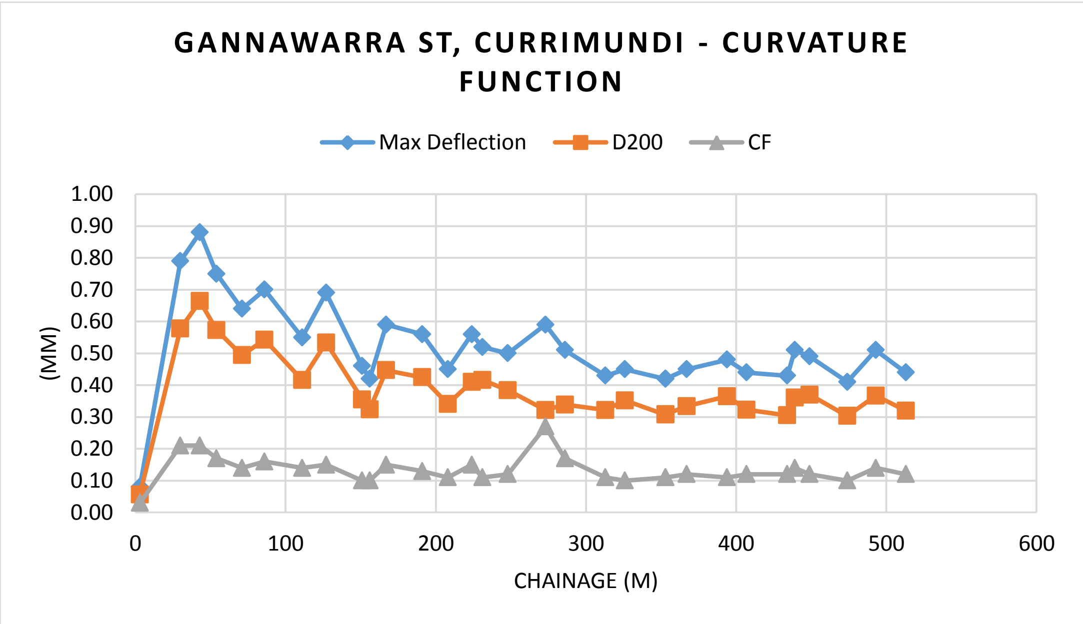


Figure 34: Gannawarra St, Currimundi - Curvature Function

One (1) additional location returned a marginally high value of curvature in relation to maximum deflection, with a correlation of 46%. This locations returned a CF value of 0.27, marginally exceeding the ideal CF value of 0.2mm or less as outlined by TMR (2012).

Figure 35 indicates an average rutting of 4.00mm and 3.78mm for the right and left outer wheel paths respectively. Chainage 70 to 85 represents a section of high rutting. As the rut depths measured at these locations do not correlate with pavement deflection and there is no shoving evident TMR (2012) suggests the likely cause is densification of the pavement layers under traffic early in the life of the pavement. This section is located at the intersection of Doondoon Street and experiences higher traffic loadings than the remainder of the project. This may also contribute to the high rutting measured by Radar Portal Services due to changes in cross-fall associated with keying in to the adjacent street.

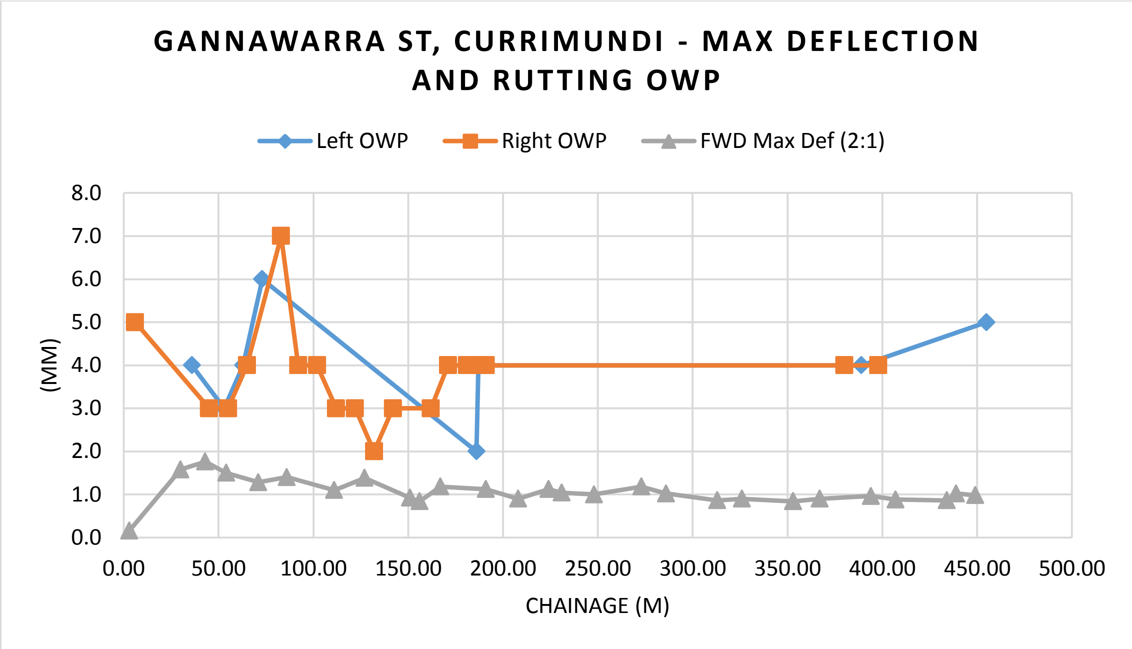


Figure 35: Gannawarra St, Currimundi - Rutting and Maximum Deflection

Reporting requirements of severity and extent of outer wheel path rutting is outlined in Section 4.4.3. In accordance with this, Gannawarra Street displays rutting with severity ranges of 0–5mm and 5–10mm extending for 93% and 7% of the project respectively. Austroads (2007) reports that structurally, rutting less than 10mm is not regarded as significant. The above results indicate adequate pavement strength and depth, however, minor rutting may be due to densification of the pavement layers under early traffic.

5.3 Lyon Street – Dicky Beach

The section of Lyon Street which underwent pavement rehabilitation in 2013 extended from Cooroora Street to the end of Lyon Street which terminates in a cul-de-sac. The length of the street was approximately 310m.

A visual inspection was undertaken in June 2012 following a period of heavy rain. The inspection revealed that the pavement was showing signs of significant structural damage including rutting and crocodile cracking. The pavement had been extensively patched with significant crocodile cracking evident in the patches indicating that the repairs had not been successful. Photos in Figure 36 show the typical pavement distress evident.

Figure 36: Lyon St, Dicky Beach - Site Photographs

There was evidence of poor drainage over much of the section with water appearing to saturate the pavement base course at many locations. Recently constructed sections of kerb and an associated gully pit on the western side of Lyon Street showed evidence of subsidence. The intersection of Lyon Street and Lawley Street had been constructed using brick pavers. Subsidence of the pavers was also observed.

The three (3) boreholes which had been undertaken revealed granular pavement thicknesses of 120, 90 and 60 mm and seal thicknesses of 60, 50 and 50 mm i.e. total pavement thicknesses of 180, 140 and 110 mm. Subgrade soils were generally medium to high plasticity clays. Laboratory CBR values of 3%, 10% and 2% were recorded.

At the time of the site inspection there was abundant evidence of poor drainage with water observed seeping through cracks in the surfacing and evidence of pumping of clay fines. Inspections of gully pits revealed the presence of apparently functional subsoil drains in the section of road to the south of Lawley Street. At the gully pit located on the south-western corner of the intersection with Lawley Street a partly blocked subsoil drain entering the pit from the uphill direction. No evidence of functioning subsoil drains was seen elsewhere. Significant drainage issues were observed for approximately 80m back from Cooroora Street. These issues included flooded gully pits and meter pits and evidence of pumping of clay fines through cracks in the pavement surface.

5.3.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- In-Situ Stabilisation;
- Granular Pavement; and
- Geogrids.

In addition, replacement of existing kerb and channel was included as part of the project.

Stabilisation and resurfacing of the existing pavement material was not a viable option due to the existing granular pavement thickness which varied from 60 to 120mm. Considerably less than the thickness required for a granular pavement with the design traffic and subgrade conditions which applied. Stabilisation would not have addressed the significant drainage problems.

Reconstruction of the granular pavement was considered a viable option and comprised removing the existing pavement down to subgrade level and construction of a new granular pavement. Historically, this was Council's preferred option.

It was determined that geogrids may be useful at this site as it is considered in areas of known weak subgrade or where traditional treatments such as granular reconstruction will result in excessive depths and potentially cause service conflicts. It was decided in this case the use of geogrids could allow deletion of the lower sub-base layer.

The recommended design comprised boxing out and reconstruction of a new granular pavement, including the use of geogrids with asphalt surfacing. Rectification of pavement drainage and replacement of all kerb and channel was also undertaken.

5.3.2 Pavement Design and Construction

The design adopted comprised boxing out and reconstruction of a new pavement with the use of a geogrid. The recommended design comprised three sections all surfaced with a 30mm AC s wearing course over a 250mm to 385mm thick granular pavement on 30/30 Combi-grid.

The design traffic loading was 3.1×10^5 ESAs. For design purposes the value was rounded up to 4×10^5 ESAs. This value was considered appropriate when compared with similar roads.

Design CBR values of 2%, 3% and 10% was adopted based on subsurface investigation and laboratory testing.

The following pavement configuration was adopted:

CBR 2%:

- Surfacing 30mm BCC2
- Base course 125mm Type 2.1 (Min CBR 80%)
- Upper Sub Base 100mm Type 2.3 (Min CBR 45%)
- Lower Sub Base 160mm Type 2.3 (Min CBR 45%)
- 30/30 Combi-grid with 500mm minimum overlap

CBR 3%:

- Surfacing 30mm BCC2
- Base course 125mm Type 2.1 (Min CBR 80%)
- Upper Sub Base 100mm Type 2.3 (Min CBR 45%)
- Lower Sub Base 110mm Type 2.3 (Min CBR 45%)
- 30/30 Combi-grid with 500mm minimum overlap

CBR 10%:

- Surfacing 30mm BCC2
- Base course 125mm Type 2.1 (Min CBR 80%)
- Upper Sub Base 100mm Type 2.3 (Min CBR 45%)

The pre-existing drainage system was also found to be inadequate and rectification was necessary for the reconstructed pavement to perform adequately. Subsoil drains were installed along both sides of the road and kerb and channel renewed, extending the full length of the site. Pavers were also removed and replaced with the applicable design outlined above.

The program of works were as follows:

- Box out and place 30/30 Combi-grid;
- Reconstruct kerb and channel and subsoil drains;
- Reconstruct 250mm to 385mm of granular pavement; and
- 10mm primer seal ‘SURFIX PS’ PME binder and 30mm BCC2.

As Council had not used the specified 30/30 combi-grid previously, a trial section was undertaken first to ensure compaction and constructability on the section with a subgrade CBR of 2%. Shown in Figure 37 below.

Figure 37: Construction Processes of Lyon St, Dicky Beach

5.3.3 Investigation Results

Figure 38 demonstrates a significant improvement in maximum deflection at two (2) of the three (3) locations tested in 2012. Test results within the first 75m are not relevant as this section was only resurfaced due to low 2012 maximum deflections and existing pavement conditions. Results from the remaining two (2) locations suggest a significant improvement in pavement strength. Extensive subsoil drainage works may have assisted in drying out the previously saturated subgrade. Average maximum deflection results improved from 1.14mm prior to pavement rehabilitation works in 2012 to 0.61mm post construction works in 2014.

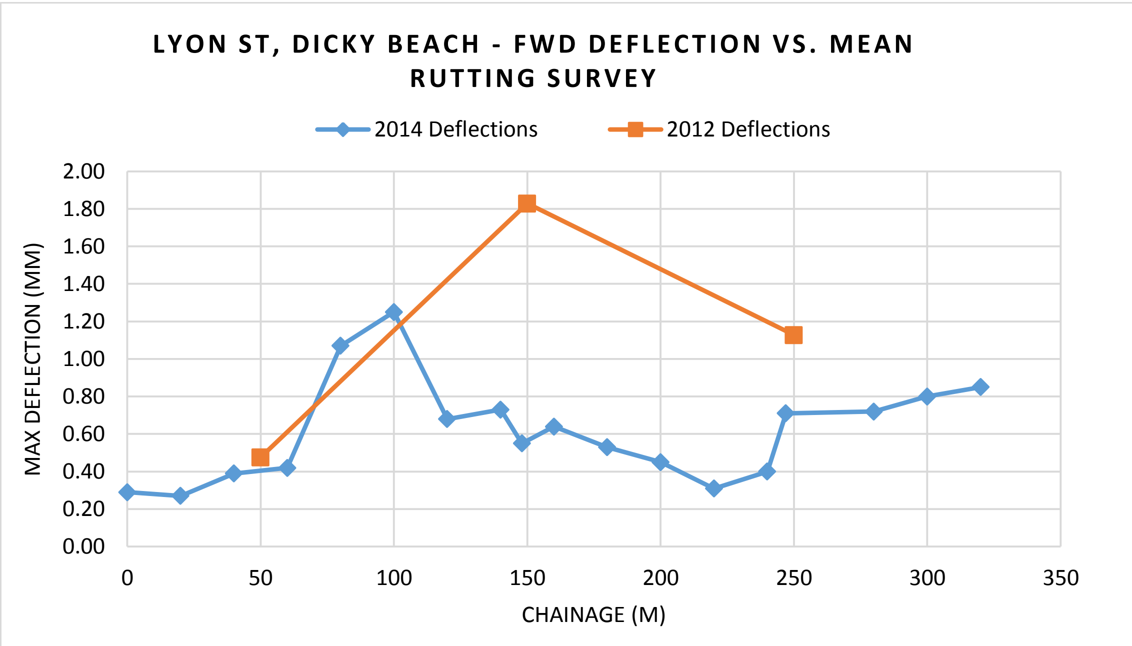


Figure 38: Lyon St, Dicky Beach - 2012 and 2014 Maximum Deflection

Lyon Street, Dicky Beach has an average relationship between CF values and maximum deflection of 28% for the length of the project, as demonstrated in Figure 39. Within the acceptable range for granular pavements of 25% to 35%, as outlined by Austroads (2009). Two (2) locations have a higher than desired relationship, returning values of 38% and 42%. These locations are however accompanied by CF values of 0.15mm and 0.13mm respectively which suggesting a stiff pavement and no further investigation is required.

Evaluating the project on CF values, 83% of Lyon Street returned results indicating high stiffness and strength in the pavement layers, with 17% between 0.2mm and 0.4mm. Below the suggested 0.4mm representing low strength and pavement stiffness as outlined by TMR (2012).

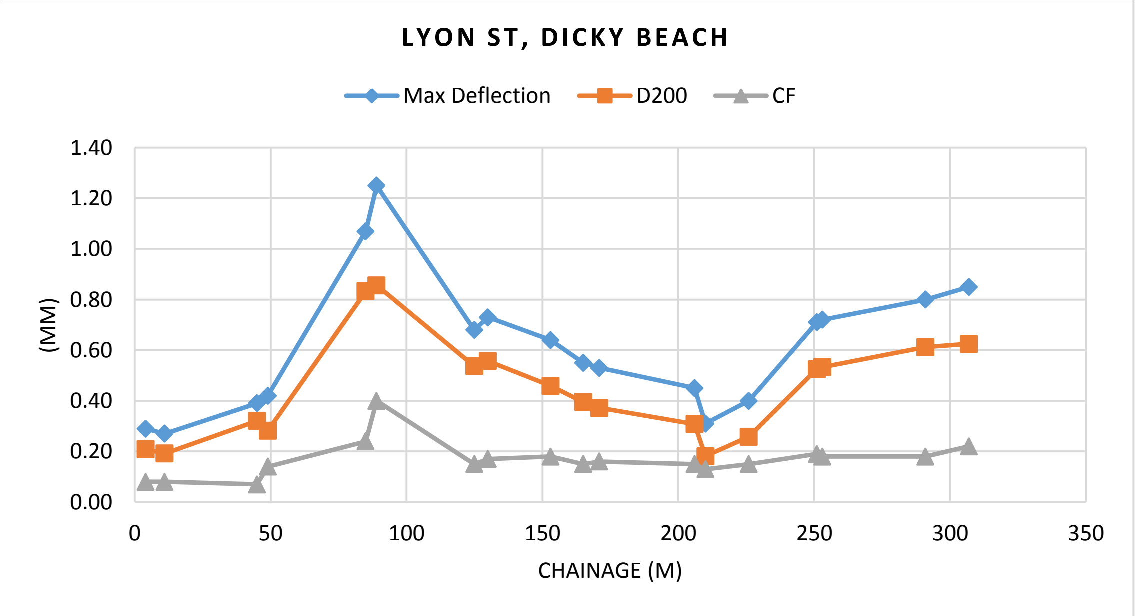


Figure 39: Lyon St, Dicky Beach - Curvature Function

Results shown in Figure 40 indicate an average rutting of 4.78mm and 4.42mm for the right and left outer wheel paths respectively. Reporting requirements of severity and extent of outer wheel path rutting is outlined in Section 4.4.3. In accordance with this, Lyon Street displays rutting with severity ranges from 0-5mm, 5-10mm and 10-15mm extending for 65%, 32% and 3% respectively. As outlined, rutting less than 10mm is not considered structurally significant. Two (2) locations are exhibiting rutting in the high range between 10-15mm and may be susceptible to water ponding and a potential safety issue. Given the road hierarchy of Lyon Street, it is not considered a safety concern.

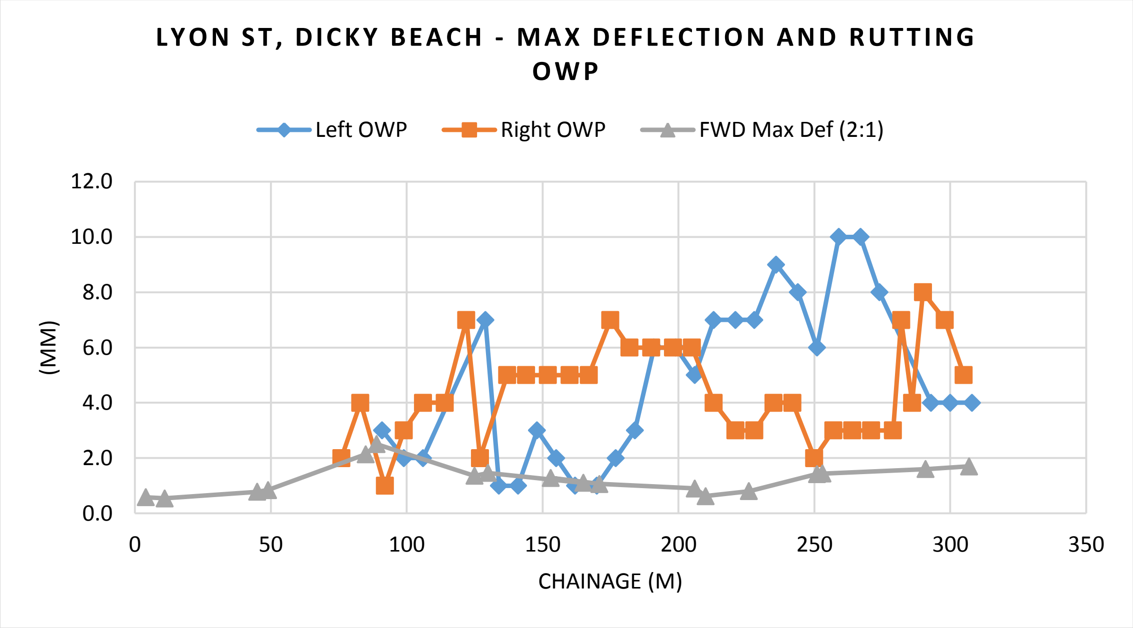


Figure 40: Lyon St, Dicky Beach - Rutting and Maximum Deflection

The results from testing indicate the use of 30/30 combi-grid has assisted in strengthening the subgrade and improving pavement strength, evident with the significant improvement in maximum deflection and low CF values. The severity of rutting in a few locations is of concern and could be due to a number of factors including moisture ingress, quality of materials and work methods used. During construction significant rainfall was experienced and may have contributed to isolated areas of higher than desirable moisture content within the pavement.

5.4 Bunya Road – Bridges

Bunya Road, Bridges between Burtons Road and Monak Road was considered due for pavement rehabilitation in 2012. Bunya Road originally consisted of several different sections with varying existing pavement compositions. Low lying sections consisted of a concrete pavement with gravel shoulders which had been overlaid with asphalt. The remaining sections were comprised of granular sections which varied between 80mm to 390mm of gravel. The seal thickness varied between 15-45mm with some locations identifying what appeared to be patches and shoulder repairs varying between 70mm to 160mm.

The subgrade in this area was generally average, varying from high plasticity silty sandy clay, silty sand and medium to high plasticity sandy gravelly clays. Geotechnical investigations indicated that all test pits include significant moisture. Seventeen (17) geotechnical test holes were undertaken, determining soaked CBR's from 5% to 14% and with one location as low as 3%.

During the site inspection it was evident that the pavement had undergone extensive patching however, due to the rural nature of the road there was scope to raise the finished surface levels. During the site visit an elderly Roadtek employee stopped to enquire having worked on this section of road in a previous life. He advised that all low lying areas were constructed in 150-200mm of concrete with granular pavements being used in the areas of better ground. Observations on the ground supported this.

Rutting and failures along the outer wheel path and shoulder of the pavement suggested the road had been widened over a period of time. Separate treatments were required to provide a uniform surface capable of satisfying the 20 year design life. Typical pavement and exposed batter conditions prior to pavement rehabilitation is shown below.

Figure 41: Bunya Rd, Bridges - Site Photographs

5.4.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- Granular Pavement;
- In-situ Stabilisation;
- Subgrade Improvement and Reconstruct; and
- Widen and Overlay.

According to the TMR pavement design manual methods it was determined the entire length of the road was deficient in gravel for normal design standard (20mm rut at subgrade level). This deficiency ranged from 125mm to 310mm with the average being approximately 210mm. The gravel thickness required for a new granular pavement varied from 290mm to 520mm over the length of the entire project, with the mean thickness being 380mm. Preliminary estimates priced this option at $3,200,000.

Considering the TMR pavement design method second design standard, allowing a 30mm rut at subgrade level at the end of the design life, the mean pavement deficiency was 160mm.

Due to the highly variable base thicknesses, the in-situ concrete and narrow shoulders that may have been eligible for treatment combinations of cement treated base and in-situ stabilisation was not recommended.

Subgrade improvement and reconstruction was considered in the form of stabilisation the subgrade followed by full base course replacement, however targeting a subgrade improvement CBR of 15% would have resulted in a 100mm average reduction in thickness compared with the granular replacement options. The stabilisation costs outweighed the cost of the gravel and potentially posed higher financial and performance risk. This option would also have provided significant inconvenience to traffic during construction and for these reasons this approach was not considered.

Widening and overlay of the concrete pavement in asphalt was a possible solution achieved through:

- Boxing out and replacing shoulders with the required thickness of asphalt;
- Overlaying the asphalt / concrete joint with strips of glass-grid to minimise cracking at the joint between differing materials; and
- Overlay full width (Shoulder and concrete) with asphalt.

This option was not adopted due to the risk of exposing poor subgrade and requiring a working platform for asphalt compaction. This option was estimated at $1,315,000.

The recommended treatment included overlaying the existing pavement with a granular material. It was thought that this treatment would offer the most convenient low risk option. Granular overlay of a rigid pavement is not always favoured however, this configuration is recognised in the TMR pavement design manual. The recommended overlay option was estimated at $910,000.

5.4.2 Pavement Design and Construction

The design consisted of varying depths of granular overlay, subgrade replacement and treatment of concrete / asphalt joints with Bitac crack sealing tape. The recommended design comprised two sections all surfaced with a two coat bitumen seal over a 100mm to 140mm thick granular overlay.

The design traffic loading was 1.1×10^6 ESAs with 8% commercial vehicles and 3% compound growth. This value was considered appropriate when compared with similar roads. Design CBR values of 5% to 14% and 3% were adopted based on subsurface investigation and laboratory testing.

The following pavement configuration was adopted:

Section 1:

- Surfacing 2-coat bitumen seal
- Base course 140mm Type 2.1 (Min CBR 80%)
- Existing Pavement material

Section 2:

- Surfacing 2-coat bitumen seal
- Base course 100mm Type 2.1 (Min CBR 80%)
- Existing Pavement material

Shoulder Replacement and Patching:

- Surfacing 2-coat bitumen seal
- Base course 100mm Type 2.1 (Min CBR 80%)
- Sub base 200mm Type 2.3 (Min CBR 45%)

The program of works were as follows:

- Box out and reconstruct shoulders where required;
- Treat expansion joints / cracks in existing asphalt / concrete sections;
- Overlay existing pavement with 100mm to 140mm of Type 2.1 material; and
- 2 coat bitumen seal.

Typical photos of the construction stages are seen in Figure 42.

Figure 42: Stages of Construction - Bunya Rd, Bridges

5.4.3 Investigation Results

Figure 43 outlines the improvement in maximum deflections from testing conducted prior to pavement rehabilitation works in 2012 and post construction in 2014. Results suggest areas of significant improvement and sections of no improvement. Average maximum deflection results were 0.78mm and 0.48mm for 2012 and 2014 testing respectively, an improvement of approximately 40%.

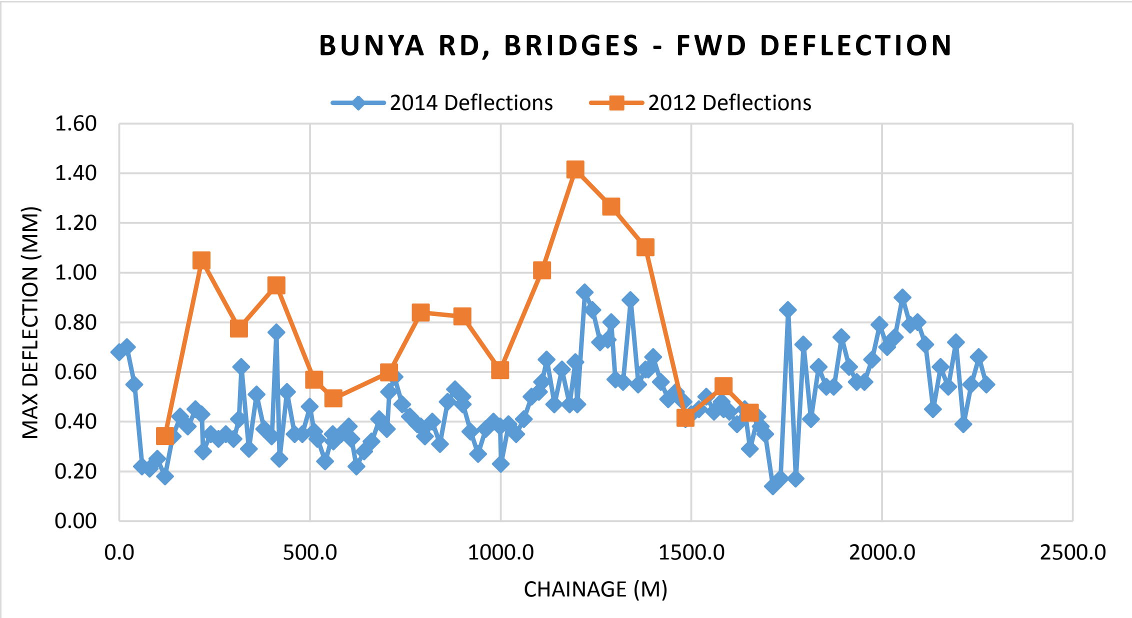


Figure 43: Bunya Rd, Bridges - 2012 and 2014 Maximum Deflections

Bunya Road, Bridges has returned an average relationship of 27% between CF values and maximum deflection for the length of the project. Acceptable under criteria outlined in Austroads (2009) suggesting CF values are likely to be 25% to 35% of the maximum deflection for granular pavements.

Evaluating for severity of CF values results in 89% of the project with a CF value between 0.0–0.2mm, representing a stiff, sound pavement. 10% of the project returned values between 0.2 and 0.4mm which is acceptable, shown in Figure 44. One (1) location is exhibiting concerning characteristics, with a CF value greater than 0.4mm, indicating a pavement lacking stiffness. Further analysis determined this testing was undertaken on the preceding road section to where the pavement rehabilitation works commenced. Therefore, this test location is excluded from further discussion.

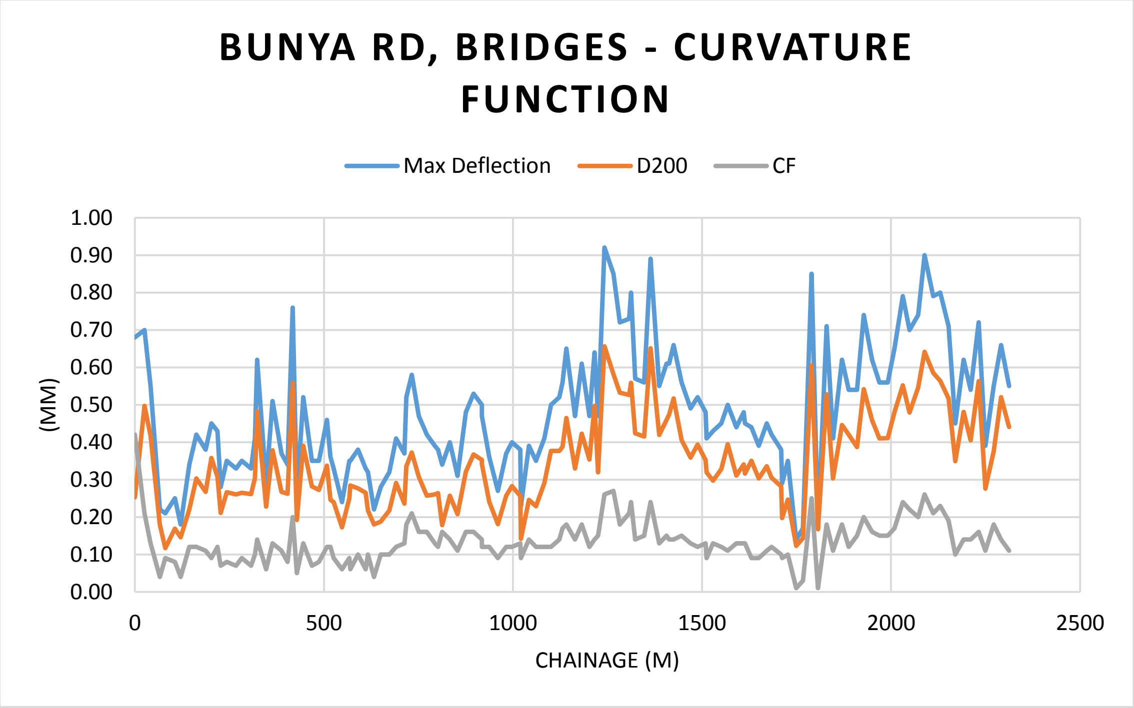


Figure 44: Bunya Rd, Bridges - Curvature Function

Results shown in Figure 45 indicate an average rutting of 4.30mm and 5.17mm for the right and left outer wheel paths respectively. Between CH1300 and CH1350 results indicate high rutting in both directions. This section is located between two large agricultural property driveways and based on a further site inspection on 10 October 2014, it was evident results may have been effected by foreign material located on the road due to the use of these driveways. Therefore, these results have been excluded from further discussion.

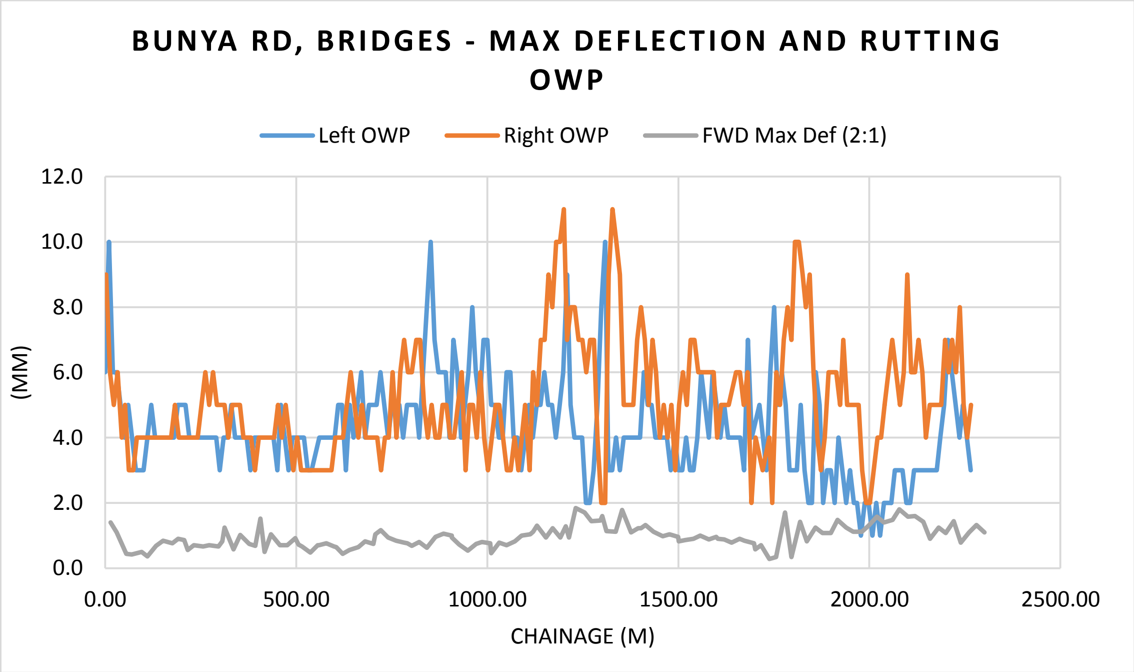


Figure 45: Bunya Rd, Bridges - Rutting and Maximum Deflection

Austroads (2007) reporting requirements outline, Bunya Road displays rutting with severity ranges of 0-5mm, 5-10mm and 10-15mm extending for 75%, 24% and 1% of the project respectively. As rutting less than 10mm is not regarded as significant, only five (5) locations represent areas of concern. Overall the results indicate adequate pavement strength and depth. Significant rainfall and inundation of sections of this project during construction may have contributed to isolated areas of high rutting and weak pavement strength. Between Ch1850.0 and CH1920.0 experienced several periods of inundation during these events, accounting for three (3) locations with high severity rutting.

5.5 Point Cartwright Drive – Buddina

The section identified for pavement rehabilitation and consequent investigation on Point Cartwright Drv was located between the Nicklin Way and Orana Street. Initially the pavement design considered subgrade conditions, existing pavement fatigue / condition, constraints imposed by existing infrastructure and traffic flow over the site.

Pavement rehabilitation options that were considered included deep lift asphalt, flexible pavements, asphalt overlay and semi-rigid pavements. Rigid pavements were neglected due to the requirement that council would need to close the site for curing.

A detailed inspection along the subject section of road revealed block cracking, indicating that the existing pavement had been previously stabilised. It was also noted that due to the urban nature with central islands and kerbing, the option to overlay and raise the finished surface level was limited. It was noted that sections of kerb and channel was stained from prolonged groundwater seepage spilling over the kerb. This supported that the ground conditions are wet for extended periods of time.

Four (4) geotechnical test holes were undertaken, which determined the following:

- The subgrade in this area is generally good varying from silty sand, sandy clayey gravel to sand. All test pits indicated moisture in the upper levels with in-situ subgrade moisture content between 8% - 17%. Water table was observed at depths of 0.7 to 0.85m;
- Soaked CBR’s were determined at 20% to 30%;
- The existing pavement was gravel with asphalt thicknesses varying between 45mm to 80mm; and
- Existing gravel base varied from 155mm to 320mm.

The rutting evident in this section of roadway suggested the pavement was at or close to its theoretical failure and the end of its useful life. The rutting observed throughout the length of the works were typically <10mm (say 70%) with balance areas measuring up to 30mm in a 1200mm straight edge (remaining 30%). Photos from the detailed inspection are shown in Figure 46.

Figure 46: Point Cartwright Drv, Buddina - Site Photos

5.5.1 Treatment Options Considered

Council’s preferred rehabilitation options for this section were:

- Granular Pavement;
- Deep Lift Asphalt; and
- CTB Combinations including (in-situ stabilisation).

Given the functional class of the road a granular pavement of 230mm to 250mm was recommended to satisfy the TMR design method using the second design standard. The construction cost of this option was estimated at $720,000.

Deep lift asphalt was considered due to the function class of the road. 190mm of pavement was recommended for the straight section of carriageway and increased to 210mm in the roundabout area. Forecasted construction costs of this option were $1,040,000.

During the detailed inspection it was determined that the pavement had been previously cement stabilised which precludes the effective re-stabilisation of this pavement. Notwithstanding there was also insufficient gravel to consider stabilisation of the pavement based on traffic and design ESA's.

Granular was considered a low performance risk and offered future opportunities for stabilisation however there would have been more interruption with this option compared to deep lift pavement options. Consideration was specifically given to traffic management on the sub-arterial road, local school, both shopping centres and the Translink bus station. Given the time of construction required and the increased risk should wet weather be encountered the preferred solution was deep lift asphalt.

5.5.2 Pavement Design and Construction

The design consisted of varying depths of deep lift asphalt. The recommended design comprised two sections of 190mm and 210mm pavement sections.

The design traffic loading was adopted from actual traffic measured on site. Accordingly this figure was used and the higher figure was rounded up to 3×10^6 ESA's which was then adopted for design purposes.

Design CBR values of 20% to 30% were adopted based on subsurface investigation and laboratory testing.

The following pavement configuration was adopted:

Straight sections:

- Surfacing		2x 45mm DG14 layers
- Base course		100mm DG20 base layer
- Existing Pavement material

Roundabout:

- Surfacing		2x 45mm DG14 layers
- Base course		120mm DG20 base layer
- Existing Pavement material

The program of works were as follows:

- Box out and place DG20 sub base; and
- Place DG14 wearing surface.

5.5.3 Investigation Results

Figure 47 demonstrates inconsistent results for maximum deflections from testing prior to the pavement rehabilitation in 2012 and post construction in 2014. Maximum deflections are well within Sunshine Coast Council's acceptable range with an average maximum deflection of 0.26mm. An improvement from 0.32mm tested in 2012. This indicates the resulting failure of the road was due to the existing pavement reaching the end of its useful life on a sound subgrade.

Point Cartwright Drive is a trunk collector experiencing high traffic volumes. This area is also experiencing a lot of development associated with the adjoining shopping centres and redevelopment of housing blocks within the area. Results from maximum deflection, and confirmed with geotechnical testing suggest adequate subgrade strength and rehabilitation of the pavement layers was required.

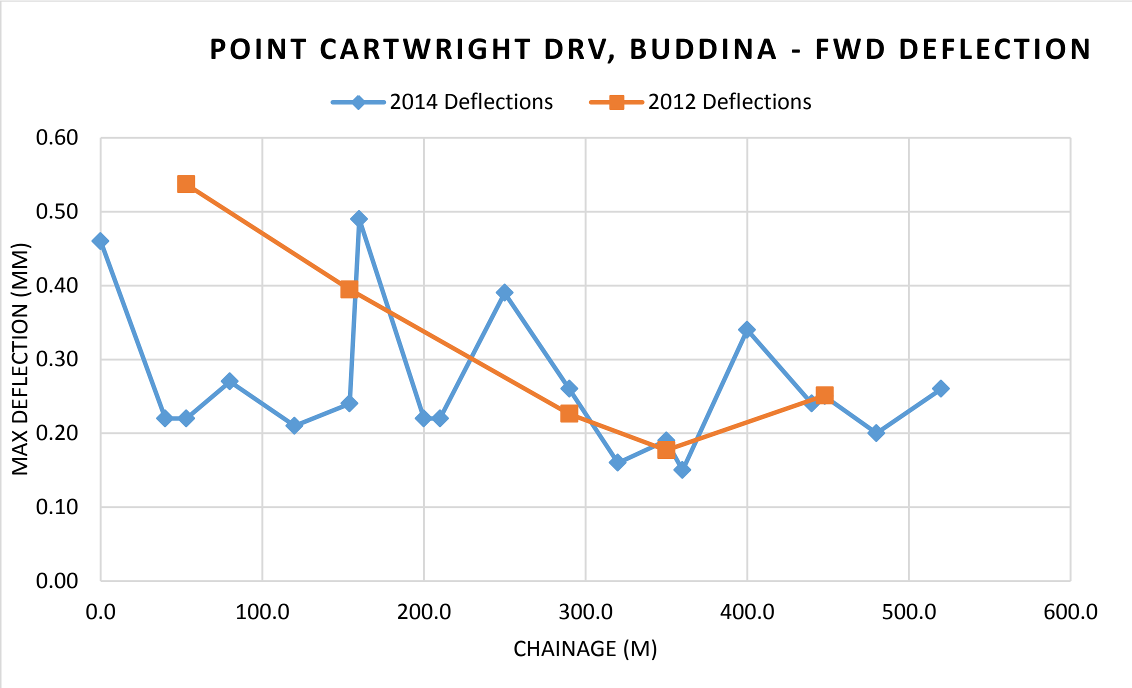


Figure 47: Point Cartwright Drv, Buddina - 2012 and 2014 Maximum Deflection

Deflection testing has a much more limited application to rigid pavements (Austroads, 2009). Notwithstanding, Point Cartwright Drive, Buddina has an average relationship between CF values and maximum deflection of 26% with six (6) locations which returned high results ranging between 38% and 64%.

Further investigation into the construction processes used is required to identify areas of pavement joins and if these results are related to paving patterns or failures due to underground infrastructure, common in areas of ageing infrastructure and sandy sub grade materials.

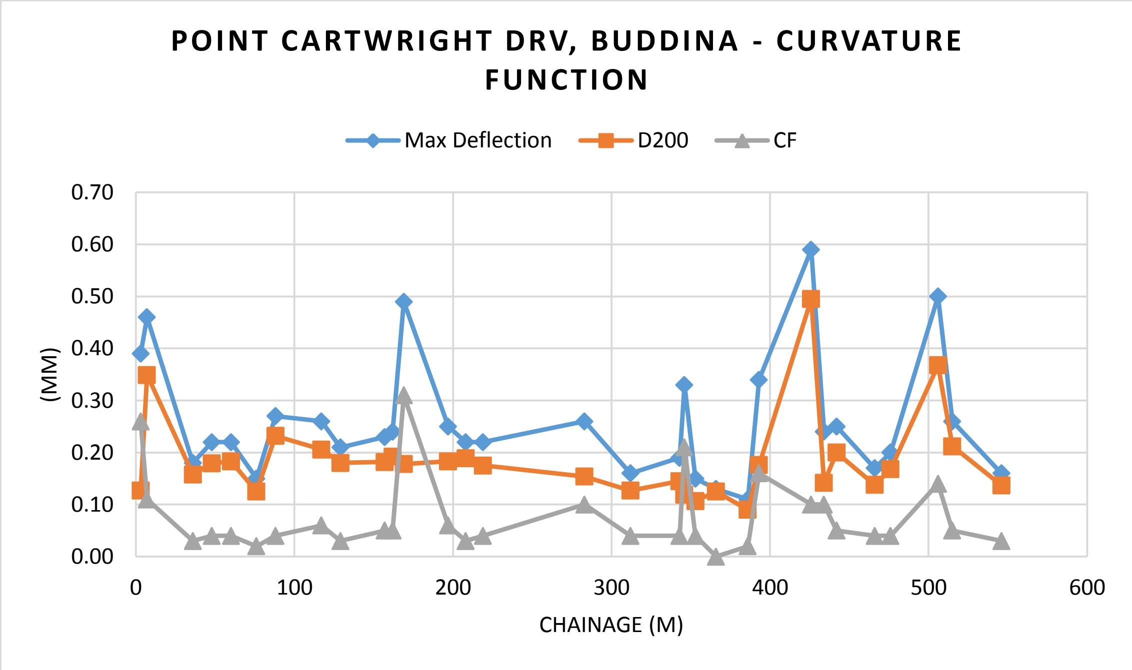


Figure 48: Point Cartwright Drv, Buddina - Curvature Function

Figure 49 indicates no correlation between rutting and maximum deflection. Indicating rutting may be a result of early trafficking. Results indicate an average of 4.48mm and 4.62mm rutting for the right and left outer wheel paths respectively. Reporting the severity and extent of outer wheel rutting in accordance with Section 4.3.3 results in severity ranges of 0-5mm and 5-10mm with 88% and 12% of the project respectively. Indicating no areas of structural concern.

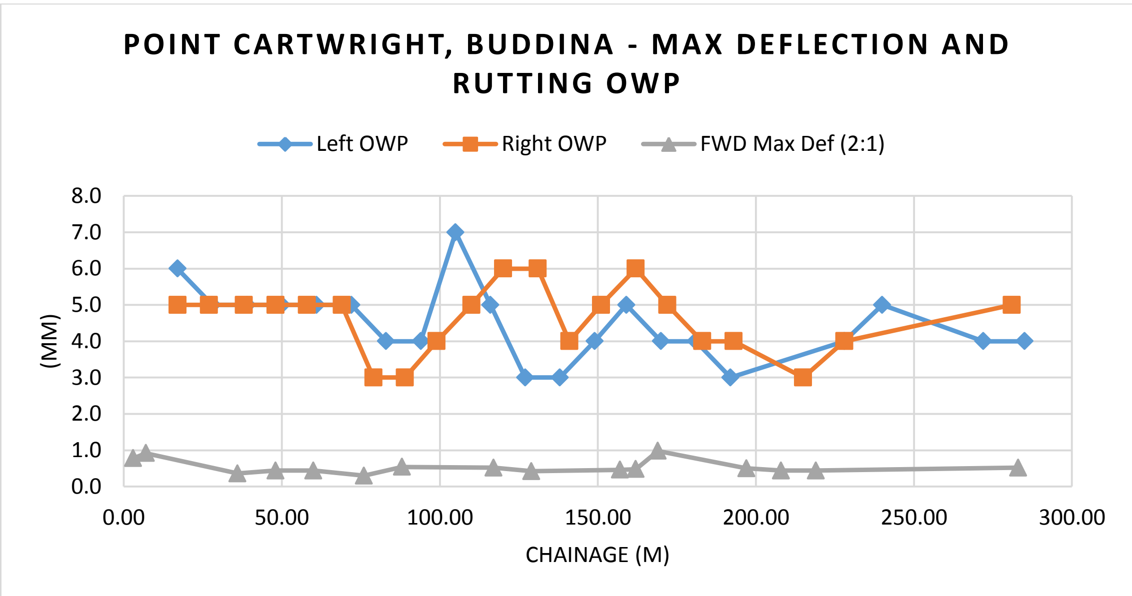


Figure 49: Point Cartwright Drv, Buddina - Rutting and Maximum Deflection

The results above demonstrate the lowest relationship between deflection measurements and rutting results. Consistently high rutting is evident within the first 75m, approaching a major Transport and Main Roads Queensland intersection. Previous experience in deep lift asphalt pavements suggests rutting may be a result of this pavement rehabilitation option due to early trafficking of the pavement and higher order roads and higher stress areas used i.e. roundabouts and major intersections. Sunshine Coast Council has recently specified Polymer Modified asphalts be used on all roundabouts and major intersections to alleviate this problem.

5.6 Glenview Road – Glenview

Glenview Road, Glenview between Clinton Court and Isambert was identified for pavement rehabilitation in 2012 and completed in 2013. Design considerations for this carriageway included granular replacement, overlay and cement treated base treatments. Site inspections prior to rehabilitation works outlined the extent of patching and failure types within the section of Glenview Rd. There was evidence of some patching using cement, evident on the ground. The pavement had also undergone significant patching, particularly at the outer wheel path. Initial suggestions included raising the finished pavement levels, considering existing driveways and transitions.

Despite the poor condition of the pre-existing pavement the rutting in the pavement typically throughout the length of the works was around 10mm (approximately 80% of the project) with the balance up to 40mm with a 1200mm straight edge. Low rutting but poor condition suggested a reasonable subgrade and lack of adequate base material.

Three (3) test holes were undertaken. The subgrade in this area is generally good being predominantly clayey sand, silty sand and pockets of sandy clay. All test pits indicated moisture. Freshwater crayfish holes were observed in one section of the table drain, indicating regular moisture. Soaked CBR’s were determine at 25%, 25% and 10%. The existing pavement was gravel with a seal thickness that varied between 20mm to 40mm. The existing pavement gravel base varied from 95mm to 190mm of sandy gravel. Photos from the site inspection are shown in Figure 50 below.

Figure 50: Glenview Rd, Glenview - Site Photos

5.6.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- Granular Pavement;
- Deep Lift Asphalt;
- CTB Combinations; and
- In-situ Stabilisation + 125mm Granular Overlay.

Granular was considered and the design recommendations included granular pavement replacement at depths of 200mm to 275mm. This option also provided the best method of increasing the existing pavement width and achieving a paved shoulder. This was a significant advantage given the pre-existing shoulders which consisted of minimal gravel and largely comprised of loam and grass. The construction cost of this option was estimated at $249,000.

Deep lift asphalt is not generally considered for rural locations however, was proposed at this location due to the moist ground conditions. It was dismissed as there would generally by no base course gravel remaining as a working platform. Furthermore, the cost of this option was estimated at $375,000.

In-situ stabilisation was excluded as there was insufficient base gravel to consider stabilising the existing pavement. However, the existing base gravel was sufficient to consider cement stabilising and consequently overlaying the gravel. It was determined that stabilisation to a depth of 150mm with 125mm granular overlay would be required. Improvement of the carriageway width with this option would have been messy in construction, combine that with mobilising stabilising plant and machinery for a very short length made this option unattractive.

Consideration was given to through traffic when planning the works and rehabilitation was undertaken in during the drier months due to the in-situ ground conditions. Granular replacement was considered the low risk option and offered future opportunities for stabilisation. The design was modified slightly to achieve the required design depth by boxing out and finishing 75mm higher than existing. Deep lift asphalt was dismissed due to its cost and the risk of exposing unsuitable subgrade conditions for asphalt placement and compaction machinery.

5.6.2 Pavement Design and Construction

A ‘fit for purpose’ design option consisted of granular replacement. The recommended design comprised a uniform 250mm Type 2.1 granular pavement replacement. Minor drainage works were included as part of the works. Re-establishment of table drains on both sides of the road, including the placement of subsoil drains at the interface of the shoulder and batter. Additional cross road drainage was installed to facilitate draining of the table drain on the Northern side of Glenview Road.

The design traffic loading was adopted from actual traffic measured on site. Accordingly this figure was 1.2 x 10^6 ESA’s based on 9.5% commercial vehicles and an assumed growth of 4% for 20 years.

Design CBR values of 25% and 10% were adopted based on subsurface investigation and laboratory testing.

The following pavement configuration was adopted:

Straight sections:

- Surfacing 2 coat bitumen seal
- Base course 250mm Type 2.1 material

The program of works were as follows:

- Box out and place Type 2.1 base course
- Complete 2 coat bitumen seal

Figure 51: Construction Photos - Glenview Rd, Glenview

5.6.3 Investigation Results

Figure 52 demonstrates an improvement in average maximum deflections from prior to pavement rehabilitation works in 2012 and post construction works in 2014 from 1.20mm to 0.81mm. Notwithstanding, several test locations exhibit higher than desirable maximum deflections. Maximum deflections of 1.27mm and 1.32mm at CH120 and CH160 respectively are areas of concern, resulting in deflections above Sunshine Coast Council intervention level for resurfacing.

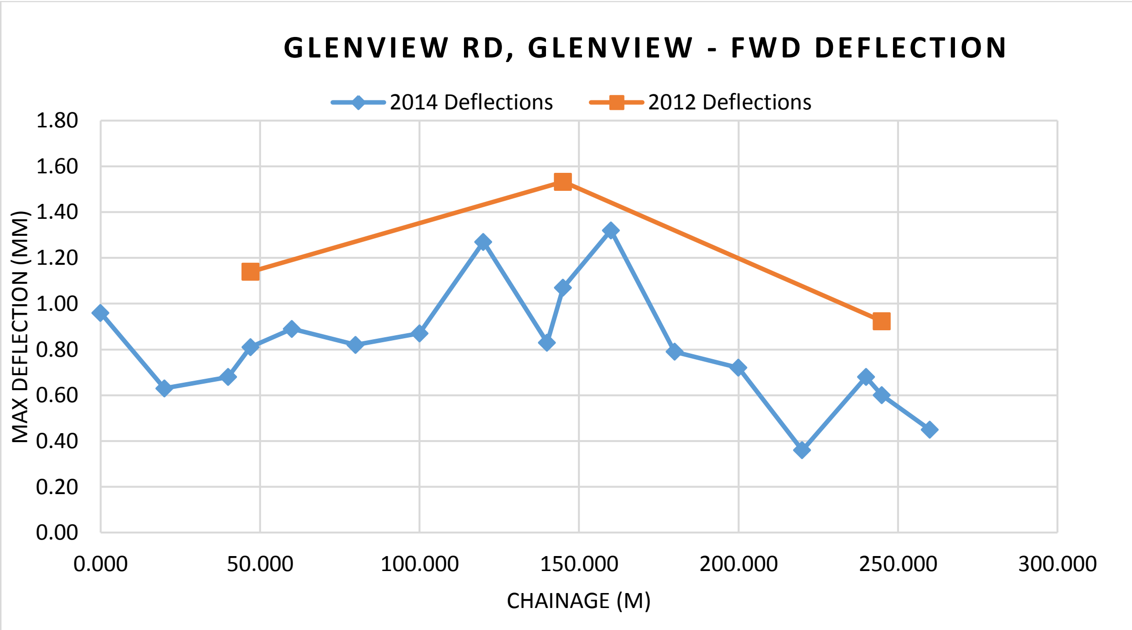


Figure 52: Glenview Rd, Glenview - 2012 and 2014 Maximum Deflections

Austroads (2009) suggests high CF values may indicate low stiffness in the upper pavement layers. Glenview Rd, Glenview has a slightly high relationship between CF values and maximum deflection of 32% for the length of the project, outlined in Figure 53. In accordance with Austroads (2009) guidelines for granular pavements where the CF values are likely to be 25% to 35% of the maximum deflection.

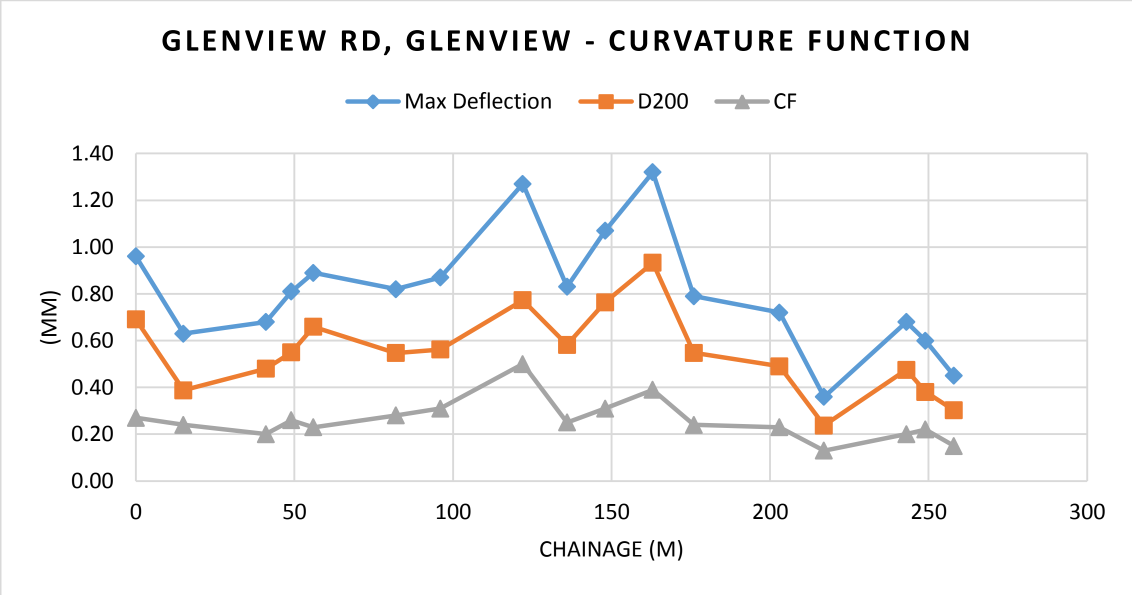


Figure 53: Glenview Rd, Glenview - Curvature Function

The average CF value for this section of pavement rehabilitation is 0.26mm. within the lower range of values, suggesting a relatively stiff pavement. One (1) location at CH122.0 demonstrates a high CF value of 0.5. Further investigation and discussions with construction crews indicate an area of unsuitable subgrade at this approximate location which was treated at the time of construction. Results may indicate an adjoining section where subgrade replacement was not undertaken.

Results indicate an average rutting of 7.19mm and 6.58mm for the right and left outer wheel paths respectively. The highest average rutting of all pavement rehabilitation projects tested as part of this research project. Figure 54 demonstrates that between CH100.0 and CH175.0 it can be seen that a small relationship between maximum deflection and rutting is evident. In this location TMR (2012) suggests the rutting may be caused due to insufficient pavement strength and deformation of the subgrade. Beyond CH200.0 there appears to be no relationship between pavement deflection and rutting and defects could be attributed to poor quality pavement materials, moisture ingress or early trafficking post construction.

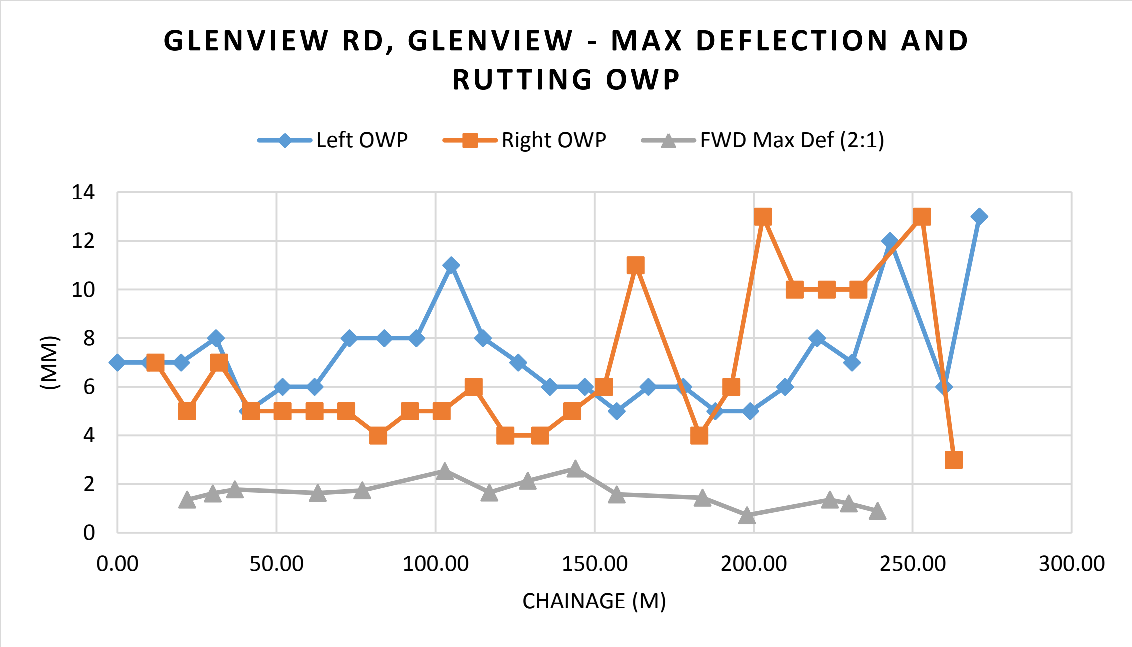


Figure 54: Glenview Rd, Glenview - Rutting and Maximum Deflection

Reporting requirements of severity and extent of outer wheel path rutting displays results for Glenview Road, Glenview in several categories of severity. This section demonstrated 34%, 48% and 18% of 0-5mm, 5-10mm and 10-15mm rutting severity. As there is no evidence of shoving along Glenview Road, the results characterise a pavement where the pavement layers are too thin to protect the subgrade.

5.7 Buderim Street - Currimundi

Buderim Street, Currimundi is located on notoriously weak subgrade foundations. The section which underwent pavement rehabilitation in 2012 is located between Currimundi Road and Coonowrin Street. Pavement rehabilitation options that were considered included deep lift asphalt, flexible pavements, asphalt overlay and semi-rigid pavements.

Prior to the pavement rehabilitation the pavement was displaying extensive block cracking and crack sealing indicating that the existing pavement had previously been stabilised. Overlay options were dismissed as the extent of existing central islands and kerb and channel along each side would limit the finish surface level.

The existing pavement was showing signs of recent failures which had required localised patching. There was evidence of pumping of fines through cracks in the asphalt; even though the cracks had been crack sealed previously. It was also noted that the existing roundabout created a ‘pinch point’ for cyclists which had been a source of complaint from bicycle user groups for a number of years and was to be rectified as part of the works.

Six (6) test holes were undertaken throughout the section and the subgrade material generally consisted of silty sand, sandy gravels, silty gravelly sand and sandy clays. Most of the boreholes were moist with moisture content as high as 15%. In-situ CBR’s soaked strengths between 9% and 50%. The existing pavement consisted of 140mm to 340mm of granular material with between 30mm and 60mm of asphalt wearing surface.

The calculated design life by analysing the deflection data was predicted to be in the order of 3 to 5 years. Data sourced from council’s Pavement Management System identified that the pavement was previously rehabilitated in 1995/96 and supports the residual life mentioned above.

Rutting which existed in the pavement suggested otherwise, observed throughout the length of the works. Ruts measured along the project were typically 10-15mm (approximately 75% of the project) with a reasonable number of areas measuring 15-20mm with a 1200mm straight edge (15-20%). However. Some areas had ruts in the 30-45mm range (remaining 5-10%). The rutting evident suggested the section was at or close to theoretical failure shown in Figure 55 below.

Figure 55: Buderim St, Currimundi - Site Photos

5.7.1 Treatment Options Considered

Council's preferred rehabilitation options for this section were:

- Granular Pavement;
- Deep Lift Asphalt;
- CTB Combinations; and
- Granular Overlay over existing CTB.

Given the performance of the existing pavement and the functional class of the road it was recommended that the second design standard of 30mm rutting depth at the subgrade is appropriate for this road. Therefore, it was determined the minimum thickness required to achieve this standard was 300mm of granular pavement constructed over the subject site, with an increase to 480mm at the roundabout.

Given the nature of the subgrade in the area, it was predicted that during construction there would likely be further areas of unsuitable subgrade encountered and it was recommended that all unsuitable areas be removed and replaced with CBR10 material in accordance with the TMR Pavement Design Manual. Based on the subgrade results of CBR 2% found on the Cooroy Street leg of the roundabout a minimum of 200mm of replacement material was allowed for. This option was costed at $970,000.

The required deep lift asphalt pavement to achieve the proposed standard was 200mm over the site, increasing to 235mm at the roundabout. In order to place the asphalt it was suggested a minimum 150mm thick CBR 10% working platform was required if poor ground conditions was encountered.

Further areas of unsuitable subgrade was expected during construction and as with the granular option above a minimum of 200mm of CBR 10% subgrade replacement was suggested.

To assist in cost mitigation it was decided that the through carriageways only would be replaced with full depth asphalt, with the parking lanes receiving a nominal 50mm asphalt overlay. Forecasted construction costs for this option were estimated at $1,040,000.

In-situ stabilisation was dismissed as a viable option due to the existing pavement indicating signs that it had previously been cement stabilised which precludes the effective re-stabilising of the pavement.

The fourth option considered prior to undertaking the works was the option to mill out 125mm of existing material and replace it with a 125mm granular base plus 50mm asphalt wearing course over the residual cement treated pavement. The proposed design involved a finished surface level 50mm higher than the existing levels due to the granular base layer, existing sub base block cracking was not expected to reflect in the upper asphalt layer. However, this option failed due to fatigue cracking of the asphalt at around 6-7 years. Therefore, this option would have required replacing 3 times within the intended timeframe and consequently become more expensive than the deep lift option, on the asphalt and traffic control costs alone.

Granular pavement was determined a low performance risk; offering future rehabilitation treatment opportunities, however, the disruptions to the traffic, local schools and businesses would have been far greater than deep lift asphalt. Deep lift was the recommended treatment due to its convenience and lessor disruptions to the public.

5.7.2 Pavement Design and Construction

The design consisted of varying depths of deep lift asphalt. The recommended design comprised three sections of 180mm, 200mm and 235mm pavement sections.

The design traffic loading was adopted from actual traffic measured on site. Accordingly this figure was used and the higher figure was rounded up to 2×10^6 ESA's which was then adopted for design purposes. Design CBR values of 10% was adopted based on subsurface investigation and laboratory testing although, it was a lot less in some locations as discovered throughout the project.

The following pavement configurations were adopted:

180mm section:

- Surfacing 50mm DG14
- Base course 70mm DG20
- Sub base 60mm DG20

200mm section:

- Surfacing 50mm DG14
- Base course 70mm DG20
- Sub base 80mm DG20

Roundabout (235mm):

- Surfacing 50mm DG14
- Base course 70mm DG20
- Upper sub base 60mm DG20
- Lower sub base 55mm DG20

The project was not constructed exactly as outlined in the above pavement configurations. Sub base layers were combined and placed in one layer for the roundabout section to provide a sufficient platform to compact the base layer on. In the roundabout section the sub base layer was placed via the use of bobcats to limit traffic and construction loads on the subgrade.

Figure 56: Buderim St, Currimundi - Construction Photos of Unsuitable Subgrade Locations

The program of works were as follows:

- Box out and place DG20 sub base material
- Place DG20 base material
- Place DG14 wearing surface
- Replace any unsuitable subgrade material (where required).

During this project several construction problems were encountered due to unsuitable subgrade locations. Additional geotechnical testing was undertaken at these locations and the relevant CBR results were in accordance with previous laboratory results. Notwithstanding, it appeared the collapsible soils present onsite had reached the point of saturation where there was a complete loss of shear strength, in some locations groundwater created ponding within the box. These locations were drained with additional subsoil drains and a working platform was constructed using rock and geofabrics. Some of the problems encountered are evident in Figure 57.

Figure 57: Buderim St, Currimundi - Construction Photos Subgrade Replacement

5.7.3 Investigation Results

Figure 58 below demonstrates significant improvement in maximum deflections from testing prior to the pavement rehabilitation in 2012 and post construction in 2014. Maximum deflections are well within Sunshine Coast Council's acceptable range with an average maximum deflection of 0.28mm. An improvement from 0.80mm tested in 2012.

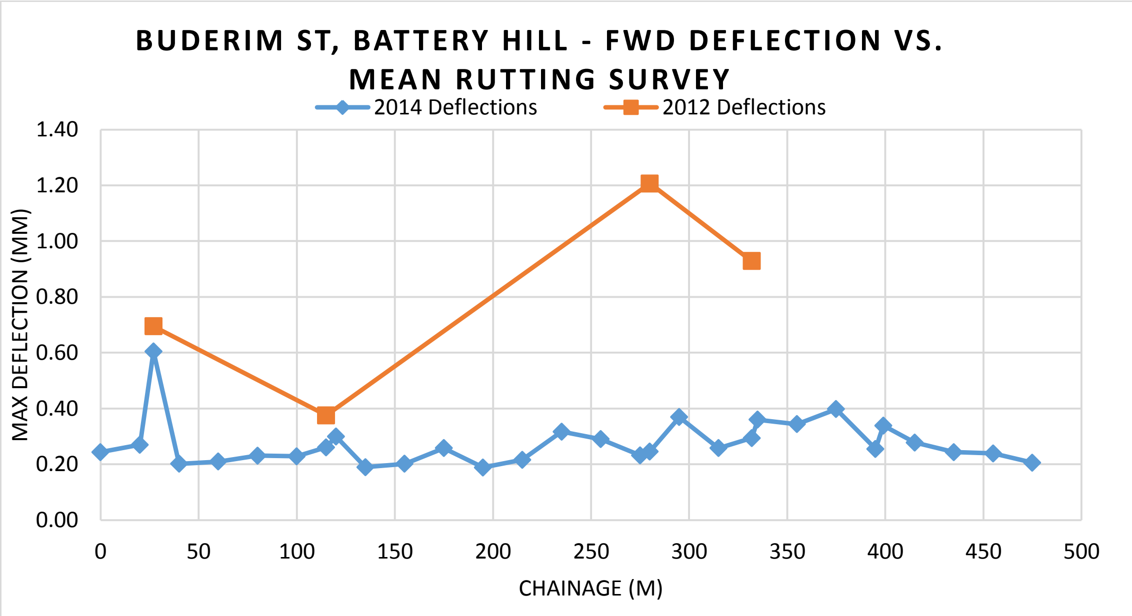


Figure 58: Buderim St, Currimundi - 2012 and 2014 Maximum Deflections

Deflection testing has a much more limited application to rigid pavements (Austroads, 2009). Notwithstanding, Buderim Street, Currimundi has an average relationship between CF values and maximum deflection of 18% with one (1) location which returned a high result ranging of 67%, shown in Figure 59. This test location was situated approximately at the join of the section of subgrade failure outlined in Section 5.7.2 above, where significant subgrade improvement was required due to groundwater and water logged clay subgrade.

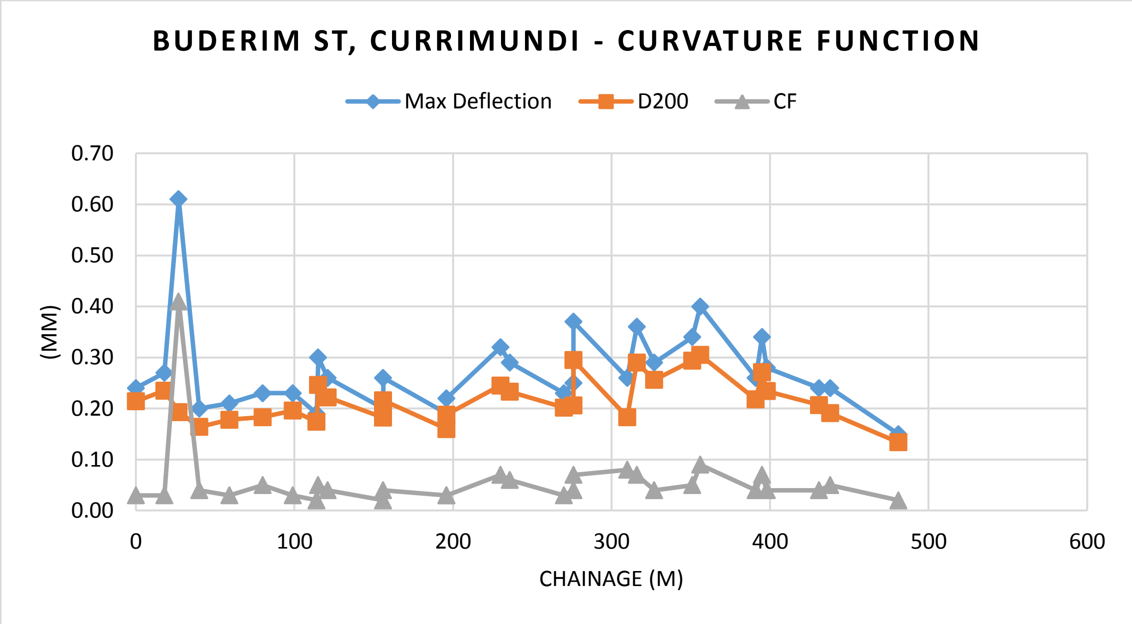


Figure 59: Buderim St, Currimundi - Curvature Function

Further investigation into the construction processes used is required to identify areas of pavement joins and if these results are related to paving patterns or failures due to underground infrastructure, common in areas of ageing infrastructure and sandy subgrade materials.

Figure 60 indicates no correlation between rutting and maximum deflection. Indicating rutting may be a result of early trafficking. Results indicate an average of 3.89mm and 3.48mm rutting for the right and left outer wheel paths respectively.

Reporting the severity and extent of outer wheel rutting in accordance with Section 4.3.3 results in severity ranges of 0-5mm and 5-10mm with 68% and 32% of the project respectively. Indicating no areas of structural concern.

The results above demonstrate the lowest relationship between deflection measurements and rutting results in conjunction with Point Cartwright Drive, Buddina

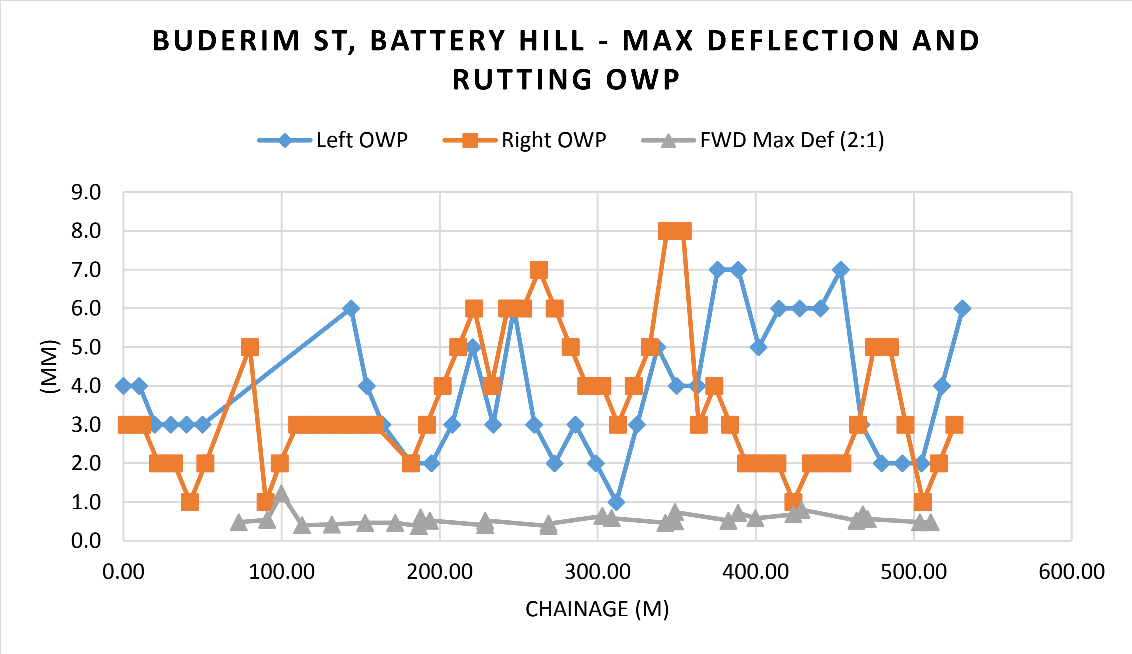


Figure 60: Buderim St, Currimundi - Rutting and Maximum Deflection

Previous experience in deep lift asphalt pavements suggests rutting may be a result of this pavement rehabilitation option due to early trafficking of the pavement and higher order roads and higher stress areas used i.e. roundabouts and major intersections. Sunshine Coast Council has recently specified Polymer Modified asphalts be used on all roundabouts and major intersections to alleviate this problem.

5.8.1 Summary of FWD Results

To determine the most effective pavement rehabilitation treatment from the projects outline in Section 5.7 a holistic approach to compare results has been adopted and provided in this section. Results aim to determine the most effective pavement rehabilitation option through comparing the following characteristics by project:

- Curvature function values;
- Calculated remaining life;
- Average maximum deflection and correlation to rutting;
- Average maximum deflection and subgrade CBR; and
- Average rutting and subgrade CBR.

Figure 61 outlines the results of testing for average curvature function per project. Results are shown in three (3) categories of severity with results below 0.2mm representing stiff pavements and pavements with CF values greater than 0.4 representing pavements lacking stiffness.

From the results Glenview Rd, Glenview a granular replacement pavement and Mary St, Alex Headlands a Tensar grid and pavement reconstruction project exhibit pavements with the highest values of CF, greater than the 0.4mm defined as high by TMR (2012).

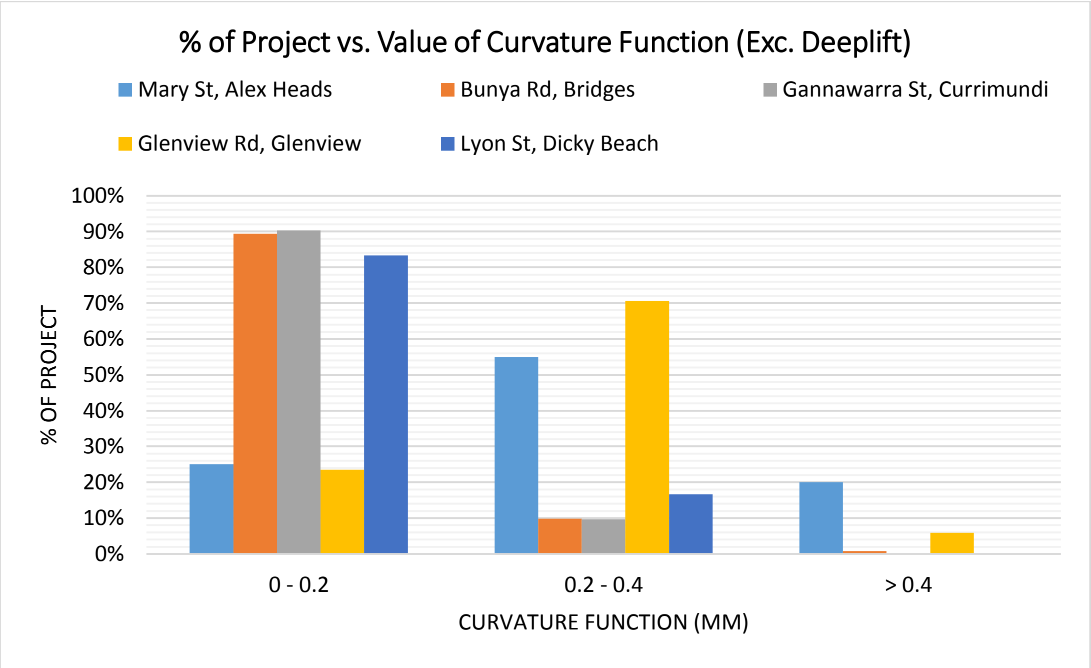


Figure 61: Curvature Functions of Various Pavement Rehabilitation Treatments

When considering the results for calculated remaining life of each pavement once again Mary Street, Alexandra Headland and Glenview Rd, Glenview demonstrate the lowest values of seventeen (17) and fourteen (14) years respectively. Lyon St, Dicky Beach also appears slightly lower than expected at eighteen and a half (18.5) years. Point Cartwright Drive, Buddina and Gannawarra Street, Currimundi achieving a remaining life of twenty (20) years.

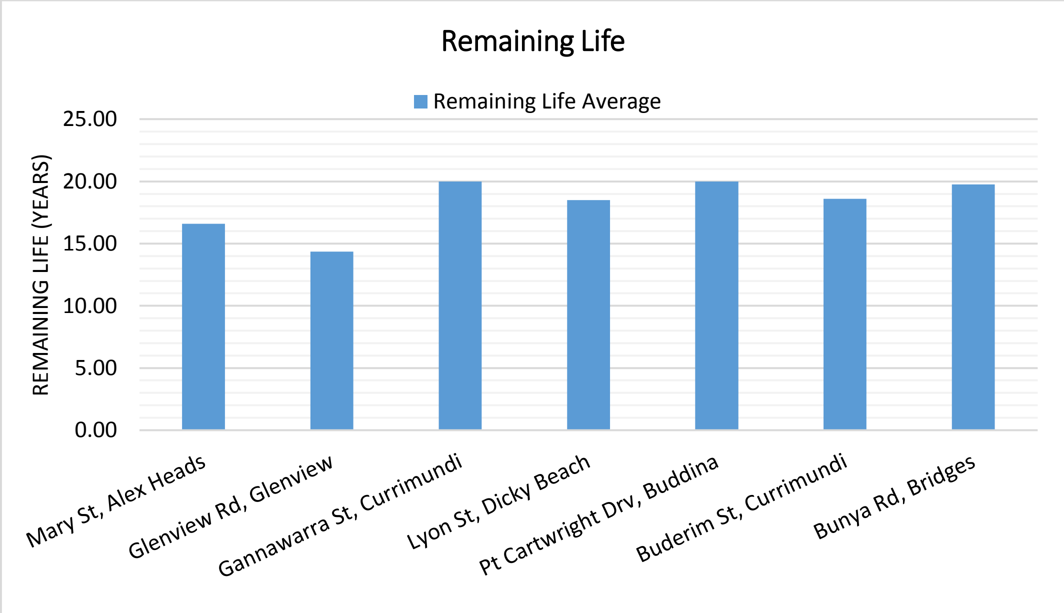


Figure 62: Remaining Life Comparison per Pavement Rehabilitation Option

The relationship between average rutting and average maximum deflection is shown in Figure 63. Glenview Road, Glenview and the two (2) asphalt deep lift pavement rehabilitations show the lowest correlation between rutting and maximum deflection. Confirming earlier results suggesting deep lift pavements are susceptible to rutting despite low deflections.

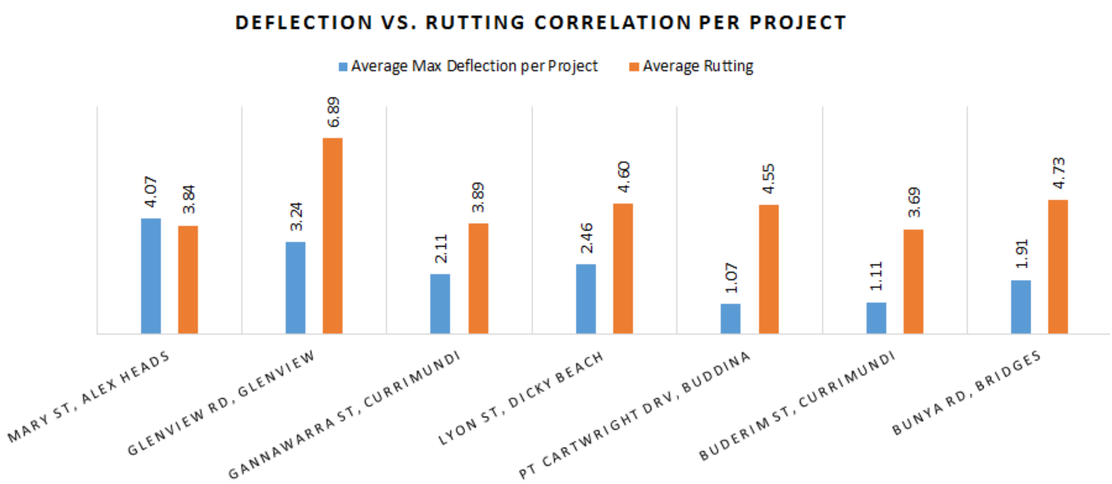


Figure 63: Relationship between Average Rutting Measured and Maximum Deflection Measured per Pavement Rehabilitation Option

The relationship between the average rutting per project and adopted design CBR strength is shown in Figure 64 shows Glenview Road, Glenview and Point Cartwright Drive, Buddina experience high rutting averages despite adopted design CBR's of 20%. Mary Street, Alexandra Headland and Lyon Street, Dicky Beach display favourable rutting results in comparison to the low adopted CBR strengths. This confirms the use of geosynthetics can minimise rutting in pavements.

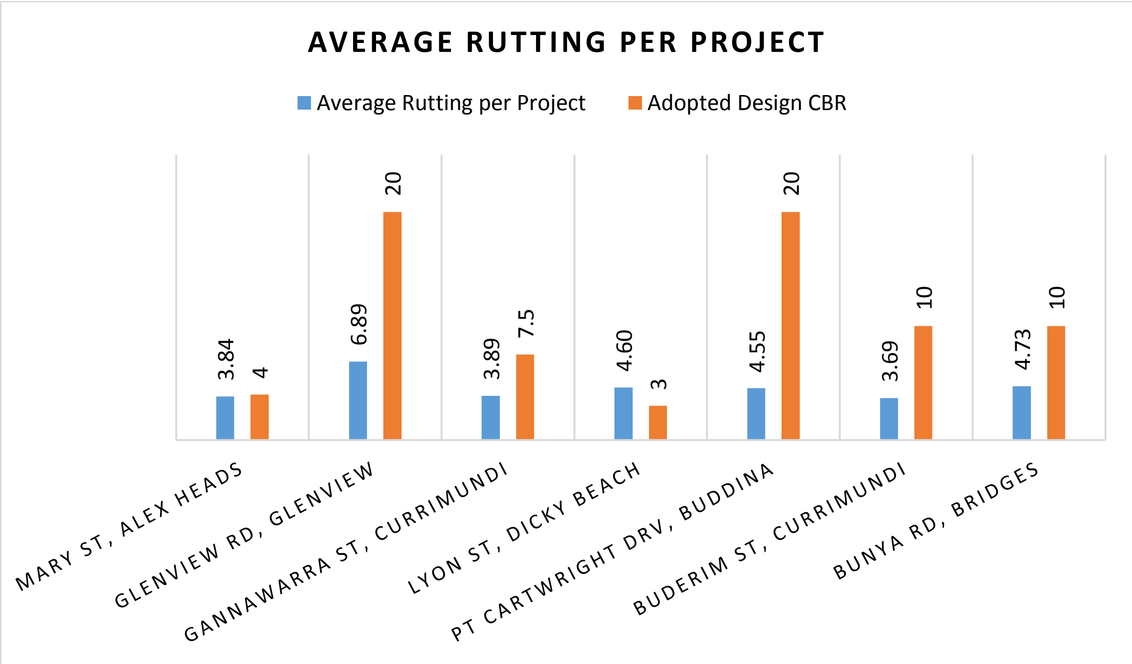


Figure 64: Relationship between Average Rutting Measured and Subgrade CBR Values

5.9 Financial Comparison of Options

Unit costs for alternative pavement rehabilitation treatments vary widely depending on factors such as degree of competition, location, availability of suitable resources, types of resources employed and how they are employed, and the scale of the project (TMR, 2012). TMR (2012) states that comparison by cost per square meter of the pavement alone is often misleading, although outlines the comparison of total project cost, including overheads can be used.

Table 9 below offers a comparison of total costs excluding costs associated with kerb and channel removal and construction, as only two (2) of the projects incurred these additional costs. A comparison by $\$/m^2$ has been provided, although depths of pavements vary between projects they are relative to the roads hierarchy. i.e. deep lift asphalt pavements for Collectors and above.

Project	m2	Cost ($)	$/m2	Unsuitable Subgrade Costs ($)
Bunya Road, Bridges	12,812	$ 821,085	$ 64.09	$ 93,483
Point Cartwright Drive, Buddina	5,089	$ 535,080	$ 105.14	$ -
Glenview Road, Glenview	1,950	$ 221,967	$ 113.83	$ 13,577
Buderim Street, Battery Hill	8,951	$ 1,039,507	$ 116.13	$ 91,570
Gannwarra Street, Currimundi	3,355	$ 520,983	$ 155.29	$ 10,290
Lyon Street, Dicky Beach	2,533	$ 393,853	$ 155.49	$ -
Mary Street, Alexandra Headland	4,400	$ 938,662	$ 213.33	$ 3,661

Table 9: Comparison of Alternative Pavement Rehabilitation Costs

The comparison of alternative pavement rehabilitation costs demonstrates that the granular overlay conducted at Bunya Road, Bridges was the cheapest per square meter. Followed by Point Cartwright Drive, Buddina which was a deep lift pavement on a trunk collector, constructed at night. While both deep lift pavement rehabilitations are competitively priced, whole of life costs associated with these options are much higher due to the inability to stabilise the existing material once it reaches the end of its useful life. Sunshine Coast Council traditionally prefers to only undertake deep lift asphalt pavements on higher order roads due to this reason, and to minimise disruption to the community and road users as a result of shorter construction times.

The two (2) projects which included geosynthetic materials were the most expensive per meter squared. This is partially due to the locations of each of these projects and the subgrade conditions available. Significant pavement reductions were applied with the use of geosynthetics, and consequently the costs could have been much higher when similar subgrade conditions and traffic volumes are experienced without the use of geosynthetics. The use of geosynthetics also largely demonstrated a reduction in costs associated with excavation of unsuitable material.

Table 9 outlines projects recently completed by Sunshine Coast Council which included the foam bitumen stabilisation at University Way, Sippy Downs, in-situ cement stabilisation of Bellvista Blvd, Caloundra, 30/30 combi-grid and pavement replacement at Rosevale Avenue, Aroona and the combination pavement including, geosynthetics, granular and deep lift asphalt at Beerburrum Street, Battery Hill.

These results demonstrate the cost effectiveness of in-situ stabilisation as a pavement rehabilitation option. Bellvista Boulevard, Caloundra outlines a significant cost reduction for treatment of Collector roads, while University Way, Sippy Downs is comparable to Gannwarra Street, Currimundi at approximately $60/m^2 less initial construction costs.

Project	m2	Cost ($)	$/m2
Bellvista Boulevard, Caloundra	6,650	$ 480,252	$ 72.22
University Way, Sippy Downs	3,963	$ 369,096	$ 93.14
Rosevale Ave, Aroona	4,980	$ 475,163	$ 95.41
Beerburrum St, Battery Hill	9,000	$ 1,447,948	$ 160.88

Table 10: Alternative Pavement Rehabilitation Cost Comparison - Recently Completed

The use of geosynthetics in the construction of Beerburrum Street reduced initial construction costs by one third. Ongoing road condition assessments are required to determine the effectiveness of these projects in relation to pavement performance.

Table 11 below defines a cost comparison by pavement rehabilitation treatment option. Granular overlays are the most economical however, only usually achievable in rural areas due to existing infrastructure in urban environments. In-situ stabilisation using foam bitumen or cement provide economical options where sufficient existing pavement materials allow.

Project	$/m2	
Granular Overlay	$	64.09
In-situ Cement Stabilisation	$	72.22
In-situ Foam Bitumen Stabilisation	$	93.14
Deep Lift Asphalt	$	110.64
Granular and Combi-grid	$	125.45
Granular Replacement	$	134.56
Combi-grid, Granular and Deep Lift Asphalt	$	160.88
Granular and Tensar Grid	$	213.33

Table 11: Alternative Pavement Rehabilitation Costs by Treatment

Further work is required to investigate construction methodologies to incorporate in-situ stabilisation where existing pavement materials are insufficient. Plant mixed stabilisation is a possible solution to this problem and will be discussed in Section 8.1.

6.0 Evaluation of Roads Less Than 10 Years Old

To gain a wider understanding of pavement rehabilitation treatments and their effectiveness within the Sunshine Coast region a study into roads constructed or rehabilitated within the last ten (10) years was undertaken. This data was extracted from Sunshine Coast Council's Pavement Management System (PMS). This data provided construction dates, pavement profiles, treatment history and subgrade CBR values and accuracy. These projects were then correlated with the recent road condition survey undertaken by Radar Portal Services and assessed for rutting and roughness characteristics. These results provide a brief overview of pavement rehabilitation options which have experienced 10 years of environmental factors and traffic loadings. This section provides an insight into the longer performance of rehabilitation options in comparison to those considered in Section 5.

Initially, data extracted from the PMS was to establish a profile of subgrade CBR strength by suburb. This is shown in Figure 65 below. Only laboratory confirmed subgrade CBR values were included in this profile. While this provides a strong insight into local conditions, subgrade materials vary largely and need to be considered on a site specific basis.

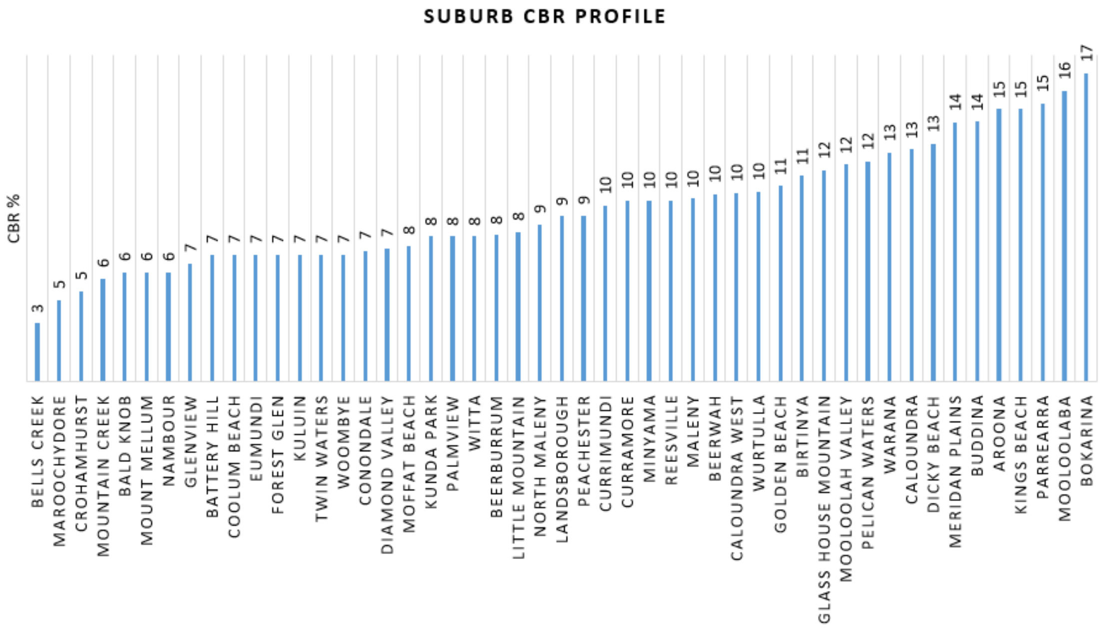


Figure 65: Subgrade CBR Suburb Profile

Through analysing Council's PMS system, treatments were categorised into nineteen (19) generic pavement types. Consisting of varying forms of in-situ stabilisation, ex-situ cement treated pavement materials, combinations of granular, stabilised material and structural asphalt, full depth asphalt and pavement incorporating geosynthetics. Figure 66 defines the severity of rutting evident by percentage of length of project pavement types in accordance with Austroads (2007) severity levels.

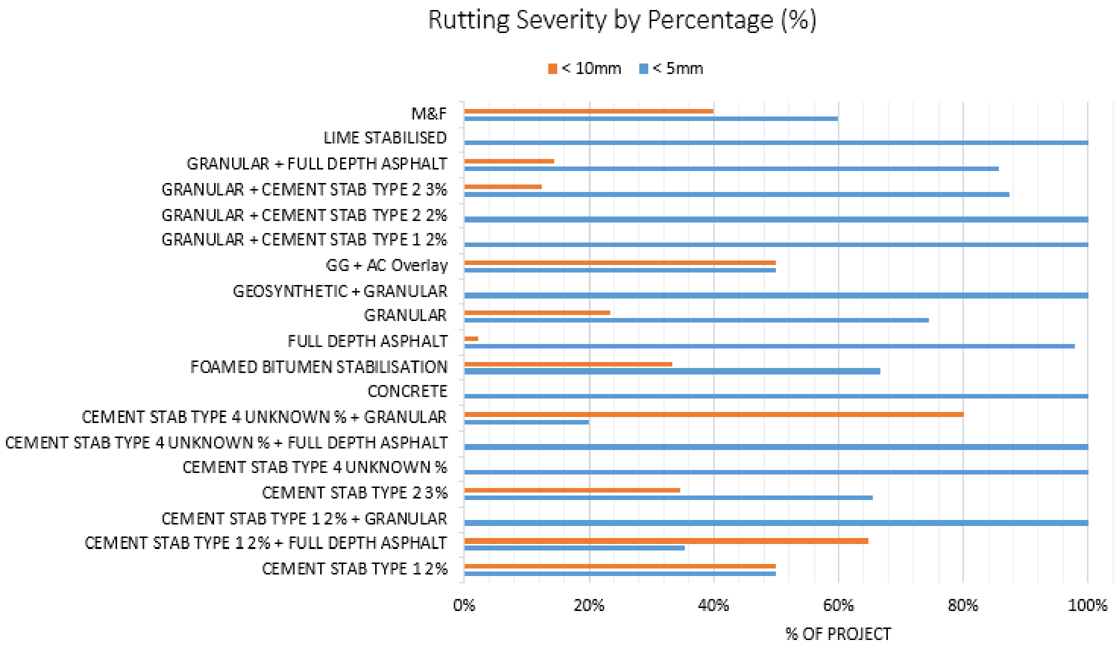


Figure 66: PMS Roads less than 10 Years - Rutting Severity by % and Pavement Rehabilitation Option

From Figure 66 it can be determined that the following treatment types exhibit high percentages of rutting severity greater than 10mm:

- M&F – Milling the wearing surface and reinstating (40%);
- Granular and full depth asphalt (15%);
- Granular and cement stabilised type 2 3% (12%);
- Glass grid and overlay (50%);
- Granular pavements (24%);
- Foamed bitumen stabilisation (33%);

- Cement stabilised and granular (80%);
- Cement stabilised type 2 3% (35%);
- Cement stabilised type 1 2% and full depth asphalt (64%); and
- Cement stabilised type 1 2% (50%).

Figure 67 displays the average rutting mean by treatment type. From this it can be clearly seen that combinations of cement stabilised and granular or deep lift asphalt experience high rutting averages, 5.80mm and 5.82mm respectively. The use of glass-grid and asphalt overlay also underperforms with a rutting mean of 5.38mm. Combinations where cement treated pavement materials are used as a base in comparison to a sub-base perform much better. Geosynthetics and granular pavements return a mean rutting of 3.14mm and is placed in the lower quartile. This may be due to the relatively young age of these pavements in comparison.

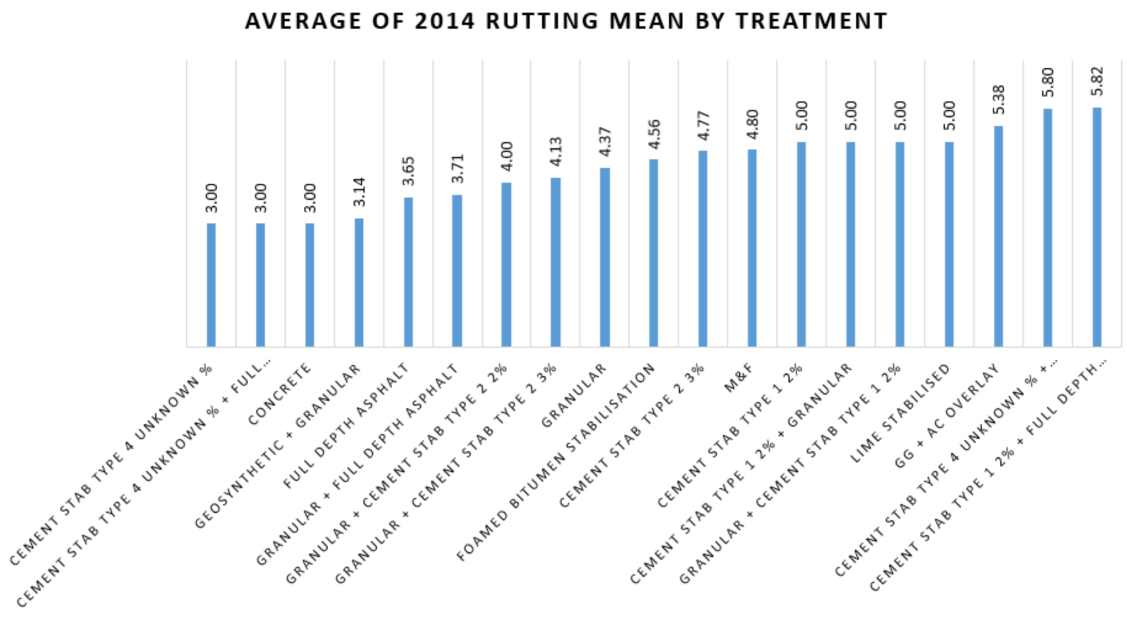


Figure 67: PMS Roads less than 10 Years - Average Rutting by Pavement Rehabilitation Option

The hierarchy of the road also significantly increases the likelihood of higher severity rutting. Figure 68 determines the mean rutting values by hierarchy for roads within the Sunshine Coast, constructed or rehabilitated within the last 10 years. Sub-arterial, rural collectors and industrial collectors return the highest rutting means of 5.22mm, 5.27mm and 5.36mm respectively. Collectors and trunk collectors returned mean rutting averages of 3.94mm and 4.11mm. Cameron Shields of the Sunshine Coast Council suggested this could be explained by Sunshine Coast Council's recent prioritisation of roads of regional significance including various collectors and trunk collectors, reducing the age of these roads in comparison to rural and industrial collectors.

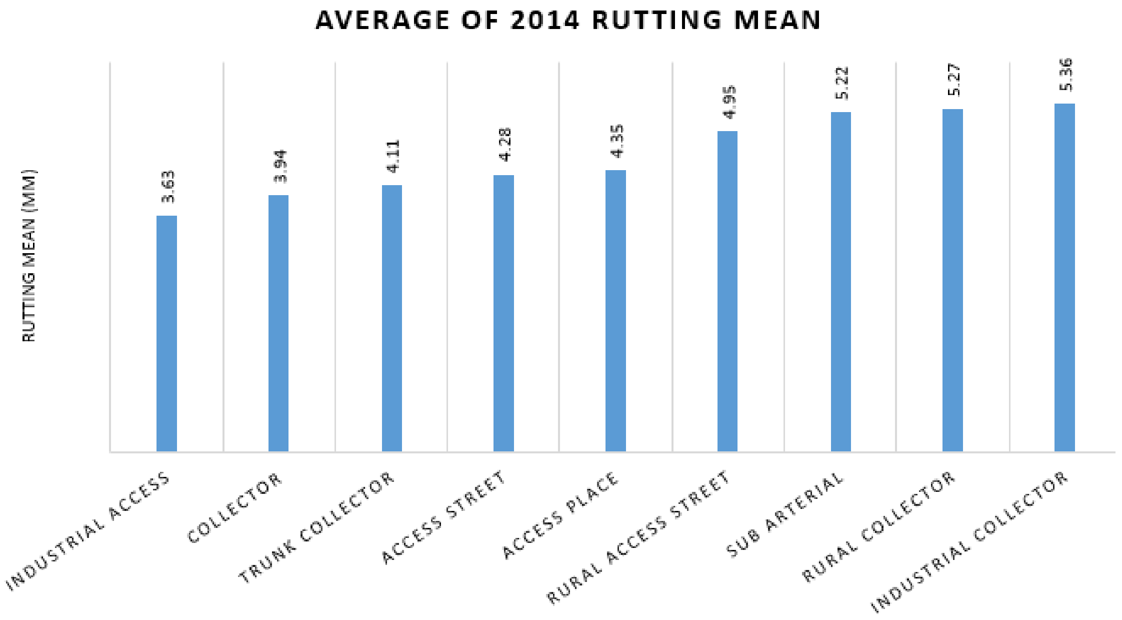


Figure 68: PMS Roads less than 10 Years - Average Rutting by Hierarchy

Shown in Figure 69 below are the average rutting means for construction years, reported by Sunshine Coast Council's recent road condition survey. The general trend depicts an expected outcome of lower rutting means for projects constructed within the last 5 years, compared with those constructed between 2004 and 2008.

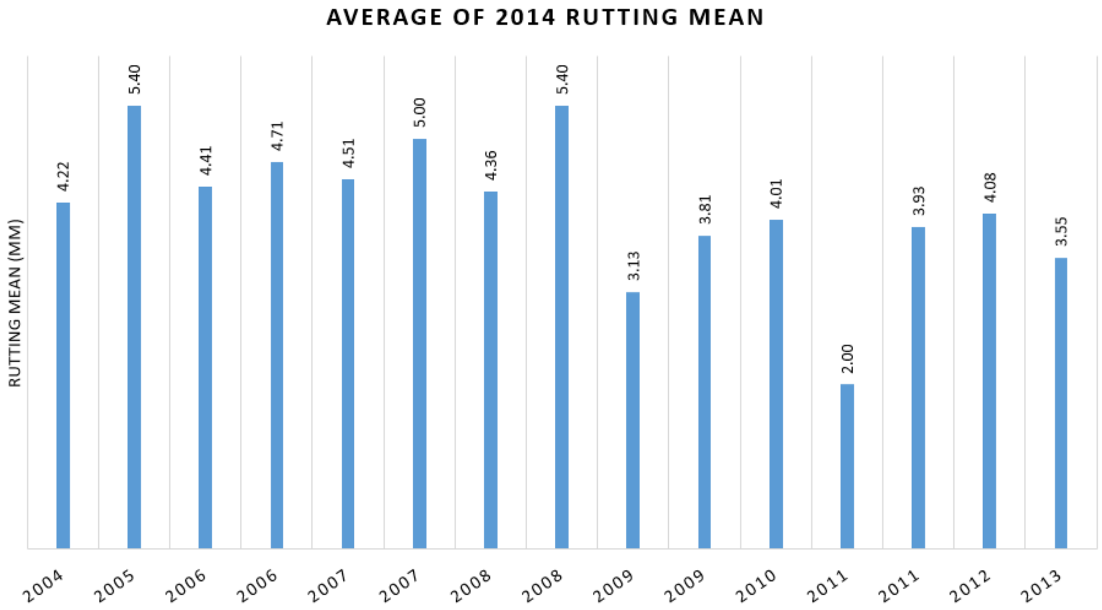


Figure 69: PMS Roads less than 10 Years - Average Rutting by Construction Year

Roughness is used to represent the riding quality of a pavement and can be an indicator of the serviceability and/or structural condition of a pavement (TMR, 2012). TMR (2012) suggests that the roughness of a pavement usually increases with time from initial construction to the end of its useful life.

TMR (2012) suggests intervention levels for roughness, however, this is only relevant for motorways, urban arterials, urban sub-arterials and rural highways. As suburban streets are designed for variable speed, Transport and Main Roads criteria is not applicable. For the purpose of this research project the levels of severity of roughness measured in units of International Roughness Index (IRI) are outlined in Table 7, Section 4.4.3.

Therefore, Figures 70 and 71 display the results of the 2014 road condition survey in units of IRI and IRI3 respectively. IRI3 as previously outlined is a method applied by Council's contractor to achieve a more accurate roughness measurement for suburban roads. From these results the following treatment types experience high roughness measurements, with a small percentage of results within the Poor (12-20mm) severity level:

- Granular replacement pavements; and
- Granular and full depth asphalt.

Followed by projects which demonstrate a significant percentage of their length in the moderate condition range (6-12mm):

- Granular replacement and cement stabilised type 2 3% base material;
- Granular replacement and cement stabilised type 2 1% base material;
- Glass-grid and asphalt overlay;
- Full depth asphalt;
- Foamed bitumen stabilisation; and
- Cement stabilisation type 2 3%.

This reduces to the following when considering the contractors methodology (IRI3):

- Granular and full depth asphalt;
- Full depth asphalt;
- Granular replacement pavements; and
- Geosynthetic and granular pavements.

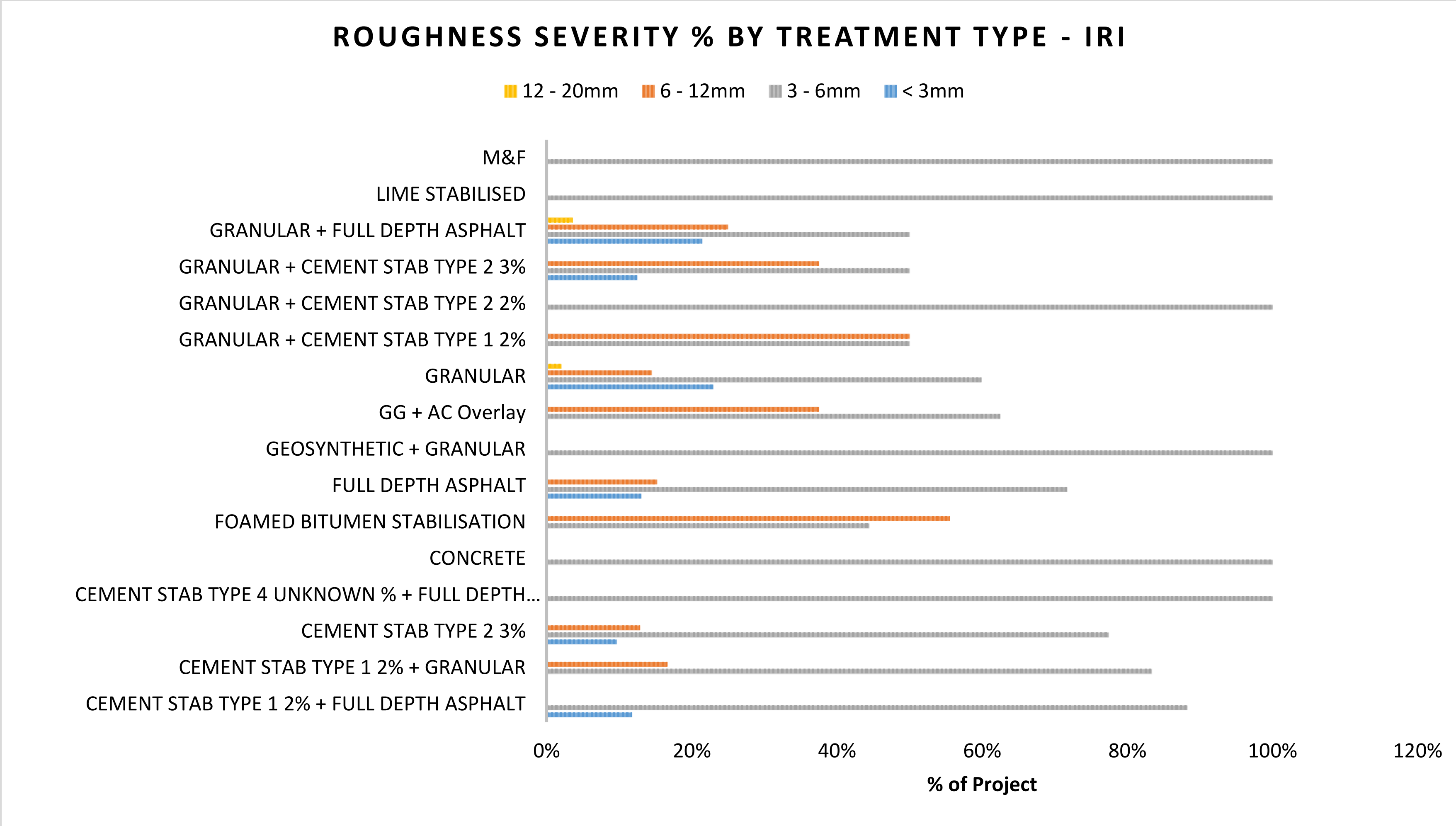


Figure 70: PMS Roads less than 10 Years - Rutting Severity (IRI) Intervention Levels by Pavement Rehabilitation Option

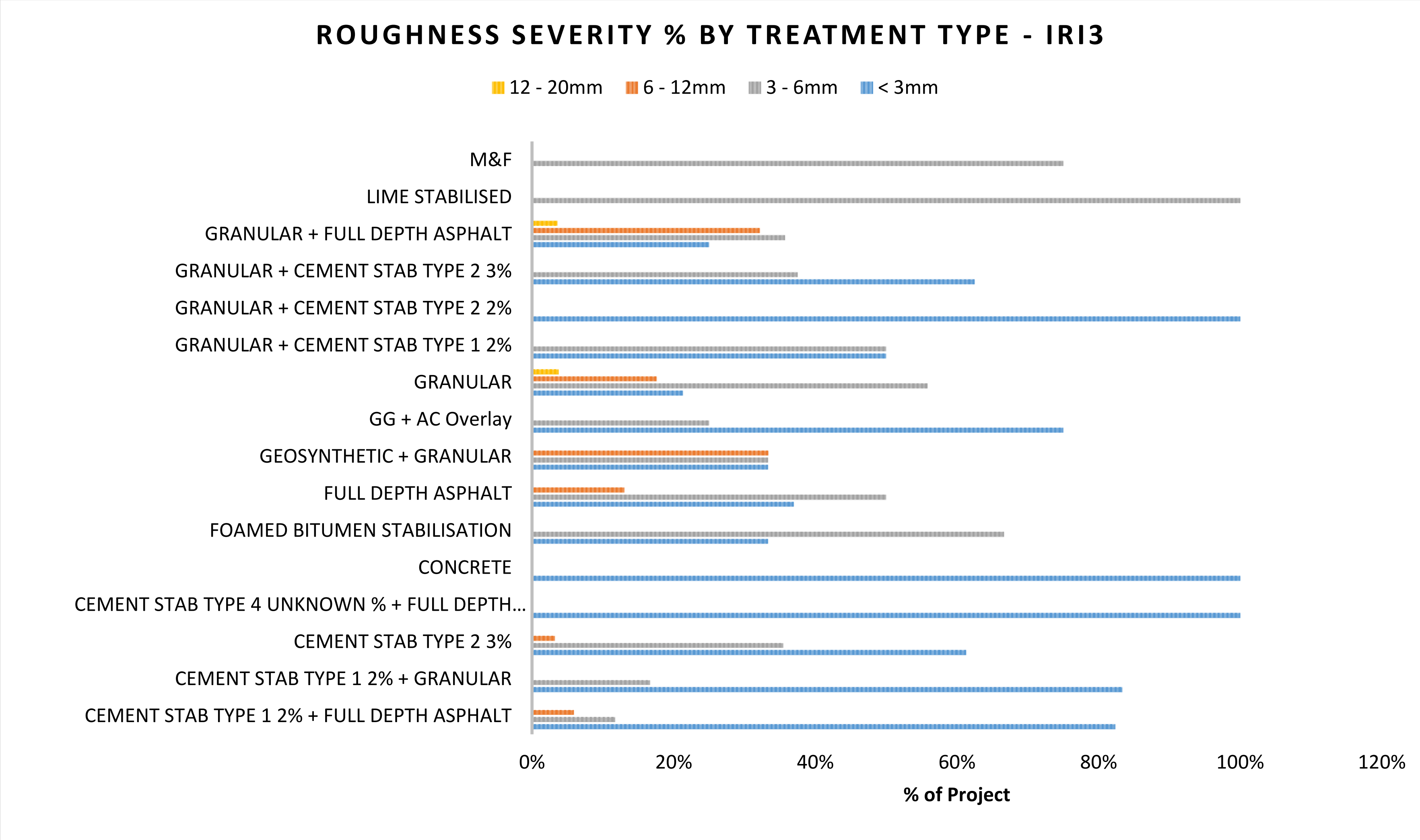


Figure 71: Rutting Severity (IRI3) Intervention Levels by Pavement Rehabilitation Option

Figure 72 below simplifies the results of the road condition survey in relation to roughness by comparing the average roughness by treatment type. It shows both methodologies including both IRI and IRI3.

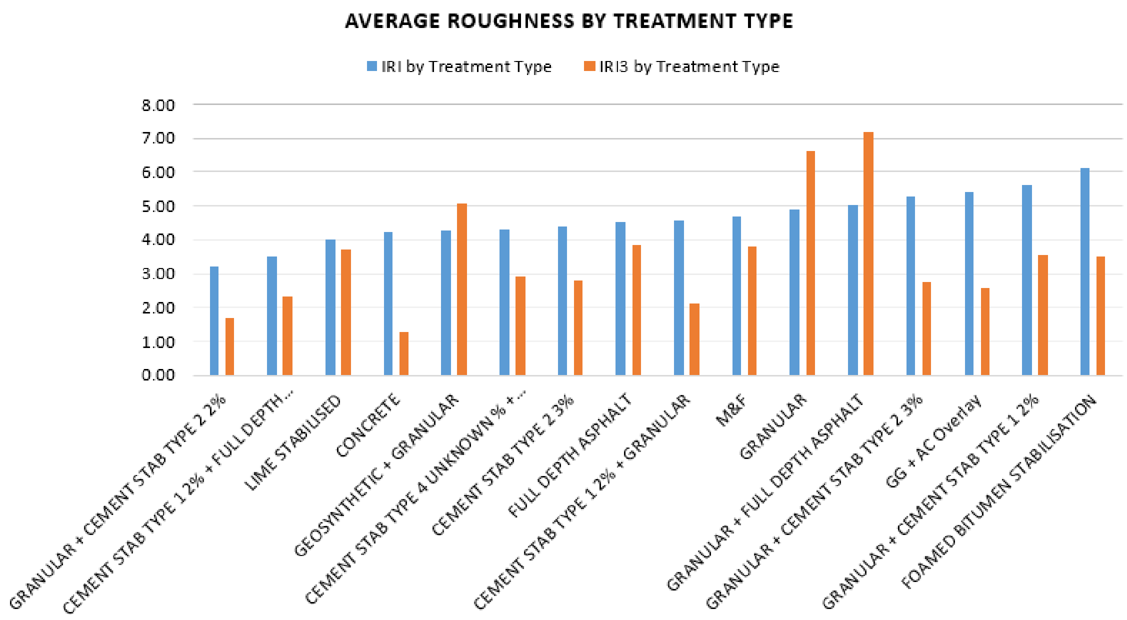


Figure 72: PMS less than 10 Years - Average Roughness by Pavement Rehabilitation Option

Results from Figure 72 suggest foam bitumen stabilisation, deep lift asphalt, glass-grid and asphalt overlay, and granular and cement treated base combinations result in high roughness. Geosynthetic and granular combination pavements perform adequately and return roughness values both IRI and IRI3 less than 5mm.

Further research and testing is required to determine the effectiveness of the various cement treated stabilisation pavements. Pavements which consist of cement treated base or sub-base materials show inconsistent results. This may be due to a number of factors including but not limited to the quality of materials, quality assurance during construction and inconsistent spread rates at the time of construction.

7.0 Discussion

7.1 FWD Results

Section 5 discusses seven (7) recent pavement rehabilitation projects undertaken by Sunshine Coast Council on pavements with varying traffic loadings, and with the exception of Point Cartwright Drive, Buddina extremely poor subgrade conditions. A wide range of results has been received across a variety of treatments including granular replacement, granular overlay, deep lift asphalt and pavements incorporating geosynthetic products.

As expected the deep lift pavements constructed on Buderim Street, Battery Hill and Point Cartwright Drive, Buddina returned the lowest maximum deflection results of 0.28mm and 0.26mm respectively. While the granular pavement constructed at Glenview Road only demonstrated a slight improvement, returning a relatively high maximum deflection of 0.81mm post pavement rehabilitation works. The worst performing project was Mary Street, Alexandra Headland with an average maximum deflection of 1.09mm, far exceeding the 0.61mm results on Lyon Street, Dicky Beach, a road of similar hierarchy and equivalent reinforced geosynthetic pavement. This suggests a difference in performance relative to the type of geosynthetic product used, however, performance could also be due to subgrade properties, quality of construction materials used and methods of construction. Further investigation into the pavement and subgrade materials would be required to investigate properly.

Austroads (2009) suggests that flexible pavements should return a curvature function approximately 25% to 35% of the maximum deflection. Results suggests Mary Street, Alexandra Headland is once again the worst performing project exceeding the maximum desirable curvature function in five (5) locations, with CF values ranging from 0.4mm to 0.96mm. Lyon Street, Dicky Beach constructed using an alternative geosynthetic product returned two (2) locations exceeding the maximum desirable percentage of maximum deflection, however, the corresponding CF values were 0.13mm and 0.15mm respectively. Therefore, well below the acceptable 0.2mm for new pavements.

The granular pavement at Glenview Rd, Glenview is experiencing average rutting of 7.19mm and 6.58mm per lane with 48% and 18% of the project experiencing severity rutting of 5–10mm and 10–15mm respectively, far in excess of any other project. In comparison a granular pavement at Bunya Road, Bridges, also a rural collector is experiencing higher than desirable rutting results with 4.30mm and 5.17mm and severity composition of 24% and 1% respectively. Suggesting granular pavements in these situations are currently being constructed too thin to protect the subgrade.

The remaining life of each project was determined through back calculation analysis from the FWD testing with results suggesting several roads will not reach their design life of twenty (20) years, in particular Mary Street, Alexandra Headland and Glenview Road. Results have indicated that Glenview Road has a remaining life of sixteen and a half (16.5) years, while Mary Street has fourteen (14) years remaining. Results suggest that pavement rehabilitation options applied at each of these locations are not suitable to obtain a twenty (20) year design life, this may be due to insufficient pavement thickness, subgrade conditions, quality of materials used and work procedures during construction.

Despite the negative results, Mary Street has performed the best when comparing the correlation between rutting and maximum deflection. Considering a high average deflection of 1.09mm, the rutting mean for this section of road was only 3.84mm, the lowest of any project. Concluding the geosynthetic product is performing adequately to control rutting fatigue and suggests that there is inadequate pavement thickness and strength to protect the subgrade from influencing surface deflection.

Comparing FWD and rutting results per treatment type concludes:

- Granular pavements in rural areas on clay subgrades experience high maximum deflections, greater than desirable CF values and higher than average rutting results. An increase in thickness may be required to minimise loading on the subgrade;

- Granular pavements on sand subgrades perform well with low deflection results and acceptable rutting means;

- Deep lift asphalt pavements on sand subgrades perform adequately and experience low maximum deflection results. Deep lift asphalt pavements experience moderate to high rutting results in comparison to maximum deflection. Rutting is likely due to early trafficking and densification of the upper layers. The use of Polymer Modified Asphalts in high stress areas such as intersections and roundabouts may decrease mean rutting;

- Deep lift asphalt pavements on clay subgrades may require subgrade improvement and alternative paving equipment to reduce the risk of unsuitable replacement delaying works. Further research is required into the effect of construction loads and hot mix asphalt on the behaviour of subgrade materials; and

- The results of pavements incorporating geosynthetics are site and product specific. Geosynthetics have reduced pavement thicknesses and controlled rutting to a moderate level on very poor subgrades. In one (1) location geosynthetics have provided significant improvement in pavement performance, significantly reducing maximum deflection and producing relatively low CF values. Improvement in how geosynthetic products are modelled in mechanistic pavement design is required.

7.2 Road Condition Survey Results

Rutting

Considering rutting results of roads constructed or rehabilitated within the last ten (10) years on the Sunshine Coast, the best performers were:

- Concrete pavements;
- Geosynthetic and granular pavements; and
- Granular and cement stabilised pavements.

The worst performing pavements included:

- Cement stabilised sub-base and granular base pavements;
- Glass-grid and asphalt overlay pavements;
- Mill and fill asphalt wearing surfaces; and
- Cement stabilised (2%) and full depth asphalt pavements.

While concrete pavements are resistant to rutting, it is not a cost effective pavement rehabilitation option for local government roads. Granular and cement stabilised pavement combination results were inconsistent and the results varied greatly depending on the percentage of cement added. It was also noted that roads older than six (6) years demonstrated a significant increase in average rutting results.

Considering rutting results, it is suggested Council continues to invest in pavement combinations of geosynthetic and granular materials concentrating on the way in which products are installed during construction and modelled during mechanistic design. Results also suggest variable results for cement stabilised pavements, with a wide distribution of behaviour characteristics, however, low rutting results are evident when cement stabilised materials are used as base materials.

Roughness:

Considering roughness results of roads constructed or rehabilitated within the last ten (10) years on the Sunshine Coast the best performers were:

- Granular and cement stabilised 2%;
- Concrete;
- Lime stabilisation; and
- Geosynthetics and Granular.

Pavements which demonstrated moderate roughness included:

- Glass-grid and asphalt;
- Granular and 3% CTB base;
- Deep lift asphalt;
- Cement Stabilisation 3%; and
- Foamed bitumen.

The worst performing pavements included:

- Granular and full depth asphalt combination; and
- Granular replacements

As the measurement of roughness focuses on characteristic dimensions that affect vehicle dynamics and hence road user costs, ride quality and dynamic pavement loads it is an important consideration when evaluating the effectiveness of previous pavement options.

As found when evaluating the rutting results concrete pavements performed well, however, will not be widely used due to associated costs. Continuing investment into pavement combinations of geosynthetic and granular materials is supported by favourable roughness results. Once again cement stabilised pavements have varying results. Granular and cement stabilised base materials with 2% additive display low roughness results. Results suggest further use of pavement rehabilitation treatments which include 2% cement stabilised base materials.

7.3 Financials

Unit costs for alternative pavement rehabilitation options depend widely on factors including locality, availability of resources (i.e. plant, personnel and materials), types of resources and their use. Sunshine Coast Council has records of unit costs from past pavement rehabilitation projects. For Sunshine Coast Council these can be used to estimate the costs of options being considered. As with all pavement rehabilitation projects there are several other associated costs which warrant consideration, including the scope of works and what the difference between what constitutes a pavement rehabilitation compared with a reconstruction.

Some options require extensive excavation (where finished surface levels are fixed by existing infrastructure), some may interfere with public utilities, or require significant shoulder and widening works to increase the road to current standards. Savings can be made through the consideration of various options for example, the selection of materials for shoulder widening could contain asphalt or stabilised layers which are generally thinner than granular. Consequently comparing costs per square meter is often misleading. For this research project costs included all ancillary works excluding concrete kerb and channel renewal as Sunshine Coast Council considers this reconstruction, funded from a different sub-program within Council’s budget. The total costs used for comparison includes project overheads and non-pavement activities, which vary between options.

As suggested by Transport and Department of Main Roads (2012), other costs included which were not part of the pavement unit costs, which vary between projects included:

- Provision of traffic management;
- Wet weather;
- Establishment and disestablishment;
- Supervision;
- Overheads;
- Relocation of public utilities; and
- Testing.

Detailed costing by activity types for projects investigated are included in Appendix E.

Considering the unit cost per square meter of recent pavement rehabilitation projects granular overlays at Bunya Rd provided the cheapest capital costs at $64/m^2 however, from testing results indicate long term maintenance costs incurred for these treatments may be high, with shorter useful lives.

Initially the use of geosynthetic products within granular pavements were expensive treatment options for lower order roads. Recently unit costs on similar projects have reduced significantly through the use of more experienced contractors and the recycling and regrading of existing pavement materials for use as a sub-base material. A reduction of 40% to 55% in unit costs for geosynthetic treatments has been observed since initial implementation, equivalent to the unit costs for recent foam bitumen stabilisation works, ranging from $95/m^2 to $215/m^2. Further testing is required to determine long term maintenance costs of pavement including geosynthetics.

Chemical stabilisation is the most sustainable pavement rehabilitation option where appropriate. While recent unit costs are slightly higher than granular overlays, it is available for use in areas where finished surface levels are restricted. Stabilisation is often dismissed due to insufficient depth or poor grading of materials. Improvements into how in-situ materials can be treated to be suitable for stabilisation is required, this may include plant mixed products which may have a detrimental effect on the unit costs of chemically stabilised pavements however, there is a concerted push to become increasingly sustainable and the recycling of existing pavement materials reduces demand on virgin materials. Cement stabilisation remains the cheaper alternative compared with foamed bitumen with Sunshine Coast Council undertaking minimal foamed bitumen stabilisation, long term results are largely unavailable. Stabilisation unit costs range between $72/m^2 to $95/m^2.

Deep lift asphalt pavements provide a cost effective pavement rehabilitation option for high order roads, where the duration of construction needs to be minimised. The process is relatively quick in comparison to alternative pavement rehabilitation options which are beneficial where social and environmental impact is required to be minimised. Deep lift asphalt is competitively priced within the region at $110/m^2 due to supply from Council's internal asphalt plant. Deep lift asphalt pavements can negatively impact whole of life pavement costs as once the pavement reaches the end of its useful life stabilisation is not an option, in most cases requiring removal.

Granular pavement replacement options resulted with the highest unit costs per square meter, with an average of $135/m^2. There are a number of factors which contribute to this however, in projects considered as part of this research, excavation and removal of unsuitable subgrade costs are higher than alternative treatments. The use of geosynthetics attempts to minimise costs associated with removal of unsuitable materials, hence its inclusion in several recent pavement rehabilitation treatments.

Beerburrum St, Battery Hill, a combination pavement including geosynthetics, granular and deep lift was not considered for discussion with no comparative pavement rehabilitation projects for accurate comparison. Notwithstanding, the use of geosynthetics and a modified pavement design reduced costs on this project by 50%, as outlined in Section 8.3.1.

While the pavement rehabilitation options mentioned above have been used and demonstrate a wide range of results and financial benefits, as Sunshine Coast Council is a local government organisation project prioritisation and treatments can be politically influenced. Consideration is currently being given to reviewing treatments and target design lives of pavements to increase network coverage, albeit with potential impacts in the future. Sunshine Coast Council is reviewing design procedures and standards to stretch Council funds further through designs labelled 'Fit for Purpose' treatment options. This includes reductions in standard widths and a reduction in twenty (20) year pavement design life.

Considerations are completed with a whole of life approach in an attempt to reduce capital costs due to increased loadings and the requirements of pavements subjected to this loading with a twenty (20) year design life. For example, it may prove financially viable to undertake a 150mm mill and fill with deep lift asphalt twice within twenty (20) years in comparison to constructing a 600mm granular pavement once. Council records suggest that 'fit for purpose' treatments can provide good results achieving extremely good value for money, however, as seen in some locations premature failure has occurred, resulting in political and maintenance pressures.

8.0 Alternative Treatments / Future Considerations

The main purpose for this research is to determine alternative pavement rehabilitation options for the Sunshine Coast region and propose improvements to current processes. Sunshine Coast Council historically undertook significant cement stabilisation, asphalt deep lift and full depth pavement reconstruction. Recently pavement rehabilitation costs have grown exponentially, largely due to the additional ancillary works associated with projects, including but not limited to subsoil drainage, kerb and channel replacement and sub-surface stormwater network upgrades.

Sunshine Coast Council is currently reconsidering the definition of 'pavement rehabilitation' projects and whether associated ancillary upgrades as outlined above should be treated as reconstructions rather than pavement rehabilitations. This would limit pavement rehabilitations to works conducted on improving the pavement, subsoil drainage (if required) and minor widening and alignment improvements in rural areas. Adopting a 'fit for purpose' resolution to some projects could enable Council to treat more roads within current budget restrictions.

Furthermore, Sunshine Coast Council have conducted recent trials of alternative treatment options including and not limited to:

- Foamed bitumen stabilisation;
- Geosynthetics; and
- Recycled sub-base materials.

Additional work needs to be completed with the use of these technologies to realise the benefits of individual options. Recent projects including foamed bitumen stabilisation and recycled sub-base materials were completed September 2014, with monitoring and testing to follow over the coming years. Immediate savings can be made through redefining what constitutes a pavement rehabilitation project in comparison to a full road reconstruction. Reviewing designs to resemble 'fit for purpose' solutions is also an alternative to effective rehabilitate roads as a whole of network approach, covering more kilometres with the provided budgets.

8.1 Foamed Bitumen

Foam bitumen stabilisation is a process where existing pavement materials are treated with bitumen foam, either in-situ or in a process plant. The purpose of the process is to improve properties of the pavement gravels, with design modulus of the treated materials typically in the order of 1,000MPa to 2000MPa (compared with typical gravels between 350MPa and 500MPa).

Transport and Main Roads uses lime as a secondary stabilising agent in applications to:

- Stiffen the bituminous layer,
- Reduce stripping,
- Aid dispersion of foamed bitumen throughout the material,
- Improve initial stiffness and rut resistance; and
- Reduce moisture sensitivity of the stabilised material.

In urban situations, the treatment is successfully used when:

- Existing gravel materials are high quality; and
- Subgrade / sub-base materials provide solid construction platform

It is often considered that foam bitumen is not suitable for rehabilitation projects within the Sunshine Coast region due to the risks attributed to unknown and inconsistent quality of the pavement gravels, plastic subgrade and prolonged exposure to nearby residents to high noise and dust levels, in particular related to the in situ component of the work.

Some challenges associated with the suitability of projects due to the quality of existing pavement gravels, plastic subgrades and specialist machinery required for in-situ stabilisation can be solved through exploring plant mix foamed bitumen processes.

Plant mixing is a controlled environment with load cells ensuring correct additives with the ability to correct deficiencies and grading with sieving. This process also enables the addition of new material if required to increase quantities or improve grading. The benefits of plant mixed foamed bitumen during construction are the ability to inspect the subgrade of the road and replace problem areas. Plant mixed stabilisation can increase costs significantly if considered on a project by project basis. Prior planning and coordination of multiple pavement rehabilitation projects concurrently would reduce this risk.

Further benefits from using plant mixed methods are the recycling of old pavement material and the opportunity to test and improve the grading in advance. Sunshine Coast Council has surplus reclaim located at various stockpile sites throughout the region and testing, regrading and adding to this material could provide a material suitable for plant foamed bitumen to be batched and carted directly to a new site while the excavated material is stockpiled for future treatment. This option also allows for additional subgrade removal and disposal if additional pavement depth is required.

Typical plant mix foamed bitumen processes are shown in Figure 73.

Figure 73: RPQ Foam Bitumen Batching Plant at Swanbank

Queensland Department of Main Roads have performed many trials and have developed a specification for undertaking foam bitumen treatment which Sunshine Coast Council can develop to suit the requirements of projects within the Sunshine Coast network.

Sunshine Coast Council undertook in-situ foam bitumen stabilisation works at Toolborough Rd, Yandina Creek in 2008 with varying results. Photos from the visual inspection below show signs of block cracking and slight rutting. Rutting is likely due to early trafficking.

Figure 74: Toolborough Rd, Yandina Creek - Site Photographs

Recently Sunshine Coast Council undertook further stabilisation works at University Way, Sippy Downs completed in September 2014. This site will be monitored and the suitability of this treatment further assessed in due course.

Foam bitumen stabilisation is more expensive than traditional cement and lime stabilisation however, Transport and Main Roads research suggests the results are favourable. Given its proven track record with other road authorities, foam bitumen should be further implemented within the Sunshine Coast region.

8.2 Innovative Technologies and Recycling

Brisbane City Council has outlined a number of innovative pavement technologies and processes. The Asphalt Innovations Committee was formed consisting of members from Asset Management, Quarries, City Projects Office and Asset Services. The purpose is to advance investigation and implementation of asphalt surfacing technologies. The objective is to determine new, cost effective pavement solutions.

Brisbane City Council (BCC) has outlined its 2031 vision, summarised as follows:

Towards Zero Waste is a city-wide outcome.

- Waste as a potential resource of value;
- Minimising waste generation;
- Maximising resource recovery;
- Reducing waste to landfill; and
- Environmental, social and economic impacts of waste.

Therefore, the road network provides great opportunities for resource recovery and markets for recovered materials. BCC already actively uses recycled materials in its pavement works. BCC outlined its road related recycled material sources as:

- Profiled pavement – asphalt and granular;
- Waste glass;
- Crushed concrete; and
- In-situ stabilisation.

BCC's Pine Mountain Quarry recycling facility collects Recycled Asphalt Pavement (RAP) materials from across the city. Approximately 50,000 tonnes of RAP is reused in council's asphalt. Second class RAP is utilised in granular pavements with spoil used for quarry rehabilitation.

BCC specifications allow up to 20% use of RAP in structural asphalt layers and 15% in surface layers. There are limitations on the maximum amount of RAP included in mixes and wet RAP can cause drying and mixing issues. Further research is required to determine the optimal percentage of RAP for use in asphalt mixes.

BCC is also actively investigating the use of waste glass in asphalt. Waste glass otherwise ends up in landfill. Currently Type 4 Asphalt contains 5% crushed glass as a sand replacement with the potential to use 20,000 tonnes of -3mm crushed glass per annum. Council also admits there are some handling and processing challenges to be resolved.

As experienced on the Sunshine Coast many of BCC's roads were not designed or built to modern standards, comprising thin pavements with extremely variable quality pavements. BCC is not exempt from regular shrinkage cracking reflecting through the finished surface level however; it is not perceived as a structural issue for local streets. Cracking is often left untreated.

'FoamMix', is the ex-situ recycling of pavement gravel. It consists of foamed bitumen added to reclaimed pavement gravel, mixed at ambient temperatures using 97% recycled materials. Initial FWD testing indicates stiff granular material with trial sites undertaken by BCC to be monitored with contribution from QUT and industry specialists.

BCC is heading in the right direction through the use of RAP, waste glass and crushed concrete to reduce the demand for raw materials and landfill, and carbon emissions. Stabilisation has provided a low cost alternative to full reconstruction of local roads albeit with higher risk of premature failure. Plant mixed 'FoamMix' may allow greater re-use of existing pavement materials and makes good economic sense. Recycling of pavement materials contributes to BCC meeting its sustainability goals and could be adopted in some form by the Sunshine Coast Council.

The Sunshine Coast Council is an envious position compared to most local council's as it owns and operates its own asphalt plant and quarries. The replacement of Sunshine Coast Council's asphalt plant is being considered and needs to ensure it can facilitate the use of RAP and waste glass within its mixes. A component plant consisting of a variety of modules and attachments would be beneficial to include the option of batching foamed bitumen pavement materials and recycling surplus spoil and reclaimed pavement material.

8.3 Geosynthetics

The use of geosynthetics is not new and has been used in pavement design for the past 25 years. Geosynthetics cover a range of different products and materials which have a variety of different uses as summarised in Section 2. The specific use of geosynthetics this research focuses on is the reinforcement of pavement layers in poor subgrade areas.

Further use of geosynthetics for pavement reinforcement and subgrade stabilisation is recommended within the Sunshine Coast to reduce costs, reduce material and minimise excavation depths. Further testing and investigation into the variety of products available and the subgrade and traffic parameters suited to individual products. This research demonstrates geosynthetics can be used with varying results.

8.3.1 Beerburrum St, Dicky Beach

Beerburrum St, Dicky Beach between Nicklin Way and Dicky Beach recently underwent significant pavement rehabilitation works, completed September 2014. Consultants were engaged to determine appropriate treatments for the section of road. The length of the project site was 900m and consisted of both separated carriageways. Over past years various treatments had been applied to this section of road with minimal success due to traffic loadings and subgrade conditions. The pavement was designed for a 40 year design life and with an annual traffic growth of 4% the design ESA's were $2x\ 10^7$.

Visual inspections prior to commencement of the works determined the extent of defects in the pavement. The common distress types were rutting, potholes and all types of cracking. At many locations kerbs and gutters were also observed to be in poor condition. Most of the surface exhibited pumping of fines, indicating that water had penetrated the gravel and/or subgrade materials, which were suffering plastic deformation.

The soaked CBR values for the subgrade ranged between 2% and 4% for this section. All gravel materials were moist to wet, with groundwater observed as high as 200mm below the surface. Subgrade moisture content was approximately 5% above optimum on average.

Figure 75: Beerburrum St, Dicky Beach - Site Photographs

The following rehabilitation options were considered:

- Foam bitumen stabilisation;
- Cement stabilisation;
- Granular overlay;
- Asphalt overlay;
- Concrete overlay;
- Heavy patching; and
- Reconstruction.

Criteria used to assess treatment options were:

- Design life;
- Construction timing (disruptions to the residents and general public);
- Constructability, including staging;
- Construction cost;
- Maintenance cost; and
- Sustainability.

The recommended pavement rehabilitation treatment included the construction of subsoil drainage, considered essential to protect the road pavement. It was recommended that the subsoil drainage be constructed 6 to 12 months in advance of the pavement works. Construction costs associated with the subsoil drainage was estimated at $250,000.

The recommended pavement design for this section from Nicklin Way to CH900 was:

- Select subgrade replacement;
- 300mm in-situ cement stabilised subgrade;
- 290mm DG20 Class 320; and
- 50mm DG14 PMB.

The estimated cost for this option was in excess of $3,000,000. Despite the obvious cost comparison, there were significant concerns regarding the ability to successfully stabilise the CBR 2% subgrade material. The time required to undertake these works was also unacceptable.

Sunshine Coast Council in consultation with Geofabrics Australia were able to reconfigure the design to the following (sketch shown in Appendix D):

- 40/40 Combi-grid;
- 390mm Type 2.3 material;
- 185mm DG20 Class 320; and
- 50mm DG14 PMB.

Works were completed for $1,450,000.

Figure 76: Beerburrum St, Dicky Beach - Construction Stages

The use of geosynthetics have obvious benefits including cost and time of construction however, there is a wide variety of products and subsequent limitations for their use. In this instance it provided council with a cost effective solution, although sections of unsuitable subgrade material required removal even with the inclusion of combi-grid in the pavement design.

Sunshine Coast Council will continue to monitor the performance of this pavement with routine road condition assessments and maintenance inspections.

8.4 Fourth Generation Pavement Monitoring Devices - USC

Sunshine Coast Council in partnership with the University of the Sunshine Coast are in the process of installing several instrumentation systems called the Generation 4 Superior Monitoring Acquisition Road Response Transmitter System (G4 SMARRT System). The G4 SMARRT System measures temperature, pavement strain, pore water pressure, soil pressure and soil moisture within the pavement layers. It is based on real time pavement data being sent wirelessly from the Roys Road site to a mobile data logger, which has the capacity to send data at any time during the pavement design period. The system also has an added feature of a camera attached adjacent to the gauges in the road, it has the capabilities to take photos at a specific time of day and programmable to take a photo as a heavy load passes over the gauges. This feature allows for a better understanding of what type of traffic passes the area along with the frequency and time of use.

The instrumentation systems are located at Sippy Downs, Bellvista and Beerwah. The following figure shows a general description of the proposed instrumentation for one of the sites.

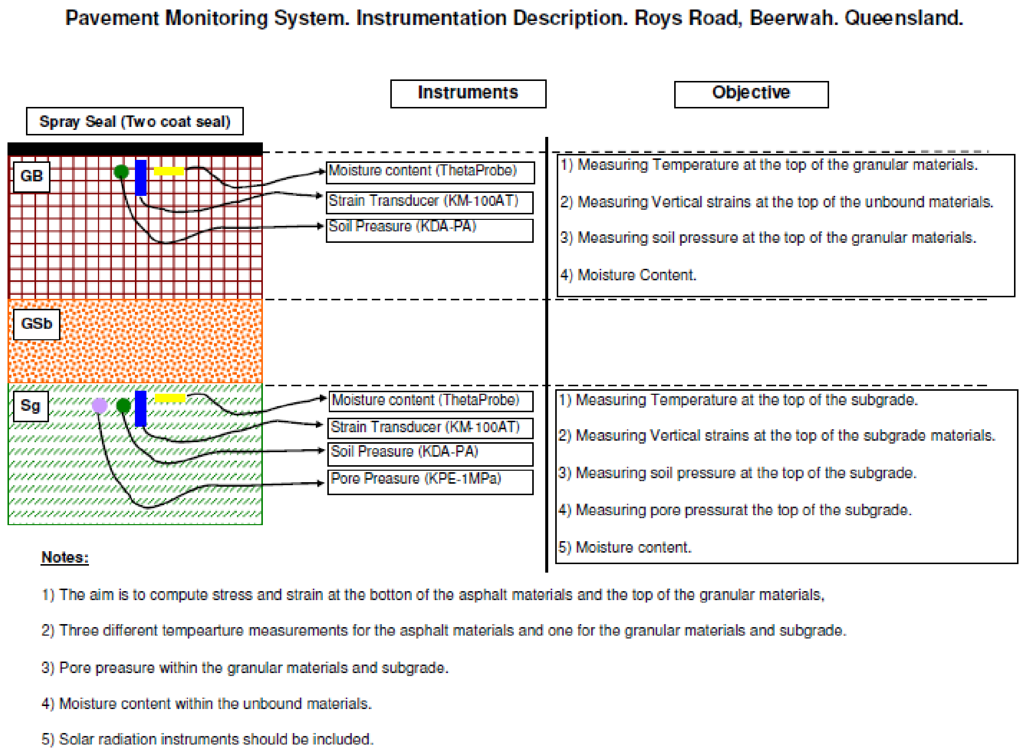


Figure 77: G4 SMARRT Instrumentation Schematic – Example

The initiative aims to determine any correlation r variances between the sites, along with further analysing various pavement aspects such as pore water pressures in the subgrade. The objective of the systems is to investigate and compare different structural pavement parameters under real site conditions and loads. Testing is undertaken to determine the specific properties of the pavement material, critical points of failure and ultimately defining the optimal pavement materials and process.

Sunshine Coast Council and the University of the Sunshine Coast envisage the G4 SMARRT System redefining the way in which pavement designs are undertaken within the region. Improving the understanding of pavement materials subject to environmental conditions and traffic and construction loads. Further research and monitoring of results is required to determine tangible outcomes and suggested improvements.

9.0 Recommendations and Conclusion

9.1 Recommendations

The following recommendations and improvements are a result of this research and could be investigated further to increase the effectiveness of Sunshine Coast Council's pavement rehabilitation treatments. Some of the recommendations noted include:

- Subgrade replacement depths should be minimised and alternative options considered. Geosynthetics are proving effective with each type providing specific benefits such as reinforcement, drainage and separation with carrying results.

- Identifying the cause of pavement failure and accurate assessment of pavement and subgrade material is essential to enable best practice rehabilitation. The main contributors of pavement failure on the Sunshine Coast appear to be inadequate pavement structure for current traffic loads, asphalt and bitumen fatigue and subgrade movement due to moisture content and highly expansive / collapsible soils.

- Chemical stabilisation provides a cost effective pavement rehabilitation solution. Consideration needs to be given to road pavements with marginal quality or insufficient thickness of existing pavement materials and how these can be treated or work practices altered to allow more stabilisation projects.

- Evaluate the effectiveness of recently implemented polymer modified seals on stabilisation works and their improvement to reflective cracking.

- Current pavement rehabilitation and construction methods used by Council, and that which is specified by Council's planning scheme vary considerably, leading to premature pavement rehabilitation on recent developments and high ongoing maintenance costs for Council.

- Implementing a program to validate and update data stored within Council's pavement management system to increase the accuracy of data and in-turn assist with timely intervention, through frequent road condition surveys.

9.2 Further Research

Continued improvement to pavement rehabilitation practices requires ongoing research into technologies being developed and trialled around the world. Further research or development that would enhance pavement rehabilitation treatments within the Sunshine Coast could include:

- Development of a subgrade material map for the Sunshine Coast and a dataset on previously successful pavement rehabilitation treatments.

- Increase the use of recycled materials in pavements i.e. crumbed rubber, recycled asphalt pavement (RAP), crushed glass, existing base or sub-base materials for reuse in lower pavement layers.

- Limiting moisture infiltration into road pavements through the installation of subsoil drainage, accompanied by routine subsoil drainage maintenance.

- Evaluate the effectiveness of polymer modified bitumen seals i.e. SAMI seals when used on stabilised pavements and their prevention of shrinkage cracking.

- Continued use of geosynthetics and further education of work crews on installation methods and standards.

9.3 Conclusion

The major aim of this project was to analyse the current road pavement rehabilitation methods used on local government roads within the Sunshine Coast region. Sunshine Coast Council has been proactive in its approach to pavement rehabilitation, trialling new technologies and searching for cost saving initiatives where appropriate. Council practices are generally sound and in accordance with the latest Austroads and Department of Transport and Main Roads standards and specifications, aligning with current world best practice for pavement design and rehabilitation.

The effectiveness of pavement rehabilitation treatments are case-specific, however, Sunshine Coast practices could be improved by considering sustainable rehabilitation methods including stabilisation, plant mixed foam bitumen and further use of geosynthetics. Council should continue to build its relationship with the University of the Sunshine Coast's Engineering Department and internal quarry to trial recycled materials in pavement and asphalt layers, including but not limited to the use of recycled asphalt pavement (RAP), crushed glass, crumbed rubber and modified bitumen and asphalt products. Further recommendations include aligning the Sunshine Coast Council Planning Scheme more accurately with Austroads and Department of Transport and Main Roads documentation, accompanied with internal practices for specific subgrade conditions.

References

Agarwal, P. R. S. &. S. R., 2012. Drainage and Flexibile Pavement Performance. *International Journal of Engineering Scince and Technology,* pp. 1308-1311.

Austroads, 2007. *Guide to Asset Management Part 5B: Roughness,* Sydney: Austroads Inc.

Austroads, 2007. *Guide to Asset Management Part 5C: Rutting,* Sydney: Austroads Inc.

Austroads, 2007. *Guide to Asset Management Part 5D: Strength,* Sydney: Austroads Inc.

Austroads, 2009. *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design,* Sydney: Austroads Inc..

Black, P. &. H. R., 1999. Performance of Geotextile Separators Five Years after Installation. *Journal of Geotechnical and Geoenvironmental Engineering,* pp. 404-412.

Casagrande, A., 1958. Notes of the design of the liquid limit devise. *Geotechnique,* pp. 84-91.

Ceylan, H. S. V. &. W. D., 2008. *Design Guide for Improved Quality of Roadway Wubgrades and Subbases,* Iowa: SUDAS.

Chen, D. S. T. B. J. L. D. &. Z. F., 2003. Forensic Evaluation of Premature Failures of Texas Specific Pavement Study - 1 Sections. *Journal of Performance of constructed Facilities,* pp. 267-274.

Chen, F., 1988. *Foundations on Expansive Soils.* , Elsevier: s.n.

Christopher, B. B. R. &. P. S., 2001. *Geosynthetic Reinforcement in Roadway Sections,* s.l.: NCHRP.

Das, B., 2010. *Principles of Geotechnical Engineering: 7th Edition.* Stamford CT: Cengage Learning.

Driscoll, R. M. a. C., 2001. Shrinking and Swelling of Clays. *Problematic Soils Symposium,* pp. 53-66.

Engineers Australia, 2014. *Engineers Australia: Code of Ethics.* [Online]
Available at:
http://www.ipwea.com/Code%20of%20Ethics_Consultation%20Draft09.pdf

Gillespie, T., 2014. *Everything you always wanted to know about IRI, but were afraid to ask!.* [Online]
Available at: http://www.umtri.umich.edu/divisionPage.php?pageID=64

Houston, S., 1988. Pavement Problems Caused by Collapsible Subgrades. *Transport Engineering,* pp. 673 - 683.

Howayek, A. H. P. B. R. a. S. M., 2011. *Identification and Behavior of Collapsible Soils,* West Lafayette: Joint Transportation Research Program.

Jones, L. &. J. I., 2012. Expansive Soils. In: *ICE Manual of Geotechnical Engineering.* London: ICE Publishing, pp. 413-441.

Lovelace, A. B. W. a. L. R., 1982. *A Test Section for the Stabilisazation of Collapsible Soils on Interstate 25,* s.l.: New Mexico State Highway Department.

Lytton, R. L. P. D. E. a. M., 1993. *An Integrated Model of the Climatic Effects on Pavements,* Virginia: FHWA.

Mooney, M. M. G. T. S. &. B. W., 2000. Importance of Invasive Measures in Assessment of Existing Pavements. *Journal of Performance of Constructed Facilities,* p. Vol. 14.

Nelson, J. a. M. D., 1992. *Expansive Soils: Problems and Practice in Foundation and Pavement Engineering.* New York: John Wiley and Sons.

Petry, T. &. L. D., 2002. Review of Stabilisation of clays and Expansive soils in Pavements and lightly Loaded Structures. *Journal of Materials in Civil Engineering,* pp. 447-460.

Ramanujam, J. &. J. J., 2007. Characterization of Foamed-bitumen Stabilisation. *International Journal of Pavement Engineering,* pp. 111-122.

Rogers, G. &. R. K., 1994. Mitigation Measures for small Structures on Collapsible Alluvial Soils. *Geotechnical Engineering,* pp. 1533-1553.

Services, R. P., 2014. *RoadScout 3 Pavement Monitoring Unit,* s.l.: Radar Portal Services.

State of New York Department of Transportation, 2007. *Geotechnical Test Method: Test Method for Liquid Limit, Plastic Limit, and Plasticity Index,* New York: Geotechnical Engineering Bureau.

TMR, 2012. *Pavement Rehabilitation Manual,* Brisbane: Department of Transport and Main Roads.

Tung, W. a. U. E., 2012. *Ground Penetrating Radar (GPR) - A Tool for Pavement Evaluation and Design,* Perth: ARRB.

Uzan, J. a. L. R., 1989. Experimnt Design Approach to Nondestructuve Testing of Pavements. *Journal of Transport Engineering,* pp. 505-520.

Walters, K., 2008. *Investigation of Cosntruction Practices and Test Procedures for Road Pavements on Expansive Subgrades,* Toowoomba: University of Southern Queensland.

Waters, T., 2002. *Moistures in Pavements,* Vermount South: ARRB Group Limited.

Willmott, W., 2007. *Roacks and Landscape of the Sunshine Coast.* 2nd ed. Brisbane: Geological Society of Australia, Queensland Division.

Appendix A – Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research Project

PROJECT SPECIFICATION

FOR:	THOMAS SANDERS
TOPIC:	EFFECTIVE ROAD PAVEMENT REHABILITATION FOR LOCAL GOVERNMENT ROADS WITHIN THE SUNSHINE COAST REGION.
SUPERVISOR:	Prof. Ron Ayers
ENROLMENT:	ENG4111 – S1, 2014
	ENG4112 – S2, 2014
PROJECT AIM:	This project seeks to critically evaluate current pavement rehabilitation methods used within the Sunshine Coast Council and to propose alternative practices.
SPONSORSHIP:	Sunshine Coast Council
PROGRAMME:	**Issue B, 13th October 2014**

1. Research current Sunshine Coast Council, Australian and international pavement rehabilitation design methods.
2. Research geological and environmental history of the Sunshine Coast region.
3. Collect soil test information for subgrade conditions within the Sunshine Coast. Evaluate the subgrade materials and their properties.
4. Analyse the effectiveness of current pavement rehabilitation practices within the Sunshine Coast through the use of laser survey data, falling weight deflectometer testing and 'As Constructed' data.
5. Critically evaluate the effectiveness and performance of current Sunshine Coast Council pavement rehabilitation design practices against world's best practices.
6. Propose improvements to Sunshine Coast Council pavement rehabilitation practices.
7. Present results and recommendations in the required oral and written formats.

As time permits:

1. Investigate results from pavement monitoring technology to be installed within a foam bitumen pavement rehabilitation project. The pavement monitoring technology is being developed by the Sunshine Coast Council and the University of the Sunshine Coast and monitors strain, temperature and moisture content of pavements in respect to air temperature, rainfall and radiation characteristics.

AGREED:

______(Student) R Ayers ______(Supervisor)

14 / 10 / 2014 13 / 10 / 2014

Appendix B – FWD and Rutting Maps

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 21 OF 57

Sunshine Coast. COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 Created by: sh027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 22 OF 57

Sunshine Coast. COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

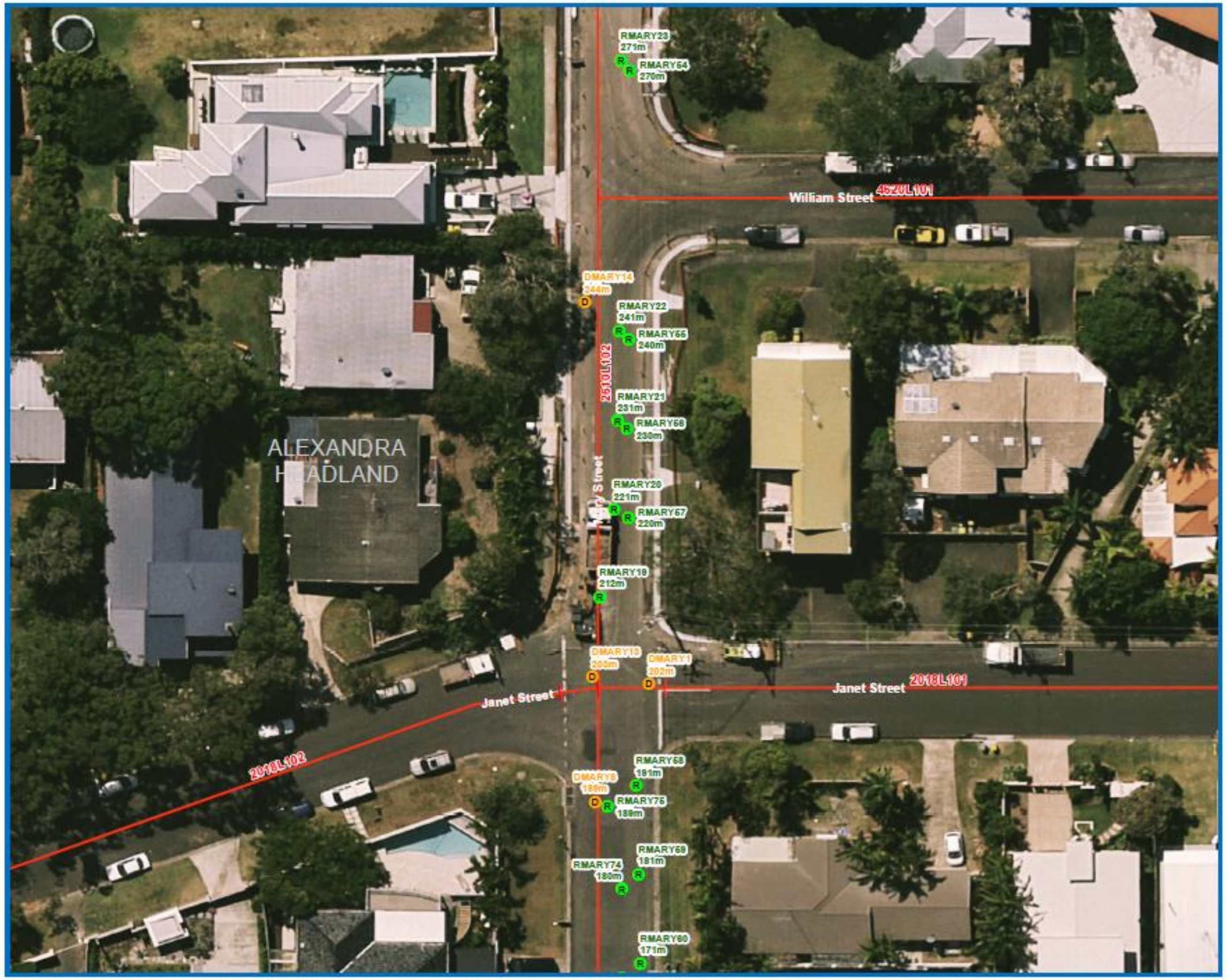
N

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 23 OF 57



Sunshine Coast. COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 24 OF 57

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

Sunshine Coast. COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 25 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 29 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

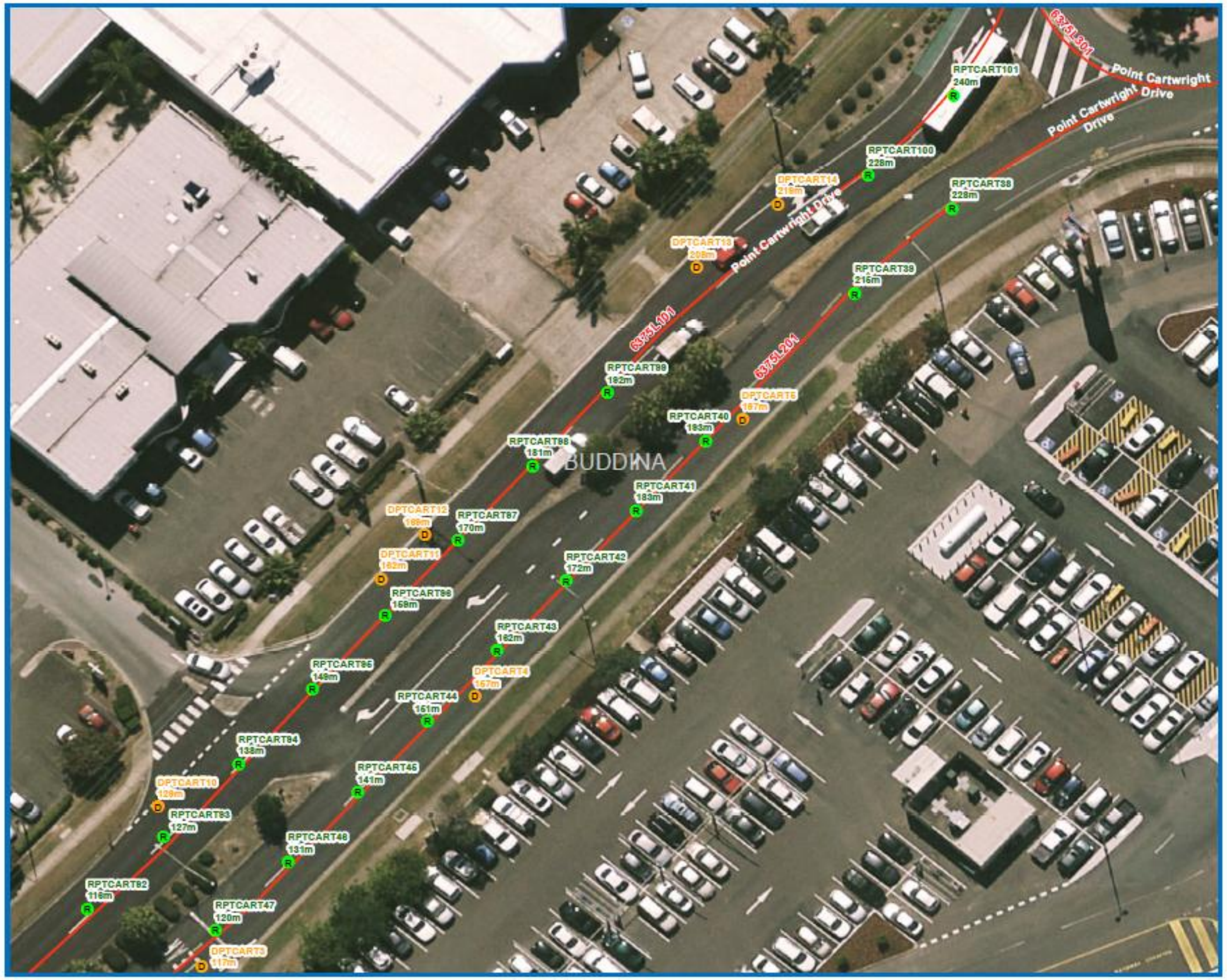
Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 30 OF 57



Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 *Created by: sh027*

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 31 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 33 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

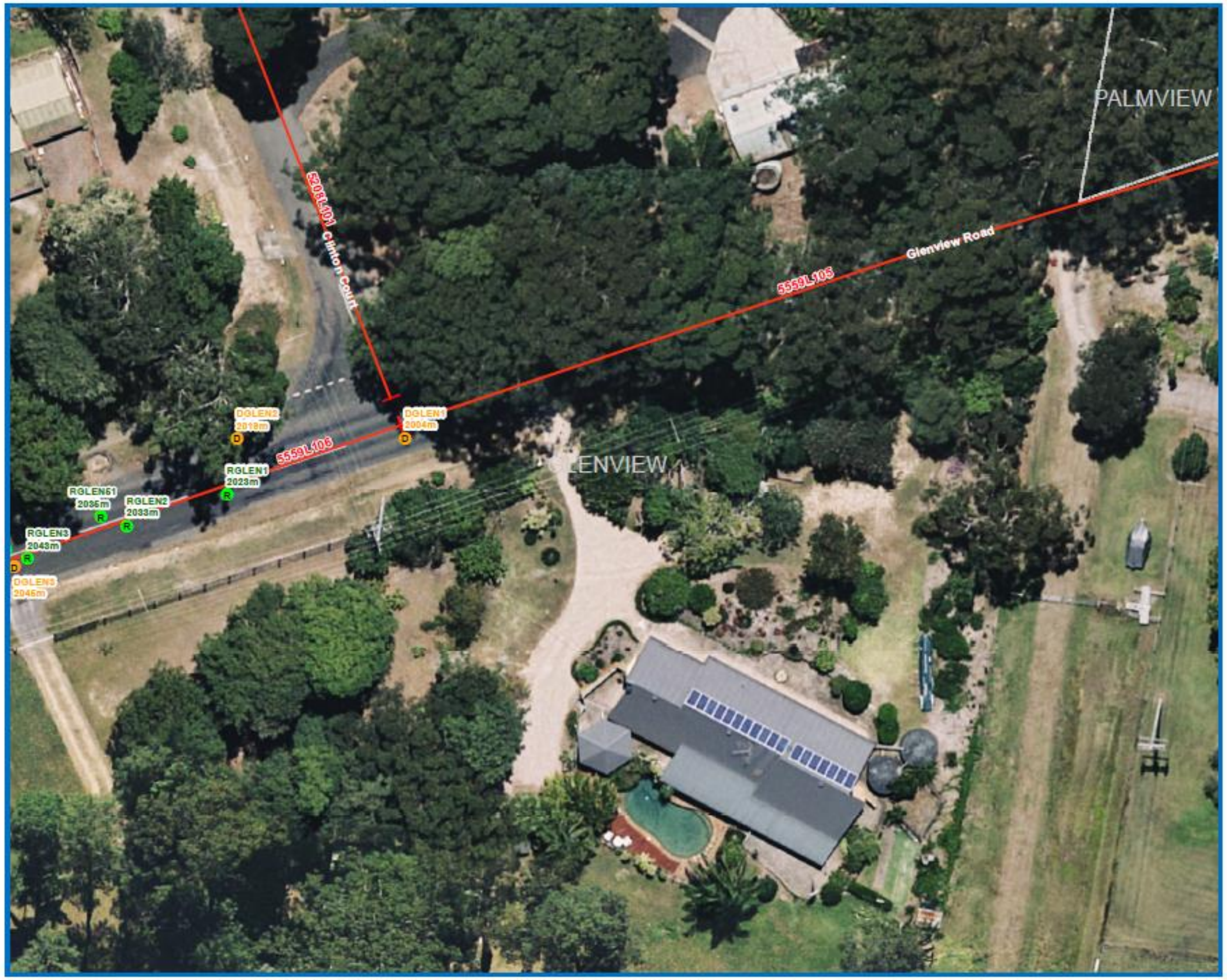
Date: Monday, 15 September 2014

Created by: afs027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 34 OF 57



Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer
While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 35 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

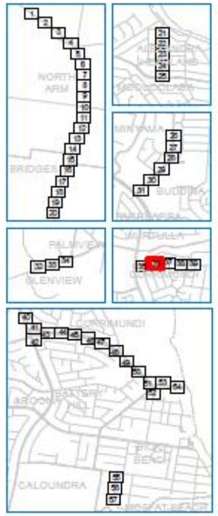
ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 36 OF 57

Sunshine Coast Council

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry



Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 37 OF 57

Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 Created by: afs027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 38 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 39 OF 57

Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

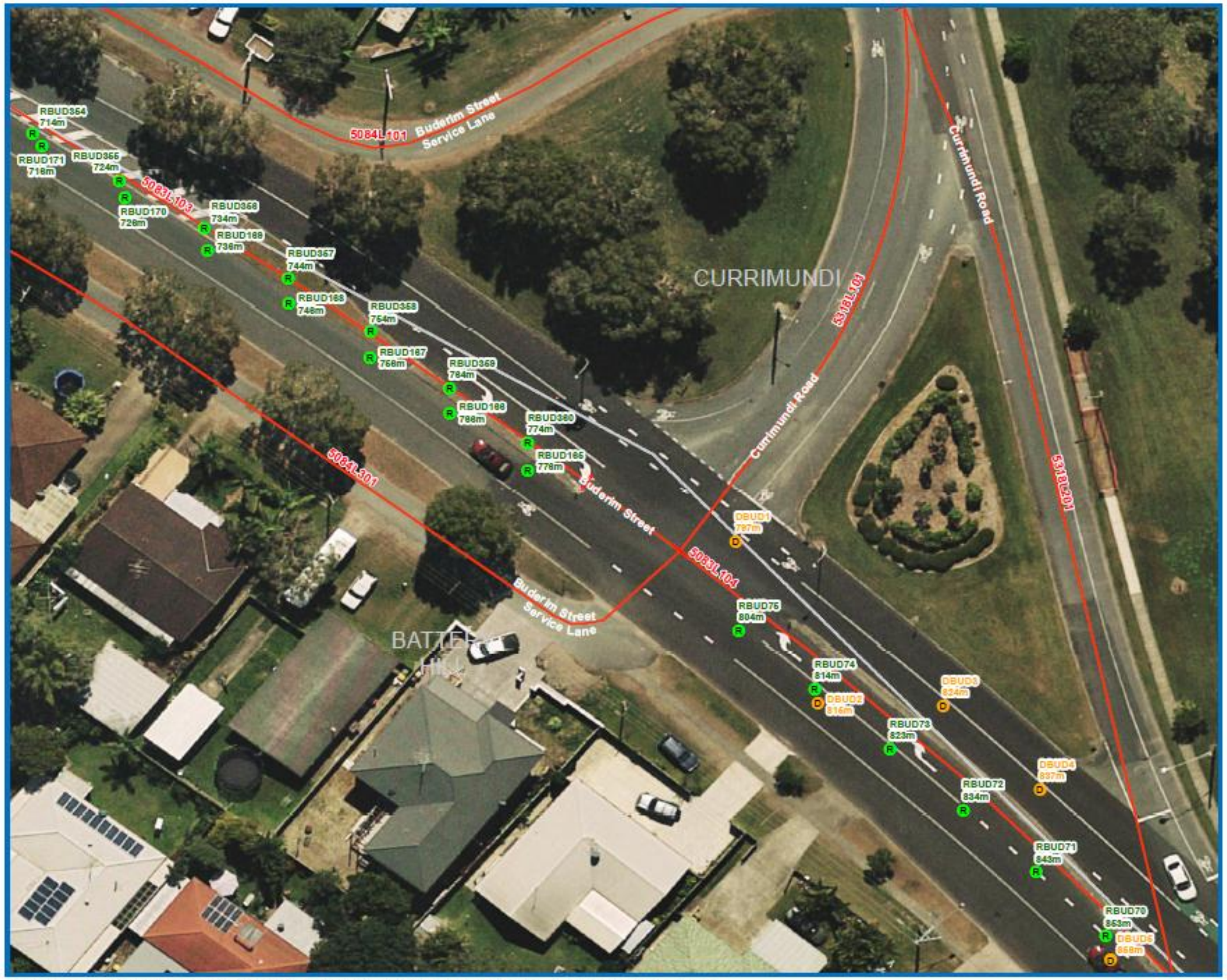
Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 48 OF 57



Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

N

Date: Monday, 15 September 2014 *Created by: a5027*

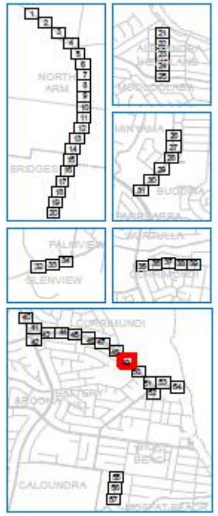
Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry



Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 50 OF 57

Sunshine Coast COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014

Created by: ah027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 51 OF 57

Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 55 OF 57

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

Sunshine Coast. COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, either the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ab027*

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 56 OF 57

Sunshine Coast. COUNCIL

Legend

Road Segment (Segment ID)

FWD Survey Location (Record ID/ Chainage [m])

Rutting Survey Location (Record ID/ Chainage [m])

Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday 15 September 2014 Created by: afs027

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

ROAD FWD/ RUTTING SURVEY ANALYSIS

SEPT 2014
SHEET 57 OF 57

Sunshine Coast COUNCIL

Legend

- Road Segment (Segment ID)
- FWD Survey Location (Record ID/ Chainage [m])
- Rutting Survey Location (Record ID/ Chainage [m])
- Locality Bdry

Disclaimer

While every care is taken to ensure the accuracy of this product, neither the Sunshine Coast Council nor the State of Queensland makes any representations or warranties about its accuracy, reliability, completeness or suitability for any particular purpose and disclaims all responsibility and all liability (including without limitation, liability in negligence) for all expenses, losses, damages (including indirect or consequential damage) and costs that may occur as a result of the product being inaccurate or incomplete in any way or for any reason.

Geocentric Datum of Australia 1994 (GDA94)

0 5 10 15 20 m

Scale: 1:400

Date: Monday, 15 September 2014 *Created by: ah027*

Path: W:\Common\Geo\Projects\140163_RoadRuttingAndFWDDeflectionChainages2014\Maps\RoadRuttingFWDChainageAnalysis_20140915.mxd

Appendix C – Surface Defect Validation Sheets

Sunshine Coast Council

ROAD DEFECT MAPPING SHEET

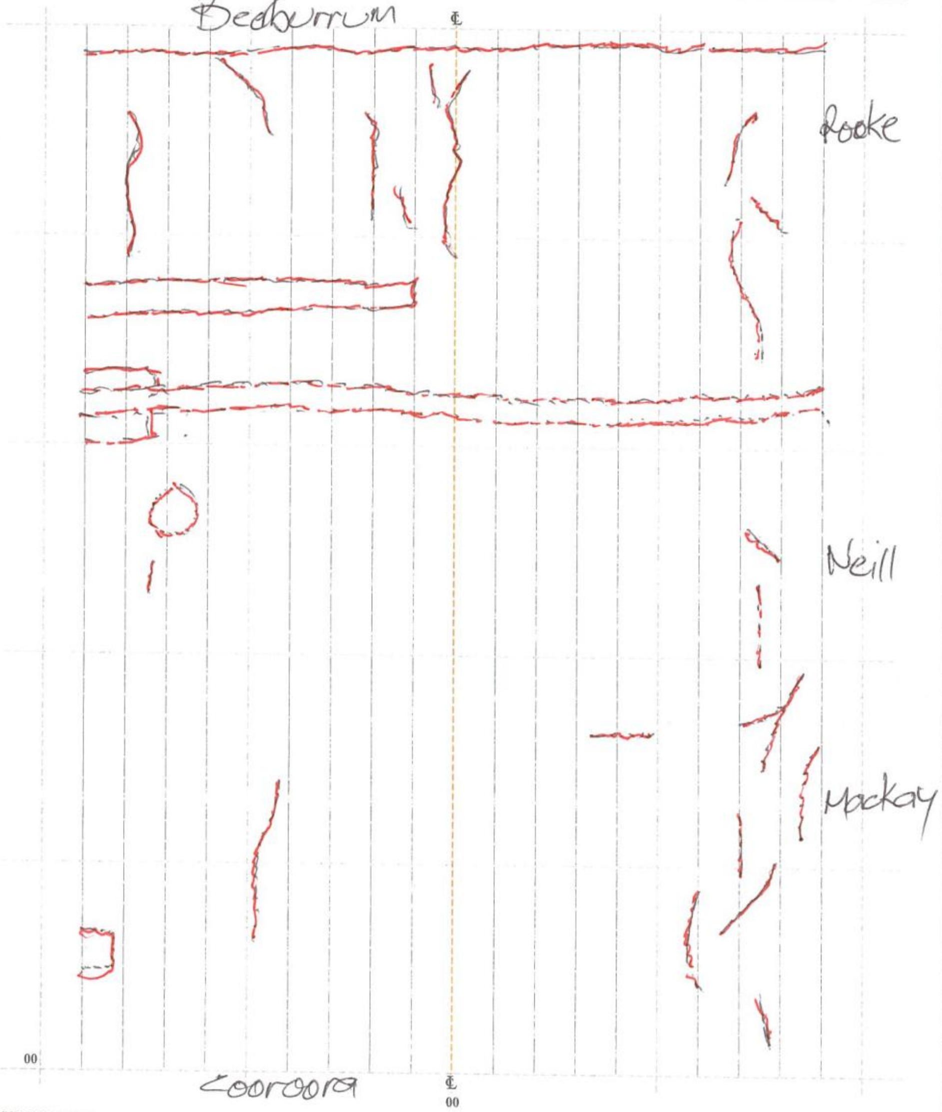
Road Name: Henzell St **Suburb:** Dicky Beach

Road No: 5654 **Block Description:** Cooroora - Beerburrum

Orientation

Sheet:

Block No: 101 **Inspector:** **Date:**



DEFORMATION		**CRACKING**					
DR	Rutting	CR	Crocodile Cracking	SR	Ravelling	HO	Pothole
DS	Shoving	CB	Block Cracking	SS	Stripping	PA	Patch Failure
DD	Depression	C	Cracking 235m 235m	SD	Delamination	P	Patch
DP	Pumping	CC	Crescent (shear) Cracking				

Sunshine Coast Council

ROAD DEFECT MAPPING SHEET

Road Name: Henzell St **Suburb:** Dicky Beach

Road No: 5654 **Block Description:** McDonald - Cooroora

Block No: 102 **Inspector:** **Date:**

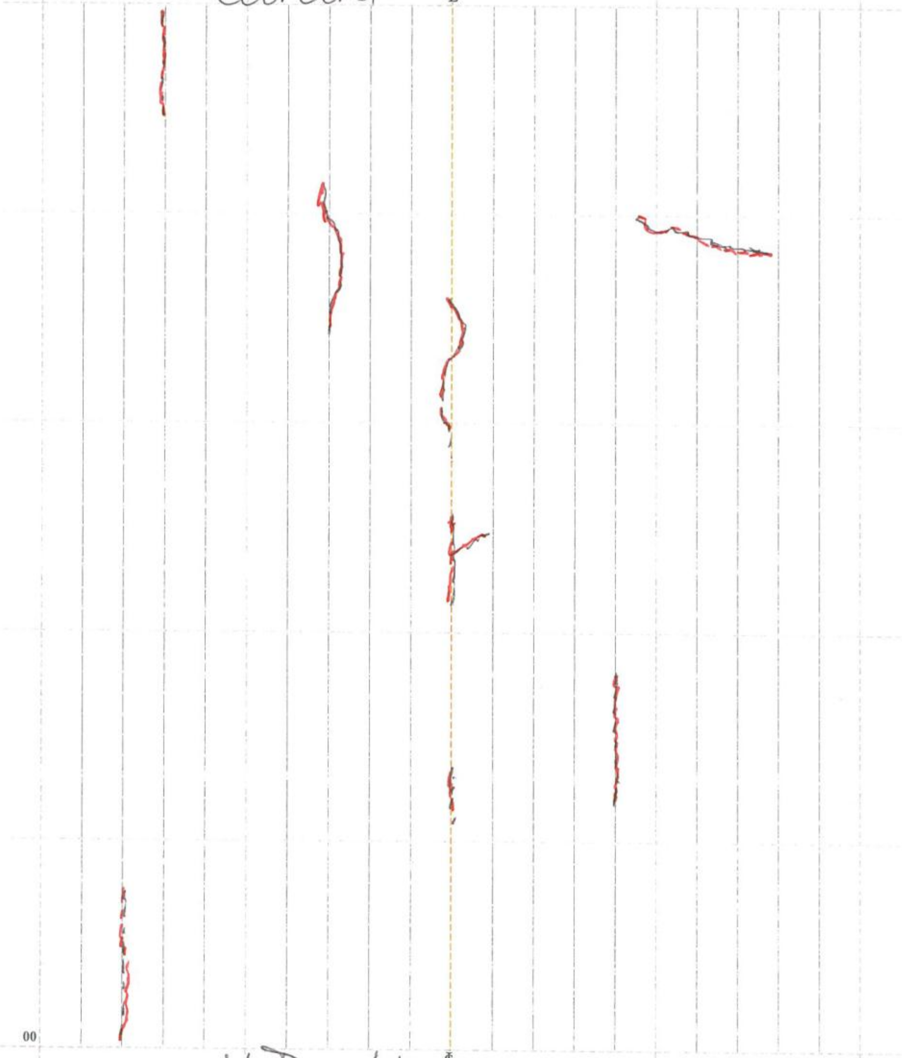
N

Orientation

Sheet:

Cooroora

℄



00

McDonald

℄

00

DEFORMATION		**CRACKING**					
DR	Rutting	CR	Crocodile Cracking	SR	Ravelling	HO	Pothole
DS	Shoving	CB	Block Cracking	SS	Stripping	PA	Patch Failure
DD	Depression	C	Cracking 34 m 34m	SD	Delamination	P	Patch
DP	Pumping	CC	Crescent (shear) Cracking				

C:\Users\sh001\AppData\Local\Microsoft\Windows\Temporary Internet Files\Content.Outlook\HPF42W8C\Road Defect Mapping Sheet.xls

Sunshine Coast Council

ROAD DEFECT MAPPING SHEET

N

Road Name: Henzell St **Suburb:** Dicky Beach

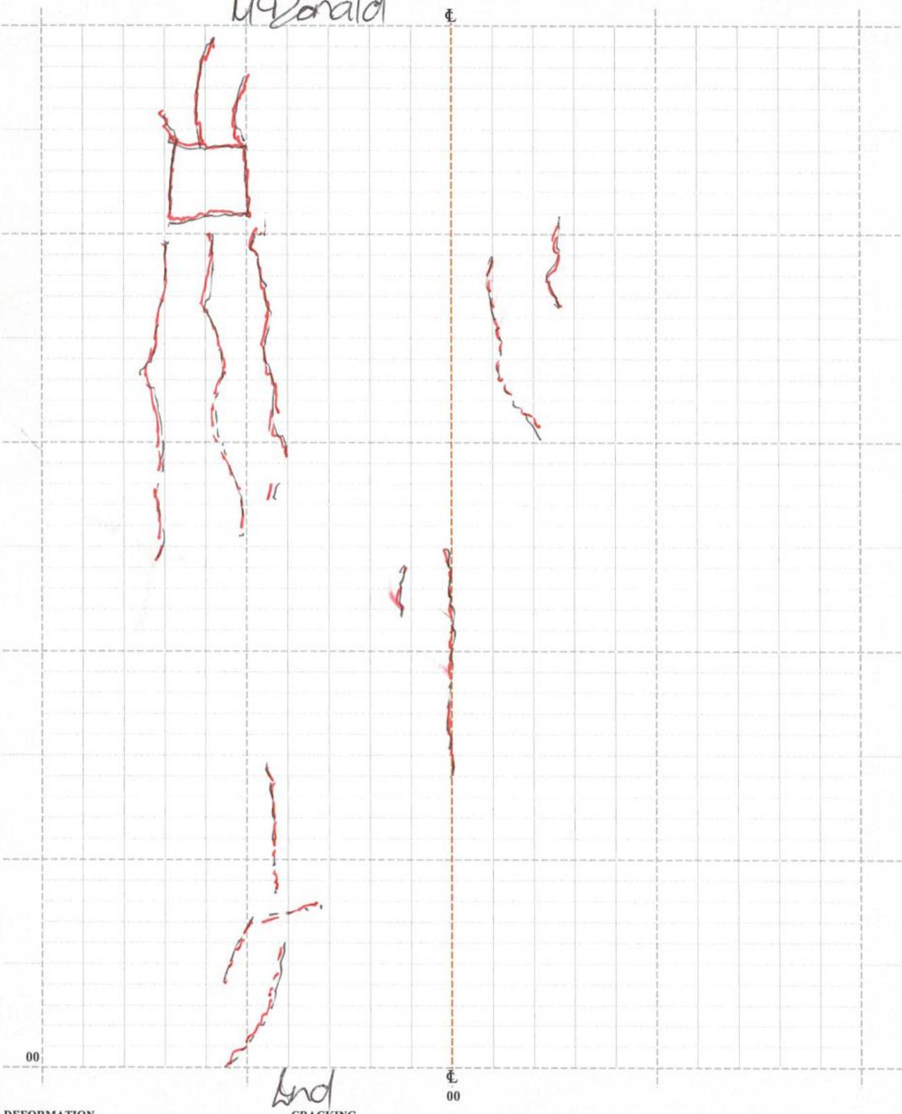
Road No: 5654 **Block Description:** End - McDonald

Orientation

Sheet:

Block No: 103 **Inspector:** **Date:**

McDonald



End

DEFORMATION		**CRACKING**					
DR	*Rutting*	*CR*	*Crocodile Cracking*	*SR*	*Ravelling*	*HO*	*Pothole*
DS	*Shoving*	*CB*	*Block Cracking*	*SS*	*Stripping*	*PA*	*Patch Failure*
DD	*Depression*	*C*	*Cracking*	*SD*	*Delamination*	*P*	*Patch*
DP	*Pumping*	*CC*	*Crescent (shear) Cracking*				

52 m

C:\Users\sh001\AppData\Local\Microsoft\Windows\Temporary Internet Files\Content.Outlook\HPF42WHC\Road Defect Mapping Sheet.xls

Sunshine Coast Council

ROAD DEFECT MAPPING SHEET

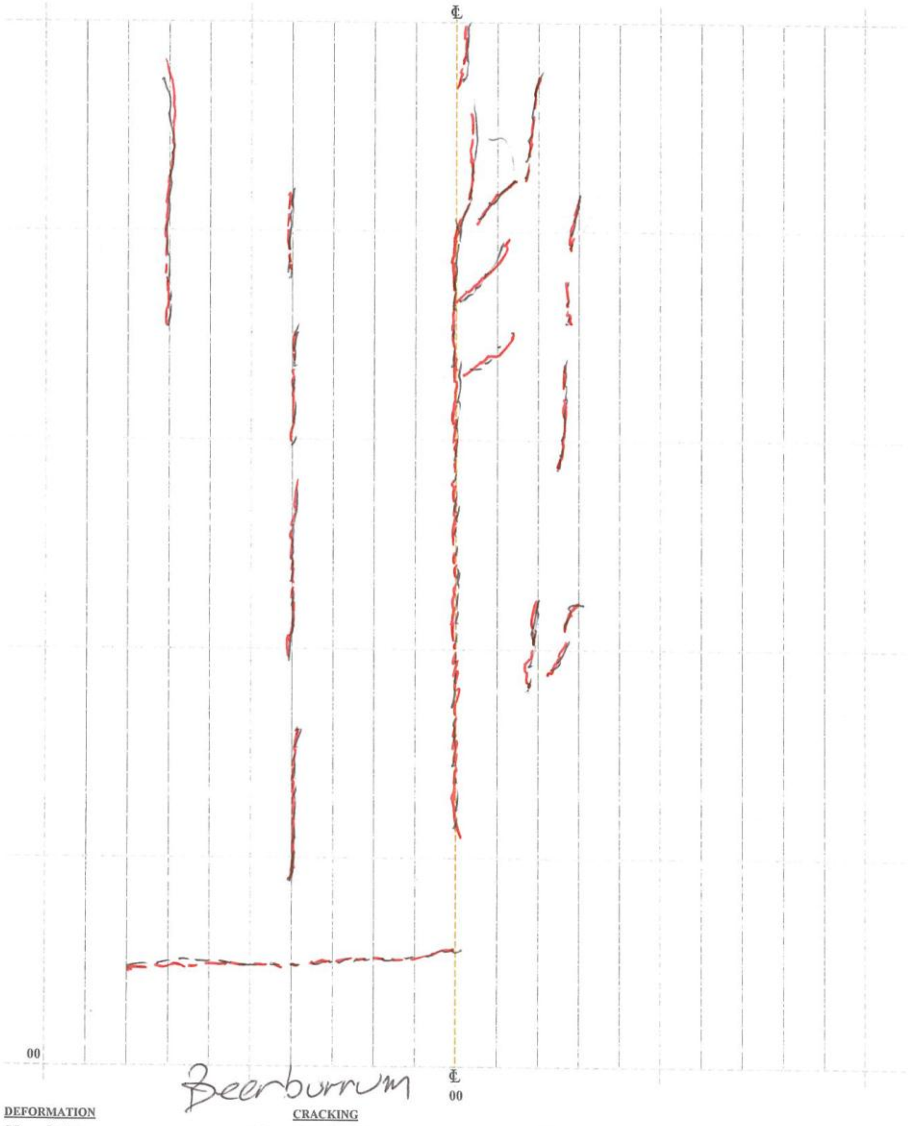
Road Name: Wilson Ave **Suburb:** Dicky Beach

Road No: 6908 **Block Description:** Beerburrum - Rooke

Block No: 101 **Inspector:** **Date:**

Orientation: S ↑

Sheet:



Beerburrum

DEFORMATION		CRACKING					
DR	Rutting	CR	Crocodile Cracking	SR	Ravelling	HO	Pothole
DS	Shoving	CB	Block Cracking	SS	Stripping	PA	Patch Failure
DD	Depression	C	Cracking 83m	SD	Delamination	P	Patch
DP	Pumping	CC	Crescent (shear) Cracking 83m				

Sunshine Coast Council

ROAD DEFECT MAPPING SHEET

1/5

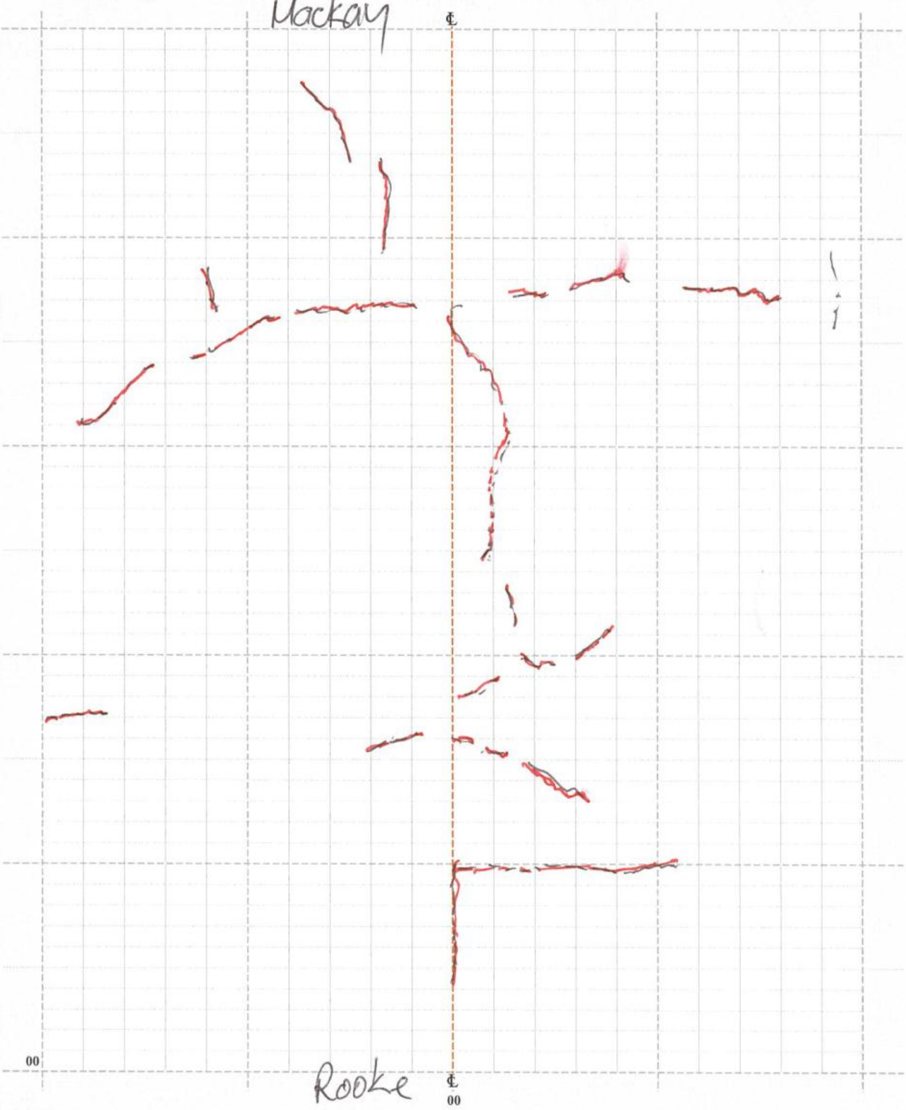
Road Name: Wilson Ave **Suburb:** Dicky Beach

Road No: 6908 **Block Description:** Rooke - Mackay

Orientation

Sheet:

Block No: 202 **Inspector:** **Date:**



DEFORMATION

DR *Rutting*

DS *Shoving*

DD *Depression*

DP *Pumping*

CRACKING

CR *Crocodile Cracking*

CB *Block Cracking*

C *Cracking* 65m 65m

CC *Crescent (shear) Cracking*

SR *Ravelling*

SS *Stripping*

SD *Delamination*

HO *Pothole*

PA *Patch Failure*

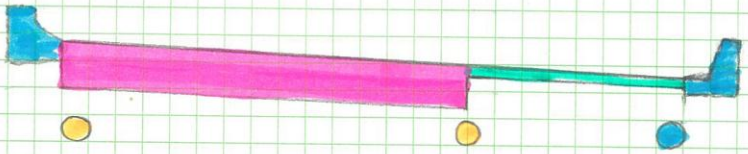
P *Patch*

Appendix D – Beerburrum St Pavement Option 2 Sketch

7/8/13

Beerburrum st. Cross Section

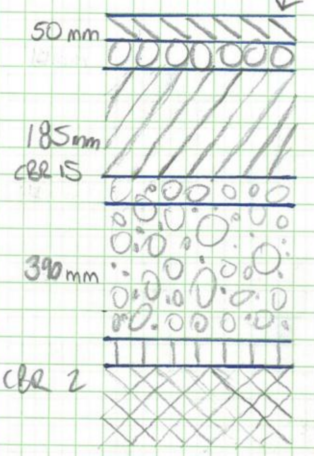
Combi Grid Option



Legend

- Pavement 'A' running lanes
- Pavement 'B' parking lanes
- Existing Features not to be touched
- New subsoil drainage.

Pavement 'A'



Pavement 'B'

Note:

* PAVEMENT DESIGN TO BE USED FOR ESTIMATION ONLY. REFER ENGINEERS DESIGN FOR CONSTRUCTION.

Appendix E – Financial Reports Rehabilitation Options

Gannawarra St - Pavement Rehabiliation

Exp Type	Description	Units		Actual Expenditure 11/12		Actual Expenditure 12/13		Actual Expenditure 13/14		Actual Expenditure Whole of Life		Total Project Cost WOL
0000	Non Specific Activity	Item	$	3,089	$	25,574	$	-	$	28,663	$	28,663
0001	Budget	Item	$	-	$	-	$	-	$	-	$	-
1101	Site Facilities & Camp	Lump sum	$	-	$	12,318	$	2,171	$	14,489	$	14,489
1201	Provision for Traffic	Lump sum	$	-	$	57,828	$	-	$	57,828	$	57,828
1316	Control Of Erosion And Sedimentation	Lump sum	$	-	$	865	$	-	$	865	$	865
2100	Removal or Demolition of Existing Compon	Lump sum	$	-	$	35,402	$	-	$	35,402	$	35,402
2404	Concrete Kerb & Channel	m	$	-	$	25,777	$	-	$	25,777	$	25,777
2405	Concrete Kerb Crossings	m	$	-	$	16,099	$	-	$	16,099	$	16,099
2413	Concrete Gullies	Each	$	-	$	10,528	$	-	$	10,528	$	10,528
2501	Sub-Soil Drains Type B	M	$	-	$	63,101	$	-	$	63,101	$	63,101
3100	Earthworks, Excav/Embank/Subgrade Treatments	M3	$	-	$	75,513	$	-	$	75,513	$	75,513
3108	Excavation & Disposable of Unsuitable Material	Lump sum	$	-	$	10,290	$	-	$	10,290	$	10,290
3800	Landscaping Works	M2	$	-	$	14,958	$	-	$	14,958	$	14,958
4103	Base, Unbound Pavement	M3	$	-	$	52,659	$	-	$	52,659	$	52,659
4104	Subbase, Unbound Pavement	M3	$	-	$	61,630	$	-	$	61,630	$	61,630
5103	One Coat Seal	M2	$	-	$	19,677	$	-	$	19,677	$	19,677
5500	Asphalt Pavements & Seals	Tonne	$	-	$	55,035	$	-	$	55,035	$	55,035
6300	Pavement Marking	M	$	-	$	7,023	$	-	$	7,023	$	7,023
8885	Portable Long Service Leave Levy	Lump sum	$	-	$	-	$	-	$	-	$	-
8901	Quality Assurance	Lump sum	$	-	$	1,490	$	-	$	1,490	$	1,490
8925	Consultant Fees Design	Item	$	-	$	10,528	$	-	$	10,528	$	10,528
9005	Location Of Existing Services (Incl. Lia	Lump sum	$	-	$	4,801	$	-	$	4,801	$	4,801
9050	After Hours "Call Out"	Provisional	$	-	$	149	$	-	$	149	$	149
9052	Restore Household Drains To K&C	Item	$	-	$	-	$	-	$	-	$	-
9053	Airport Safety Officers	Lump sum	$	-	$	-	$	3,208	$	3,208	$	3,208
9054	Electrical	Lump sum	$	-	$	-	$	2,101	$	2,101	$	2,101
9060	Tip Fees	Lump sum	$	-	$	7,552	$	-	$	7,552	$	7,552
9092	Concrete & Asphalt Cutting	M	$	-	$	1,951	$	-	$	1,951	$	1,951
9100	Supervision of Construction (Foreman)	Lump sum	$	-	$	11,390	$	-	$	11,390	$	11,390
9101	Design Fees (Actual Internal Fee)	Lump sum	$	-	$	-	$	-	$	-	$	-
9103	Management Reserve Funds	Lump sum	$	-	$	-	$	-	$	-	$	-
9104	Project Management Fees	Lump sum	$	-	$	15,911	$	59	$	15,970	$	15,970
9105	Capital Overhead Allocation	Lump sum	$	-	$	-	$	113	$	113	$	113
9999	Capitalisation Adjustments	$0.00	$	-	$	-	$	-	$	-	$	-
				$3,088.82		$598,048.31		$7,651.93		$608,789.06		**$608,789.06**

Lyon St - Pavement Rehabiliation

Exp Type	Description	Units	Actual Expenditure 11/12	Actual Expenditure 12/13	Actual Expenditure 13/14	Actual Expenditure Whole of	Total Project Cost WOL
0000	Non Specific Activity	Item	$ 3,176.35	$ 24,431.71	$ -	$ 27,608.06	$ 27,608.06
0001	Budget	Item	$ -	$ -	$ -	$ -	$ -
0808	January Rain 2013	Lump sum	$ -	$ 22,646.99	$ -	$ 22,646.99	$ 22,646.99
1101	Site Facilities & Camp	Lump sum	$ -	$ 7,302.97	$ -	$ 7,302.97	$ 7,302.97
1201	Provision for Traffic	Lump sum	$ -	$ 27,703.94	$ -	$ 27,703.94	$ 27,703.94
1316	Control Of Erosion And Sedimentation	Lump sum	$ -	$ 2,515.49	$ -	$ 2,515.49	$ 2,515.49
2100	Removal or Demolition of Existing Compon	Lump sum	$ -	$ 26,810.35	$ -	$ 26,810.35	$ 26,810.35
2201	Supply of Pipe Culvert Components	M	$ -	$ 2,345.88	$ -	$ 2,345.88	$ 2,345.88
2263	Installation of Pipes 300 dia - 450 dia	m	$ -	$ 1,435.50	$ -	$ 1,435.50	$ 1,435.50
2404	Concrete Kerb & Channel	m	$ -	$ 18,908.82	$ -	$ 18,908.82	$ 18,908.82
2405	Concrete Kerb Crossings	m	$ -	$ 23,937.23	$ -	$ 23,937.23	$ 23,937.23
2413	Concrete Gullies	Each	$ -	$ 8,734.80	$ -	$ 8,734.80	$ 8,734.80
2414	Concrete Access Chambers	Item	$ -	$ 1,054.20	$ -	$ 1,054.20	$ 1,054.20
2501	Sub-Soil Drains Type B	M	$ -	$ 18,295.66	$ -	$ 18,295.66	$ 18,295.66
3100	Earthworks, Excav/Embank/Subgrade Treatments	M3	$ -	$ 48,708.20	$ -	$ 48,708.20	$ 48,708.20
3108	Excavation & Disposable of Unsuitable Material	Lump sum	$ -	$ -	$ -	$ -	$ -
3304	Geotextile	M2	$ -	$ 6,056.25	$ -	$ 6,056.25	$ 6,056.25
3602	Entrances To Private Properties	Lump sum	$ -	$ 8,335.68	$ -	$ 8,335.68	$ 8,335.68
3800	Landscaping Works	M2	$ -	$ 3,053.85	$ -	$ 3,053.85	$ 3,053.85
4103	Base, Unbound Pavement	M3	$ -	$ 49,444.40	$ -	$ 49,444.40	$ 49,444.40
4104	Subbase, Unbound Pavement	M3	$ -	$ 55,645.40	$ -	$ 55,645.40	$ 55,645.40
5500	Asphalt Pavements & Seals	Tonne	$ -	$ 46,218.03	$ -	$ 46,218.03	$ 46,218.03
6120	General Road Guidance Systems	Lump sum	$ -	$ 73.81	$ -	$ 73.81	$ 73.81
8885	Portable Long Service Leave Levy	Lump sum	$ -	$ 325.00	$ -	$ 325.00	$ 325.00
8901	Quality Assurance	Lump sum	$ -	$ 1,058.00	$ -	$ 1,058.00	$ 1,058.00
8925	Consultant Fees Design	Item	$ -	$ 17,820.00	$ -	$ 17,820.00	$ 17,820.00
9050	After Hours "Call Out"	Provisional	$ -	$ 79.90	$ -	$ 79.90	$ 79.90
9060	Tip Fees	Lump sum	$ -	$ 9,771.11	$ -	$ 9,771.11	$ 9,771.11
9092	Concrete & Asphalt Cutting	M	$ -	$ 1,112.10	$ -	$ 1,112.10	$ 1,112.10
9100	Supervision of Construction (Foreman)	Lump sum	$ -	$ 14,565.19	$ -	$ 14,565.19	$ 14,565.19
9101	Design Fees (Actual Internal Fee)	Lump sum	$ -	$ 4,868.01	$ -	$ 4,868.01	$ 4,868.01
9103	Management Reserve Funds	Lump sum	$ -	$ -	$ -	$ -	$ -
9104	Project Management Fees	Lump sum	$ -	$ 20,579.07	$ 62.95	$ 20,642.02	$ 20,642.02
9105	Capital Overhead Allocation	Lump sum	$ -	$ -	$ 0.95	$ 0.95	$ 0.95
			$3,176.35	$473,837.54	$63.90	$477,077.79	**$477,077.79**

Mary St - Pavement Rehabilitation

Exp Type	Description	Units	Current Estimate Units	Current Estimate Rate	Current Estimate Amount	Actual Expenditure 12/13	Actual Expenditure 13/14	Actual Expenditure Whole of	Total Project Cost WOL
0000	Non Specific Activity	Item	0	$0.00	0	14,962	0	14,962	14,962
1101	Site Facilities & Camp	Lump sum	1	$24,000.00	24,000	8,727	18,532	27,259	27,259
1201	Provision for Traffic	Lump sum	1	$25,000.00	25,000	22,336	83,252	105,588	105,588
1316	Control Of Erosion And Sedimentation	Lump sum	1	$2,000.00	2,000	163	3,399	3,562	3,562
2100	Removal or Demolition of Existing Compon	Lump sum	1	$11,520.00	11,520	0	21,099	21,099	21,099
2403	Concrete Channel	m	729	$70.00	51,030	0	14,001	14,001	14,001
2404	Concrete Kerb & Channel	m	96	$300.00	28,800	0	59,761	59,761	59,761
2405	Concrete Kerb Crossings	m	0	$0.00	0	0	28,328	28,328	29,074
2413	Concrete Gullies	Item	4	$2,000.00	8,000	0	8,361	8,361	8,361
2501	Sub-Soil Drains Type B	M	1104	$30.00	33,120	78,451	9,977	88,428	88,428
3100	Earthworks, Excav/Embank/Subgrade Treatments	M3	1080	$48.89	52,801	8,779	198,393	207,172	207,172
3108	Excavation & Disposable of Unsuitable Material	M3	100	$40.00	4,000	0	3,661	3,661	3,661
3304	Geotextile	M3	5300	$6.00	31,800	24,225	34,871	59,096	59,096
3602	Entrances To Private Properties	Lump sum	120	$100.00	12,000	0	2,372	2,372	2,372
3800	Landscaping Works	Lump sum	0	$0.00	0	0	4,355	4,355	4,355
4103	Base, Unbound Pavement	M3	1082	$100.00	108,200	0	179,488	179,488	179,488
4566	Geogrids	$0.00	0	$0.00	0	0	478	478	478
4594	Profiling	Lump sum	0	$0.00	0	0	5,129	5,129	5,129
5103	One Coat Seal	M2	4400	$4.00	17,600	0	40,778	40,778	40,778
5500	Asphalt Pavements & Seals	Tonne	490	$124.00	60,760	0	87,352	87,352	87,352
6120	General Road Guidance Systems	Lump sum	1	$1,000.00	1,000	0	274	274	274
6300	Pavement Marking	Lump sum	1	$1,000.00	1,000	0	4,078	4,078	4,078
8901	Quality Assurance	Lump sum	1	$5,000.00	5,000	19	4,168	4,186	4,186
8925	Consultant Fees Design	Lump sum	1	$26,000.00	26,000	19,240	0	19,240	19,240
9001	Alterations to existing Water Services	Item	0	$0.00	0	0	29	29	29
9004	Alterations to Existing Energy Supply Services	Item	1	$5,000.00	5,000	0	2,660	2,660	2,660
9005	Location Of Existing Services (Incl. Lia	Item	1	$500.00	500	239	0	239	239
9060	Tip Fees	Item	1	$3,000.00	3,000	0	16,570	16,570	16,570
9092	Concrete & Asphalt Cutting	Lump sum	500	$10.00	5,000	4,043	6,096	10,138	10,138
9100	Supervision of Construction (Foreman)	M2	1	$15,000.00	15,000	2,169	29,984	32,153	32,153
9101	Civil Design & Documentation/Site Survey	m	1	$5,000.00	5,000	0	5,141	5,141	5,141
9103	Management Reserve Funds	Item	1	$15,792.00	15,792	0	0	0	0
9104	Project Management Fees	Lump sum	1	$7,000.00	7,000	8,193	6,081	14,274	14,274
					$559,923.20	$191,545.05	$878,666.98	$1,070,212.03	$1,070,958.18

Bunya Rd - Pavement Rehabilitation

Exp Type	Description	Units	Actual Expenditure 11/12	Actual Expenditure 12/13	Actual Expenditure 13/14	Actual Expenditure Whole of	Total Project Cost WOL
0000	Non Specific Activity	Item	0	54,252	0	54,252	54,252
0001	Budget	Item	0	0	0	0	0
0808	January Rain 2013	Lump sum	0	29,961	0	29,961	29,961
1101	Site Facilities & Camp	Lump sum	0	17,169	0	17,169	17,169
1201	Provision for Traffic	Lump sum	0	66,196	0	66,196	66,196
1316	Control Of Erosion And Sedimentation	Lump sum	0	21,405	0	21,405	21,405
2263	Installation of Pipes 300 dia - 450 dia	m	0	2,377	0	2,377	2,377
3108	Excavation & Disposable of Unsuitable Material	Lump sum	0	93,483	0	93,483	93,483
3304	Geotextile	M2	0	1,533	0	1,533	1,533
3602	Entrances To Private Properties	Lump sum	0	28,604	0	28,604	28,604
4103	Base, Unbound Pavement	M3	0	260,762	545	261,307	261,307
4104	Subbase, Unbound Pavement	M3	0	27,409	96	27,505	27,505
5103	One Coat Seal	M2	0	110,716	0	110,716	110,716
5141	Two Coat Seal	M2	0	1,419	0	1,419	1,419
6161	Steel Beam Guardrail	m	0	12,194	0	12,194	12,194
6300	Pavement Marking	M	0	8,247	0	8,247	8,247
8885	Portable Long Service Leave Levy	Lump sum	0	0	0	0	0
8901	Quality Assurance	Lump sum	0	3,613	0	3,613	3,613
8925	Consultant Fees Design	Item	4,674	10,389	0	15,063	15,063
9060	Tip Fees	Lump sum	0	0	0	0	0
9092	Concrete & Asphalt Cutting	M	0	0	0	0	0
9100	Supervision of Construction (Foreman)	Lump sum	0	19,741	0	19,741	19,741
9101	Design Fees (Actual Internal Fee)	Lump sum	0	2,105	0	2,105	2,105
9103	Management Reserve Funds	Lump sum	0	0	0	0	0
9104	Project Management Fees	Lump sum	0	44,127	59	44,186	44,186
9105	Capital Overhead Allocation	Lump sum	0	0	11	11	11
			$4,674.00	$815,701.82	$709.84	$821,085.66	$821,085.66

Glenview Road - Pavement Rehabilitation

Exp Type	Description	Units	Actual Expenditure 11/12	Actual Expenditure 12/13	Actual Expenditure 13/14	Actual Expenditure Whole of	Total Project Cost WOL
0000	Non Specific Activity	Item	1,853	15,287	0	17,141	17,141
0001	Budget	Item	0	0	0	0	0
1101	Site Facilities & Camp	Lump sum	0	3,527	0	3,527	3,527
1201	Provision for Traffic	Lump sum	0	22,607	0	22,607	22,607
1316	Control Of Erosion And Sedimentation	Lump sum	0	0	0	0	0
2100	Removal or Demolition of Existing Compon	Lump sum	0	2,154	0	2,154	2,154
2201	Supply of Pipe Culvert Components	Item	0	699	0	699	699
2263	Installation of Pipes 300 dia - 450 dia	m	0	2,918	0	2,918	2,918
2317	Pre-Cast Concrete End Structures Culvert	Item	0	0	0	0	0
2502	Subsoil Drains Type D	M	0	0	0	0	0
2642	Grouted Rock Pitching	M2	0	0	0	0	0
3100	Earthworks, Excav/Embank/Subgrade Treatments	M3	0	54,193	0	54,193	54,193
3108	Excavation & Disposable of Unsuitable Material	Lump sum	0	13,577	0	13,577	13,577
3602	Entrances To Private Properties	Lump sum	0	6,237	0	6,237	6,237
3800	Landscaping Works	M2	0	2,340	0	2,340	2,340
4103	Base, Unbound Pavement	M3	0	41,293	0	41,293	41,293
4551	Scarify Existing Pavement	m2	0	7,830	0	7,830	7,830
5500	Asphalt Pavements & Seals	Tonne	0	29,297	0	29,297	29,297
6300	Pavement Marking	M	0	1,461	0	1,461	1,461
8885	Portable Long Service Leave Levy	Lump sum	0	272	0	272	272
8901	Quality Assurance	Lump sum	0	340	0	340	340
8915	Consultant Fees Geotechnical Investigati	Lump sum	0	0	0	0	0
8925	Consultant Fees Design	Item	0	2,000	0	2,000	2,000
9000	Alterations to Existing Stormwater Services	Item	0	0	0	0	0
9005	Location Of Existing Services (Incl. Lia	Lump sum	0	865	0	865	865
9060	Tip Fees	Lump sum	0	0	0	0	0
9092	Concrete & Asphalt Cutting	M	0	1,470	0	1,470	1,470
9100	Supervision of Construction (Foreman)	Item	0	4,784	0	4,784	4,784
9101	Design Fees (Actual Internal Fee)	Item	0	232	0	232	232
9103	Management Reserve Funds	Item	0	0	0	0	0
9104	Project Management Fees	Item	0	6,731	0	6,731	6,731
			$1,853.45	$220,113.89	$0.00	$221,967.34	**$221,967.34**

Buderim St - Pavement Rehabilitation

Exp Type	Description	Total Project Cost WOL	
0000	Non Specific Activity	$	35,054
0001	Budget	$	-
1101	Site Facilities & Camp	$	128
1201	Provision for Traffic	$	58,035
2100	Removal or Demolition of Existing Compon	$	3,186
2401	Concrete Kerb	$	9,575
2413	Concrete Gullies	$	3,682
2414	Concrete Access Chambers	$	2,100
2501	Sub-Soil Drains Type B	$	5,167
3108	Excavation & Disposable of Unsuitable Material	$	91,570
3500	Backfill	$	65,844
4594	Profiling	$	121,323
5103	One Coat Seal	$	8,072
5500	Asphalt Pavements & Seals	$	587,004
6300	Pavement Marking	$	9,363
8901	Quality Assurance	$	570
8925	Consultant Fees Design	$	3,609
9100	Supervision of Construction (Foreman)	$	8,200
9104	Project Management Fees	$	27,025
	Project Total:	**$**	**1,039,507**

Point Cartwright Drive, Buddina

Exp Type	Description	Total Project Cost WOL	
0000	Non Specific Activity	$	63
0001	Budget	$	478
1201	Provision for Traffic	$	188
2413	Concrete Gullies	$	908
2631	Handplaced Concrete Paving	$	-
4594	Profiling	$	73,570
5500	Asphalt Pavements & Seals	$	447,277
8925	Consultant Fees Design	$	8,598
9100	Supervision of Construction (Foreman)	$	478
9104	Project Management Fees	$	3,521
	Project Total:	**$**	**535,080**