University of Southern Queensland

Faculty of Health, Engineering and Sciences

THE IMPACTS CONSTRUCTION TRAFFIC HAS ON PAVEMENTS WITHIN RESIDENTIAL SUBDIVISIONS

A dissertation submitted by

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in fulfilment of the requirements of

ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Honours) (Civil)

Submitted October, 2016

Abstract

The aim of this research project is to ascertain that if during the first year of an access streets design life, the road pavement is subjected to a peak in the traffic loadings. This peak is a result of the heavy vehicles used in the construction of residential dwellings. From the reviewed literature it is evident that passenger vehicles have very little effect on the pavement and heavy vehicles are the main cause of structural pavement failures. This puts a burden on the community as the local government must divert funding to rehabilitate a pavement asset which has failed prematurely.

Throughout this research project falling weight deflectometer (FWD) testing has been utilised, this is an appropriate testing method that is widely adopted by Transport and Main Roads (TMR). The non-destructive testing determined the structural characteristics of a number of existing access streets within North Shore Estate. The roads were selected to achieve a varied cross section of different access street pavements for the research. Analysis of this FWD test data highlighted that a number of roads had failed to meet the minimum deflection limits set by TMR which suggest the pavement has been impacted by the vehicles used in residential dwelling construction.

An alternative method for calculating the design Equivalent Standard Axles (ESA) has been developed to ensure the access street pavements can withstand the initial peak in the number of heavy vehicles during the first year. When applied to the Austroads pavement design charts, an increase in gravel thickness of approximately 30mm was required when compared to tradition Design ESA calculation methods. Further research and field testing of the alternative access street pavement designs are required to ensure this alternative design method can be endorsed and enforced by local government authorities.

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Limitations of Use

Certification

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.

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Acknowledgements

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

A special thanks goes to Dr Soma Somasundaraswaran for assisting me throughout this research project. To Andrew Astorquia from Stockland Development, for providing funding to conduct the testing. To Mr Duane Gibson from UDP Group, for his engineering knowledge and contacts within the engineering fields. Lastly this research project would not have been achievable if it wasn't for the support of my wife, Amanda Pease.

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1.0 CHAPTER 1 - Introduction

1.1 Background

In urban residential subdivisions a road network consists of two different categories. The higher of the two are roads which enable the distribution of traffic, and the other being streets which enable interaction with the adjoining properties. A road hierarchy is essential to ensure the safety and appropriate conveyance of the public. Each type of road within the network serves a distinct set of functions and is designed accordingly.

Streets with the classification of an access street will form the main focus of this research project. Access streets are one of the lowest ranked streets within the hierarchy and therefore only service a limited amount of residential properties per street. They are also the most commonly found street within a residential subdivision.

Flexible pavements consisting of unbound granular materials have been widely adopted in the construction of streets within residential subdivisions. These pavements include a wearing surface such as asphalt or bituminous seal, base and or subbase layers. They are then placed over either imported subgrade material or over the natural subgrade. It is standard practice when designing a pavement, to assess the natural subgrade material, determine the Design Equivalent Standard Axles from traffic data and calculate the thickness of pavement layers using the adopted method approved by the local authority.

It is well known that passenger vehicles cause little to no effect towards the structural performance of a road pavement. The true damage is caused by heavy vehicles such as concrete or delivery trucks. Areas where heavy vehicles may be travelling at low speeds, accelerating or braking are likely to be the first pavement areas in which failures will occur

This Research project plans to analyse the theory that the current standards being adopted for the design of pavements for access streets, are being exceeded by the heavy vehicles used in the construction of residential dwellings. A literature review will be conducted to establish background information relating to the design of road pavements, with respect to the design equivalent standard axles and percentage of heavy vehicles.

1.2 Problem Statement

The objective in the design of the road pavement is to select appropriate pavement and surfacing materials, types, layer thicknesses and configurations to ensure that the pavement performs to its design functions and requires minimal maintenance under the anticipated traffic loading for the design life adopted (Townsville City Council 2014).

It is common engineering knowledge that passenger vehicles have very little effect on the pavement structure, and heavy vehicles are the reason a pavement will fail from traffic loading. The current standards assume that during the typical 20-year design life of an access street, it only experiences heavy vehicle traffic such as the weekly garbage truck and the odd removal truck. However, if you were to consider the first year during residential dwelling construction, the percentage of heavy vehicles will peak and cause a higher traffic load. This results in the Design ESA not being calculated correctly and therefore in reality the pavement will fail before the intended design life.

Having a pavement fail 5-10 years prior to the intended design life, puts a strain on the local government to come up with the necessary funds to pay for the reconstruction. These costs are generally passed onto the residents of the community.

Over the past 10 years the Townsville residential property market has had some highs and lows. During the highs, the extreme rate of dwelling construction was previously unseen. This resulted in pavement failure on access streets occurring within the first 12 months of the pavements service life. At the time, several different possible causes of the pavement failure were investigated such as; construction quality, materials, design procedures and subsurface drainage. To no avail the cause of this pavement failure was not found.

In order to resolve this issue, the theoretical pavement life will be analysed against data collected from non-destructive testing of in-situ pavements. If it is established that the pavements within access streets are being impacted by heavy vehicles used in the construction of residential dwellings, an alternative design technique will be investigated.

If the access street pavement can remain unaffected by traffic loads during the first few years of service life, then it is likely that it will go on for approximately 20-30 years without failure. A goal of this research project is to establish a new pavement design technique that accounts for construction traffic and enables the residential pavement to reach its intended design life. It is also expected that these new methods will have additional upfront costs, however this will ensure that the expected design life is achieved.

1.3 Objectives

This research project is aimed at crediting or discrediting the theory that construction traffic generated by residential dwellings during the first year causes a higher than acceptable reduction in pavement life. As design standards vary from each local government, two sites within the jurisdiction of the Townsville City Council will be selected.

The objectives of this research project are as follows:

1. Research background information relating to the design of road pavements with respect to the design equivalent standard axles and percentage of heavy vehicles.

2. Gather as-constructed information from numerous residential subdivision stages and analyse the data to establish appropriate and comparable stages for non-destructive testing.
3. Establish appropriate test locations within the chosen residential stages and to conduct enough non-destructive tests to ensure a good spread of data is achieved.
4. Convert the raw data extracted from the non-destructive testing into a format in which the amount of life remaining in the pavement can be calculated.
5. Analyse theoretical versus actual design equivalent standard axles, and establish whether the residential pavement has exceeded or failed to meet the theoretical design calculations.
6. Establish an alternative design technique that accounts for construction traffic and enables the residential pavement to reach its intended design life.

If time and resources permits:

7. Repeat non-destructive testing after construction traffic has impacted the new residential pavement and analyse new results with base line data.

The overall goal of this research project is to provide a new process in which these local access streets can be designed. It is recognised that further testing and analysis will be required. This is due to the limited amount of funding and time to gather results so that adequate analysis of the proposed method can be performed. This will finally result in the proposed method being fully endorsed and enforced by the local government.

2.0 CHAPTER 2 - Literature Review

A literature review has been completed to establish the impacts heavy vehicles have on road pavements, types of pavement failures and testing methods. To determine an effective modified pavement design method for residential streets, literature was collated and reviewed under the following categories:

- Road Networks
- Pavement design;
- Heavy Vehicles;
- Visible Pavement Failures;
- Moisture Changes during Service Life;
- Circly Pavement Design Software; and
- Pavement Testing Methods.

2.1 Road Networks

A hierarchical road network is essential to maximise road safety, residential amenity and legibility. Each class of road in the network serves a distinct set of functions and is designed accordingly. The design should convey to motorists the predominant function of the road (Townsville City Council 2014).

The road network is broken up into two distinct levels; streets and roads. The lowest order of transport route (streets) have as their primary function to facilitate public interaction and movement through a place, village. Town or city. The Highest order of transport route (Roads) should have as its main function the convenient and safe distribution of traffic (Townsville City Council 2014).

Streets with the classification of an access street will form the main focus of this research project. Access streets are the lowest ranked street within the category and therefore only service a limited amount of residential properties per street, they are also the most commonly found street within a residential subdivision.

2.2 Pavement Design

Each local government has their own methods and techniques for the designing of residential pavements. For the purposes of this research project the 'City Plan' for Townsville City Council' standards have been adopted. These council standards also reference 'Austroads,' the associations of Australasian road transport and traffic agency.

The design of a road pavement involves the selection of either ridged or flexible designs, pavement surfacing materials, gravel types and layer thickness to ensure the pavement achieves its intended design functions. Each pavement design should also require minimal maintenance under the appropriate traffic loadings for the design life.

The details regarding pavement design is quite extensive. Therefore, below is a brief explanation of the major steps:

- Subgrade Evaluation – the support provided by the subgrade is the most important part of a pavement design. As per the Townsville city plan, tests are to be taken every 60 metres along the road to determine the CBR, material type, swell and particle sizing.
- Pavement Materials – there are five main areas of material: unbounds granular materials, modified granular materials, cemented materials, asphalt and concrete. Designs need to assess the availability and cost of the materials to ensure a cost effective pavement is achieved.

- Design Traffic – determining the Design ESA, which is based on the street type, percentage of heavy vehicles, expected design life and growth rates.
- Design of Pavement – using the above data the pavement thickness can be calculated. There are two main methods which achieve this. The first is in 'Austroads Design chart for granular pavements with thin bituminous surfacing' and the second 'Circly' a mechanistic pavement design software.
- A typical pavement profile can be seen in Figure 1 below.

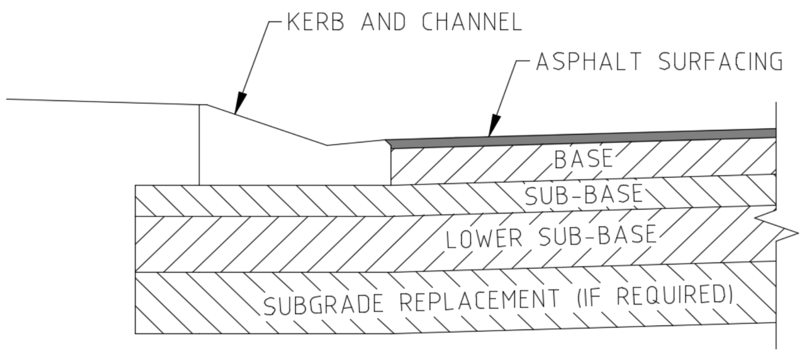

Figure 1 - Typical Pavement Profile

To ensure proper pavement performance during its lifetime, pavement structure must be designed to be able to withstand the predicted traffic and the loads it bears. Future traffic is predicted based on current yearly traffic data, social and economic factors of the area. The effect on the pavement from different loads generated by different types of vehicles is unified using the Equivalent Standard Axle Load (ESA) indicator (Janulevicius et al. 2013)

2.3 Heavy Vehicles

A road pavement must be wide enough and of suitable geometry to permit all vehicles to safely operate at an acceptable speed. In addition, it must be strong enough to cater for both the heaviest of these vehicles and the cumulative effects of the passage of all vehicles (TMR 2012).

Vehicular traffic consists of a mixture of vehicles ranging from cyclists to triple road trains. It has been well established that light vehicles (Austroads Vehicle Classes 1 and 2) contribute very little to structural deterioration, only heavy vehicles are considered in pavement design (TMR 2012). Areas where heavy vehicles may be travelling at low speeds, accelerating or braking are likely to be the first pavement areas in which failures will occur.

2.4 Visible Pavement Failures

A defect observed during a visual survey is evidence of an undesirable condition in a pavement. It may simply affect its serviceability and/or it may indicate a lack of structural capacity. The most common such indicators are potholes and patches, rutting, cracking and shoving (TMR, 2012).

Potholes provide a dramatic indication of pavement failure. They may be structural in nature, solely related to the surfacing or a combination of the two. Patches are usually repairs to a pavement and can indicate where issues exist or are likely to occur in the future. Their size can vary from small patches (e.g. a few square metres) to large/extensive patches (e.g. full lane width for several hundred metres) (TMR, 2012).

Rutting is a longitudinal deformation (depression) located in wheel paths and is most commonly found in flexible pavements. Generally, the layers suffering the deformation will be evident from associated indicators, or may be determined by inspection of test pits or trenches that reveal the pavement (cross) section through (across) the ruts (TMR, 2012). Rutting can also occur adjacent to the kerb and channel on urban roads. This is typically due to construction techniques that result in poor compaction of the gravel pavement. These deflections in the pavement allow water to pool and eventually penetrate the pavement which ultimately will cause the pavement to fail.

Cracking in the pavement surface can indicate the many number of issues such as; oxidation of the binder, permanent severe deformation of the subgrade caused by repetitive loading, instability in the upper pavement layers, cracking of underlying cementitiously bond layers, settlement and repeated deflection causing fatigue in the asphalt layers. Some commonly encountered cracks are; transverse cracks, fatigue cracks, age related cracks, longitudinal cracks and block cracks (TMR, 2012).

2.5 Moisture Changes during Service Life

The placing of a sealed pavement surfacing isolates the subgrade from some of the principal influences which affect moisture changes, especially infiltration of large quantities of surface water and evaporation. Where these influences are the controlling ones (i.e. dryer environments), the moisture conditions in subgrades generally tend to remain relatively uniform after an initial adjustment period. In such situations, the subgrade under the central region of the pavement is said to reach an equilibrium moisture condition. This region is flanked by two outer regions having moisture conditions that vary with time due to seasonal climatic influences, termed edge effects. Edge effects generally occur under the outer 1 to 2 metres of the sealed road surface. The magnitude of these fluctuations generally increases with distance from the centre of the road towards the edge of the sealed surfacing (TMR, 2016).

Townsville is located in far north Queensland and has a tropical climate. The average annual rainfall is 1143mm on an average 91 rain days, most of which falls in the six month "wet season" November to April (BOM 2016). Due to the varying nature of Townsville’s rainfall events, soils classified as having a ‘High’ expansive nature or higher, generally cause impacts on road pavements.

Table 1 - Guide to classification of expansive soils (TMR, 2016).

Expansive nature	Liquid limit (%)	Plasticity Index	PI x % < 0.425 mm	Swell (%)*
Very high	> 70	> 45	> 3200	> 5.0
High	> 70	> 45	2200 – 3200	2.5 – 5.0
Moderate	50 – 70	25 - 45	1200 – 2200	0.5 - 2.5
Low	< 50	< 25	< 1200	< 0.5

* Swell at OMC and 98% MDD using Standard compactive effort; four-day soak. Based on 4.5 kg surcharge.

In high rainfall areas, subgrade infiltration – particularly lateral infiltration through unsealed shoulders, through defects in wearing surfaces or through joints – has a major influence on the subgrade moisture conditions. Specific action should therefore be taken to guard against this influence (TMR, 2016).

2.6 CIRCLY Pavement Design Software

Remaining service life is widely used as a powerful tool to help asset managers to plan their maintenance and rehabilitation budgets and strategies. It is not only used for pavement management but also has been used for bridges, traffic signs, culverts and other infrastructure. It is defined as, the time period in years, or it can be expressed in terms of the remaining cumulative number of standard axle loads from the time of the analysis to the time the pavement is considered unserviceable or as providing substandard service (Saleh, 2014).

(Saleh, 2014) States that, Circly software was used to generate synthetic data for 140 pavement sections with different pavement structures. The analysed pavement structures cover both bound pavements with structural asphalts and unbound pavements with spray seal. The deflection bowl parameters were correlated with pavement properties and pavement structural response to estimate the remaining service life.

Circly is well known as an industry standard and is used around the world for thousands of design applications. It has the ability to perform mechanistic pavement design and analysis on road pavements.

2.7 Pavement Testing Methods

The testing of in situ road pavements is performed to gauge the extent of possible damage to the pavement and to assess remaining pavement life, such that a rehabilitation treatment can be determined. Depending on the situation, destructive testing may be an extreme measure and a less invasive testing method is more appropriate. However, in order to establish an accurate outcome, it is necessary to preform both non-destructive and destructive testing methods.

2.7.1 California Bearing Ratio

California Bearing Ratio (CBR) is defined as the ratio of the force required to cause a circular plunger of 1932 mm^2 area to penetrate the material for a specified distance expressed as a percentage of a standard force. The standard forces used in this method are 13,200 and 19,800 newtons for penetrations of 2.5 and 5.0 mm respectively (TMR, 2016).

Test specimens are prepared from passing 19.0 mm material, cured at a range of moisture contents and compacted using a compactive effort of 596 kJ/m3. They are then tested in either a soaked or unsoaked condition. This method allows for the determination of CBR Maximum Dry Density (CBR MDD) and CBR Optimum Moisture Content (CBR OMC) as well as the optional determination of swell and post penetration moisture content (TMR, 2016).

2.7.2 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) can be used to establish inexpensive and quick in situ CBR values of subgrade materials. The DCP operates by allowing the drop hammer with a mass of 9 kg and a free vertical fall of 510mm on a 16mm diameter shaft fitted with a stop and anvil. The penetration depth and number of blows allow for an in situ CBR value to be calculated.

The conversion of test data to CBR values is based on A.J. Scala: Simple Methods of Flexible Pavements Design Using Cone Penetrometers; Proceedings Second Aust-New Zealand Conference Soil Mechanics and Foundation Engineering, Christchurch, N.Z., January 1956 (TMR, 2016).

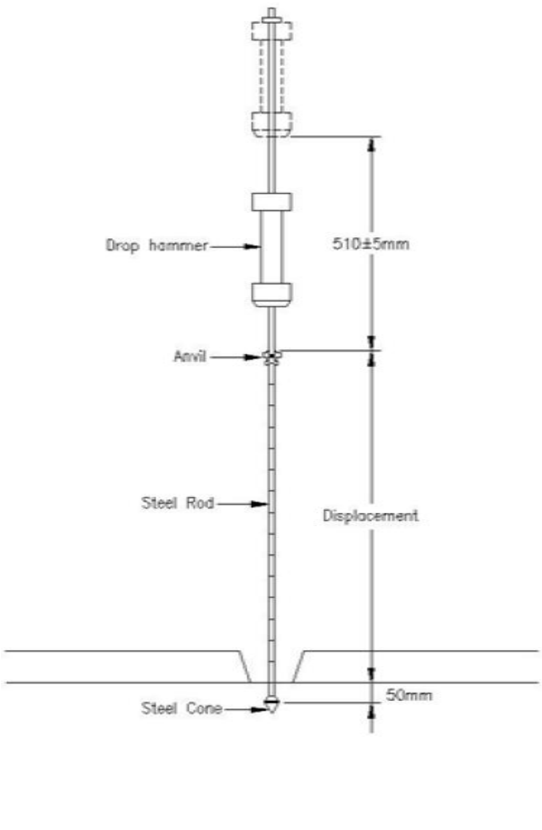

Figure 2 - Dynamic Cone Penetrometer (TMR, 2016).

2.7.3 Atterberg Limits

Atterberg limits have the ability to influence the long term service life of the pavement. There are three main Atterberg limits that are assessed for pavement designs, they are Liquid limit, Plastic Limit and Shrinkage Limit.

As defined by the (TMR, 2016) the three Atterberg limits can be described as:

- The liquid limit is defined notionally as the moisture content at which the soil passes from the plastic to the liquid state,
- The plastic limit is notionally defined as the moisture content at which the soil passes from the semisolid to the plastic state; and
- The linear shrinkage limit is defined as the percentage decrease in the longitudinal dimension of a soil bar when it is dried out from the liquid limit to the oven dry state.

Current pavement design practice takes into consideration the Atterberg limits to classify the expansive nature of the subgrade material. There are four categories; Low, Moderate, High and Very High, based on the classification a particular pavement treatment will be specified.

2.7.4 Destructive Testing

Destructive testing is best utilised when performed in conjunction with non-destructive testing. This aids with ensuring that the destructive tests are at critical locations in the pavement. Destructive testing typically involves pavement coring, excavating pits or pavement trenching.

Trenching and coring have been used in forensic and routine evaluation to determine the source of the problematic layer or layers. For example, there was severe rutting on US 281 and the district expressed a need to determine the source of the rutting. Although falling weight deflectometer and ground penetration radar tests were performed, evaluation of the data could not differentiate from which layer(s) the rutting came. Trenching provides a viable option. For example, Figure 3 illustrates the pavement section profiles on US 281. Chalk and stringlines were used to differentiate different pavement layers, as shown in Figure 3, the rutting was found to be from the surface AC layer (Texas Department of Transportation 2011).

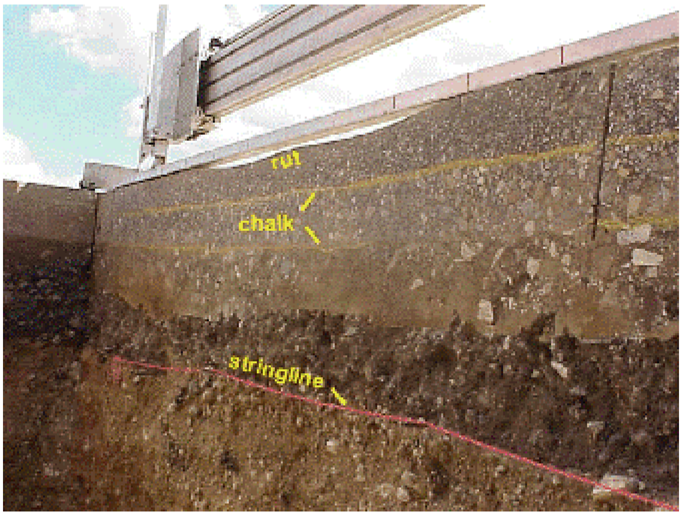

Figure 3 - Trench sidewalls showing the pavement layer profile (Texas Department of Transportation 2011).

Pavement cores can be used for compression testing and excavation pits/trenches can allow for a visual analysis of the pavement layers. From the samples obtained from coring or test pits it is possible to perform laboratory soil testing. This typically involves Atterberg limits, moisture content, soil classification and triaxial compression testing of the core samples.

Destructive testing is more expensive and invasive than non-destructive testing, however will return the true condition of the pavement.

2.7.5 Non-Destructive Testing

The capability of Ground Penetrating Radar and Falling weight deflectometer renders them very popular in pavement rehabilitation design. However, it should be noted that pavement assessment via non-destructive methods must be pursued with caution Mooney et al. (2000).

The use of a falling weight deflectometer has been adopted for most research proposals investigating pavement failure. Its ability to calculate the pavement and subgrade E modulus without destroying the pavement is a valuable tool. A falling weight deflectometer can be used to measure the vertical deflection response of a pavement surface when a load is applied. Figure 4 below, details the typical falling weight deflectometer rig. The sensors are located along the geophone beam and are used to record the pavement surface characteristics when the load plate is dropped onto the pavement. A FWD is capable of being used for the following applications:

- Pavement rehabilitation and overlay,
- Assessing the remaining life in a pavement,
- Void detection, and
- Experimental pavement materials.

Mooney et al. (2000) has stated that the reliance on non-destructive testing alone for pavement analysis and rehabilitation design would result in significant error.

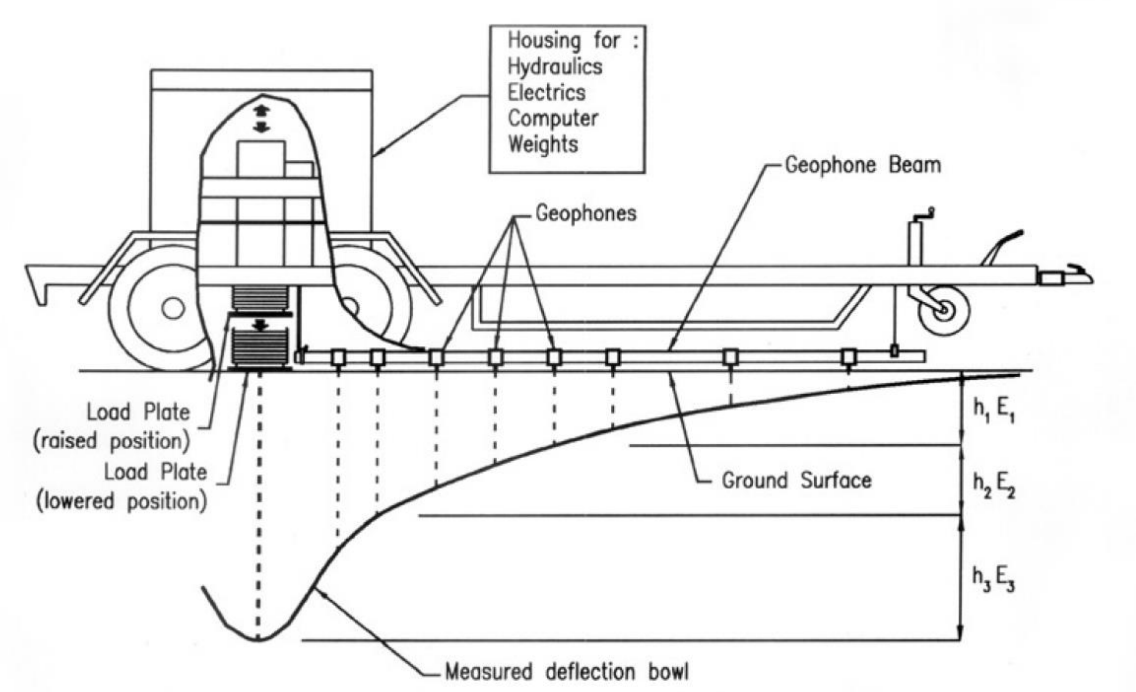

Figure 4 - Northern Pavement Consultants falling weight deflectometer diagram

Ground Penetrating Radar (GPR) is a non-destructive test that has been utilised in many other sectors over the years and is gaining popularity in pavement engineering. By using this technology, the change of material throughout the pavement depth can be detected, essentially providing the thickness of each pavement material. In addition, it can also sense voids or areas of concern like cracking in rigid pavements. The current GPR technology also allows data to be recorded at traffic speeds, the radar data is surveyed in continuous measurements and processed at intervals of 0.5m. This provides us a continuous thickness profile reading of tested sections without the ground disruption and traffic delays (Wong & Urbaes 2012).

2.8 Conclusion

Numerous results from multiple articles and manuals have highlighted that heavy vehicles cause significantly more damage than that of passenger vehicles. It also appears apparent that all major research is focused on larger road networks and not about the design of access streets. It is possible that design techniques have been formulated for high capacity roads and then scaled down for smaller access streets, which may not be the most appropriate.

A review of the available testing methods was also conducted to establish current practices and the most appropriate testing to meet the needs of this research project. Being able to test and prove that current design methods are not capable of catering for the construction loads generated from residential traffic, will enable new design techniques to be developed. Based on the above information included in Chapter 2 there is a gap in research in this area and the project is deemed feasible.

3.0 CHAPTER 3 - Methodology

Due to the equipment and the large amount of resources required, it is next to impossible to conduct the testing in a controlled environment. Therefore, the use of non-destructive testing on in-situ pavements located in Townsville was the most appropriate method. The aim will be to gather as-constructed information from numerous residential subdivision stages and analyse the data, to establish appropriate and comparable stages for non-destructive testing.

Appropriate test locations within the chosen residential stages will be established. Sufficient non-destructive tests will be conducted to ensure a good spread of data is achieved. This raw data will be used to assess the amount of actual life remaining in the pavement. This data will then be used to analyse the theoretical versus actual design equivalent standard axles. From this analysis it will be established whether the residential pavement has exceeded or failed to meet the theoretical design calculations.

3.1 Site Selection Analysis

Due to the extensive existing knowledge of the North Shore residential subdivision and existing relationships with developers, it was decided that this was the best site to conduct this research project. Within the North Shore development there are numerous different types of roads, with categories ranging from sub arterial to access streets. As per previous statements, roads within the network classified as Minor Collector and higher are not reporting pavement failures. Therefore, in order to accurately determine the most appropriate access streets for fall weight deflectometer testing the following information was collated;

- As-Constructed Pavement Design for all constructed Stages

- North Shore Road Hierarchy
- Results from subgrade testing, CBR's, bore logs and
- Visual inspection of road pavement.

The first step was to compile all the as-constructed and design data into a format which enabled patterns to be identified. It was then necessary to identify any possible issues within the stage that ruled it out from being tested. Pavement issues varied from such items as, previous pavement rehabilitation, length of the road and the road classification. Stages which were highlighted as 'Possible stage for testing' could then be compared for the FWD Testing. The two major variables that determine a pavement design is the Subgrade CBR value and the Design ESA. Therefore, it was determined that these variables would be utilised to find comparable stages for testing. The table below is an extract from the full analysis which can be found in 'APPENDIX B – Site Selection Analysis'

Table 2 - Selected Stages for Testing

Stage Number	Number of Lots	Placed On Maintenance	Years in Service	Road Classification		Adopted Design CBR's	Access Street DESA	Cement Treated	Visual Inspection	Applicable for Testing	Comments
				Access Streets	Minor Collectors						
509	35	13/10/2010	5.61	2	1	4.0	5.86E+03	No	Yes	Yes	Possible stage for testing
513	22	14/03/2010	6.19	3	0	1.5 & 2.5	5.86E+03	No	Yes	Yes	Possible stage for testing
517	23	21/12/2011	4.42	2	1	1.0 & 1.5	5.86E+03	No	Yes	Yes	Possible stage for testing
523	28	12/05/2014	2.03	3	0	4.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
570	20	22/05/2016	0.00	3	1	5.0	1.20E+05	Yes	Yes	Yes	Possible stage for testing

Stages 509 and 523 have similar Design ESA and Existing CBR values, therefore will be analysed together. This is the same situation for Stages 513 and 517. Stages 523 and 570 include the addition of cement to the lower subbase pavement layer. It should be noted that the cement is not required as part of standard practice and was added as an additional measure. The most recently constructed Stage 570, which is yet to be impacted by traffic, will be used to establish a base line strength of the modified design. Figure 5 below displays the layout of North Shore and the corresponding Stage numbers from the analysis.

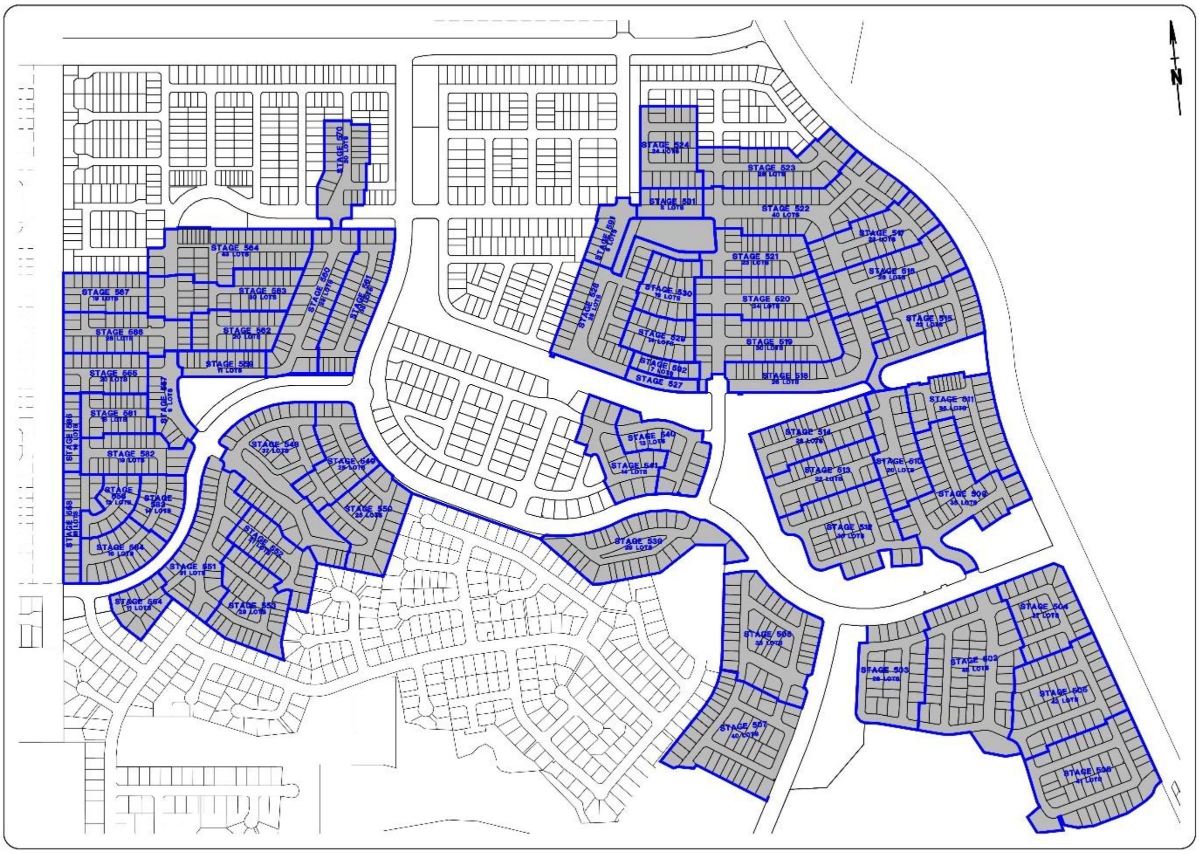

Figure 5 - North Shore Stages

- Stage 509 – Saba Street (Standard Design, Approximately 6.0 Years of Service)
- Stage 513 – Oculina Street (Standard Design, Approximately 6.5 Years of Service)
- Stage 517 – Laysan Street (Standard Design, Approximately 4.8 Years of Service)
- Stage 523 – Yanuca Street (Modified Design, Approximately 2.4 Years of Service)

- Stage 570 – Columbus Street (Modified Design, No traffic loading)

3.2 Locations of FWD Test Points

According to current construction methods the trench of a service crossing is back filled with crushed dust to the underside of the road pavement. It is expected that when the trench is compacted, a higher subgrade strength is achieved along the service crossing. Therefore, in order to achieve accurate data, it is critical that all test points are located such that they do not coincide with underground services. As-constructed data for the following underground services will be acquired from Ergon, NBN and council's data base:

- Stormwater Pipes
- Water Mains
- Sewer Mains
- Subsurface Pavement Drains
- Electrical Conduits
- Telecommunications Conduits

Each road that was selected as part of section 3.1 is between 120 – 170 meters in length. In order to achieve an acceptable number of test points per road and due to the limited funding, 10 FWD test points were adopted per road. This gave an average separation of 15.5 meters between test points.

Northern Pavement Consultants were commissioned to perform the non-destructive testing at the predetermined locations as detailed in sections 3.2.1 to 3.2.5. Northern Pavement Consultants Falling Weight Deflectometer equipment had previously been calibrated and was in good working condition. The FWD was the only testing equipment utilised and was operated by the staff at 'Northern Pavement Consultants.' A risk analysis was completed for the testing and due to the slow speeds and sufficient room to pass the testing rig, the risk was deemed as low. However, to ensure the safety of the public and to the staff, the following items were implemented in accordance with the Manual of Uniform Traffic Control Devices:

- Testing Vehicle and FWD trailer equipped with vehicle mounted warning device.
- Shadow Vehicle with mounted warning device.
- Single staff member located on the verge observing the tests.

Refer to sections 3.2.1 to 3.2.5 which details each of the 5 selected road pavements, existing services, test points and test chainage.

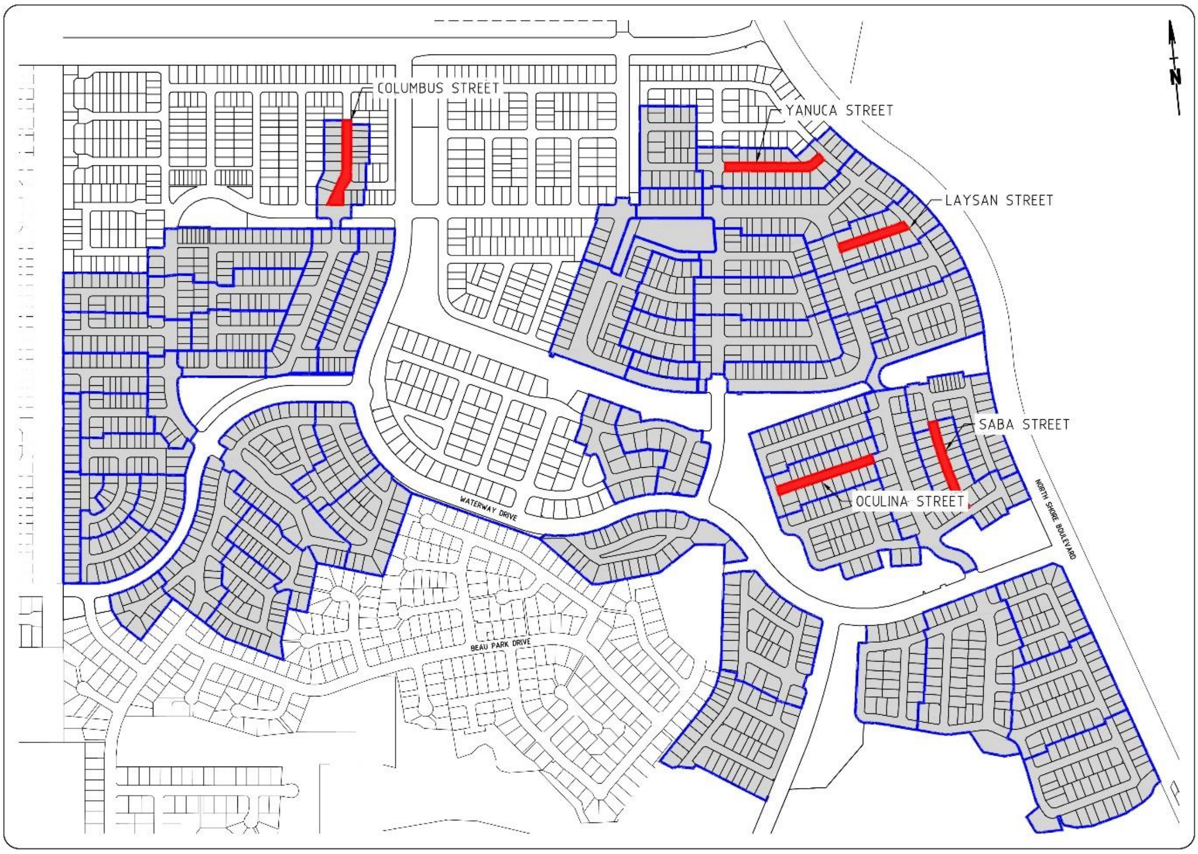

Figure 6 - Location of Test Roads

3.2.1 Test Site 1 – Saba Street

Test points 1 to 10 are located as per Figure 7 below.

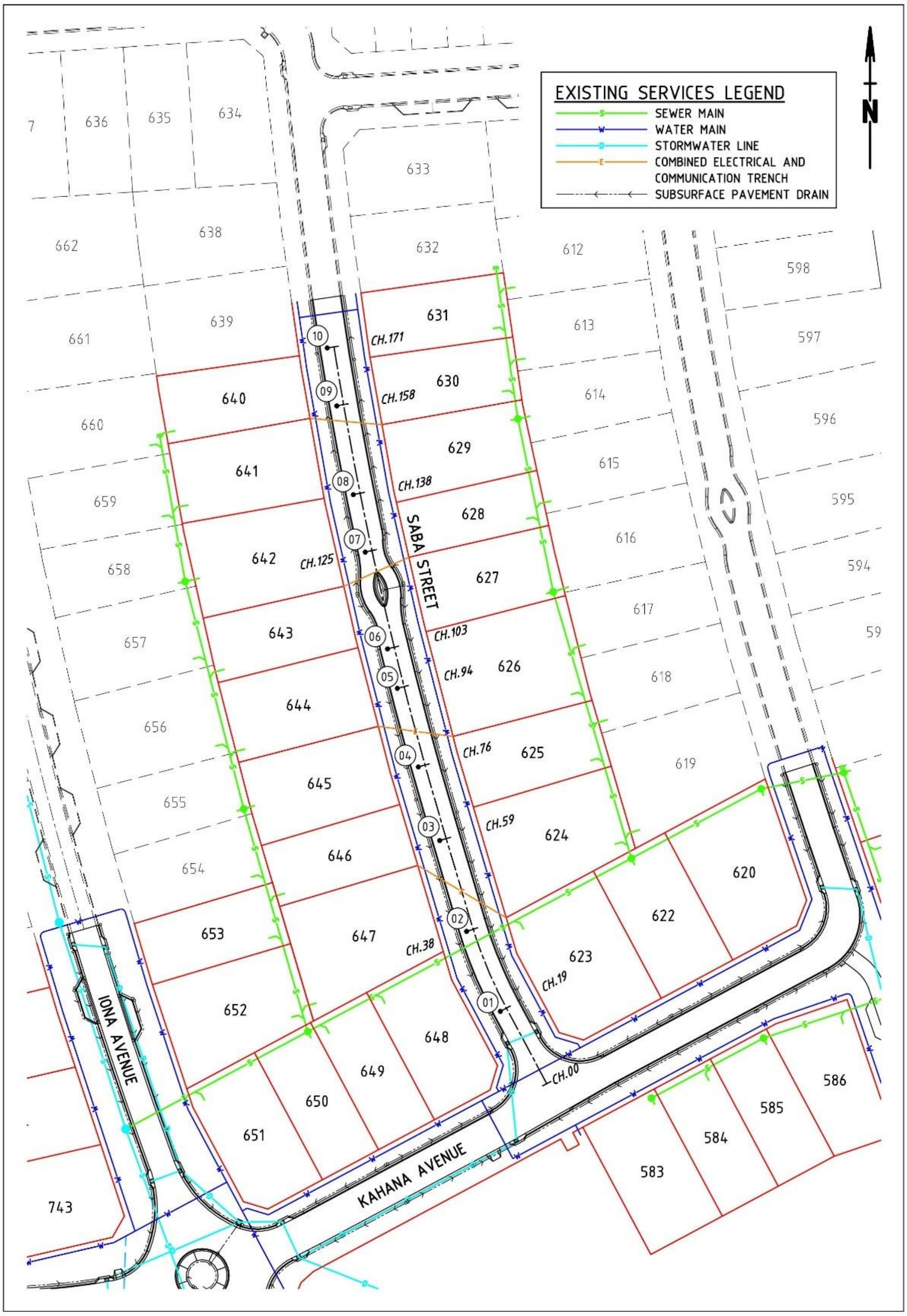

Figure 7 - Saba Street Test Locations

3.2.2 Test Site 2 – Oculina Street

Test points 11 to 20 are located as per Figure 8 below.

Figure 8 - Oculina Street Test Locations

3.2.3 Test Site 3 – Laysan Street

Test points 21 to 30 are located as per Figure 9 below.

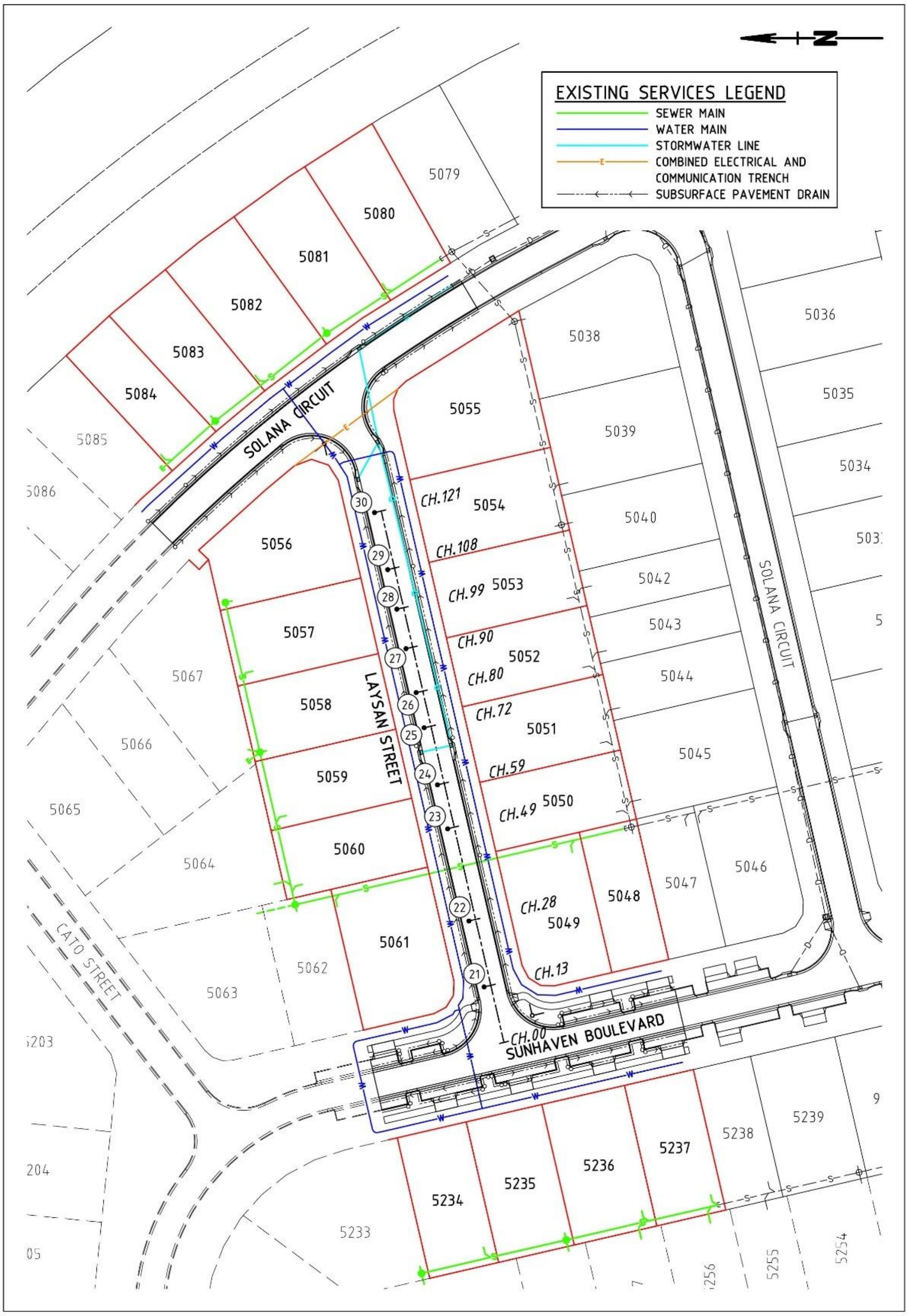

Figure 9 - Laysan Street Test Locations

3.2.4 Test Site 4 – Yanuca Street

Test points 31 to 40 are located as per Figure 10 below.

Figure 10 - Yanuca Street Test Locations

3.2.5 Test Site 5 – Columbus Street

Test points 41 to 51 are located as per Figure 11 below.

Figure 11 - Columbus Street Test Locations

3.3 Analysis Methods of FWD Test Data

Utilising the falling weight deflectometer test results for the five selected roads, the following four analysis methods have been adopted. These methods are in accordance with the Transport and Main Roads, Pavement Rehabilitation Manual (2012) and will give an indication of the structural condition of the pavement:

- Maximum Deflection
- Deflection Ratio
- Curvature Function
- Subgrade Response

3.3.1 Maximum Deflection

The maximum deflection is the maximum reading recorded for each test site. The maximum deflection is measured at the location where the FWD load strikes the pavement, this is represented as the D_0 value (i.e. 0 mm offset)

A representative deflection is determined for each test run in each section (e.g. for the inner and/or outer wheel path of each lane, Dr (IWP or OWP)) according to Equation 2.2 below. Dr can be used to verify pavement performance, to predict future performance and to design an overlay using the deflection reduction method. (TMR, 2012)

Equation 2.2 - Representative deflection $D_r = + 1.28 \times \sigma)$

Where:

- Dr = representative deflection

- σ = standard deviation of selected deflection results, typically D_0 values, for section under consideration.
- x = mean of selected deflection results, typically D_0 values, for section under consideration

In order to ascertain if the maximum deflection is acceptable, it is necessary to perform the following evaluation procedure:

- Determine the ESA value, as the volume of traffic on an access street is outside the extents of the chart, 1E5 will be adopted which is the lowest ESA on the chart;
- Determining the D_{900} value;
- Read the 'Tolerable Deflection' value from Figure 12 shown below;
- Compare the tolerable deflection with the maximum deflection from the FWD testings.

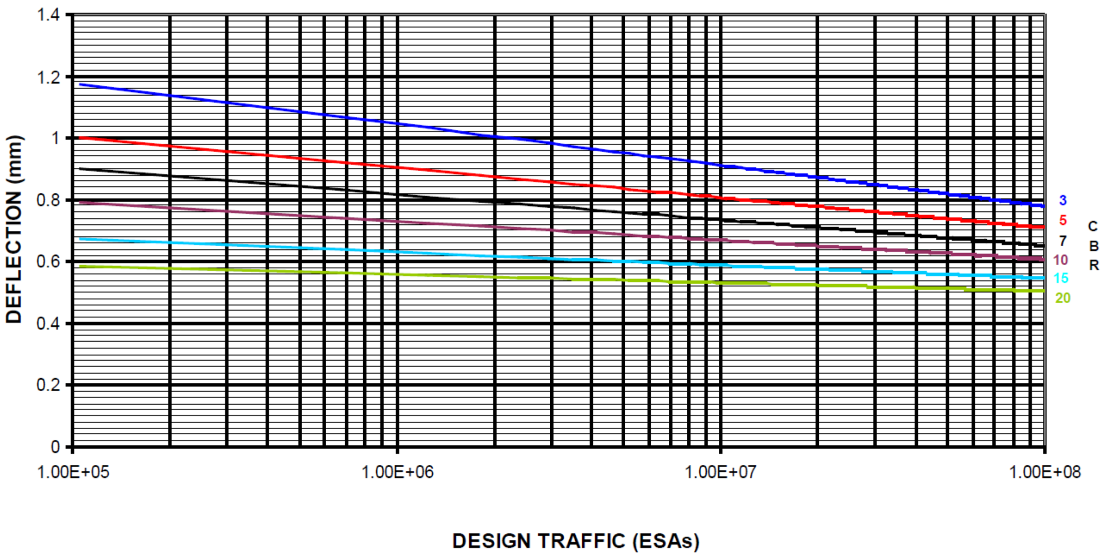

Figure 12 - Tolerable deflection for the normal design standard for the Benkelman Beam and FWD with 40 kN loading (TMR, 2012)

3.3.2 Deflection Ratio

The Deflection Ratio is used to delineate sections of road pavement that are bound, unbound or excessively weak, but not rigid such as concrete pavements. The Deflection Ratio (DR) is the ratio of the deflection at a point 250 mm from the maximum rebound deflection (D250) to the maximum rebound deflection (D0) (see Equation 2.3). The representative DR is the 10th percentile lowest DR assuming a 'normal' statistical distribution (see Equation 2.4). (TMR, 2012)

Equation 2.3 – Deflection ratio. DR = $DR = \frac{D_{250}}{D_0}$

Where:

- DR = deflection ratio
- D_{250} = deflection at a point 250 mm from the maximum rebound deflection
- D_0 = maximum rebound deflection

Equation 2.4 – Representative deflection ratio. $DR_r = x - (1.28 \times \sigma)$

Where:

- DRr = representative deflection ratio
- σ = standard deviation of deflection ratios for section under consideration
- x = mean of deflection ratios for section under consideration

For deflection ratio results derived using a FWD with a 40 kN loading the following analysis can be adopted;

- a deflection ratio of greater than 0.8 would indicate a bound pavement;

- A deflection ratio of between 0.6 and 0.7 would be expected for a good quality unbound pavement with a thin asphalt surfacing or seal; and
- A deflection ratio of less than 0.6 would indicate a possible weakness in an unbound pavement with a thin asphalt surfacing or seal (TMR, 2012).

3.3.3 Curvature Function

The shape (curvature) of the deflection bowl is used to estimate the likelihood of fatigue cracking in an asphalt layer. The curvature is defined by the Curvature Function (CF) as given in Equation 2.5. (TMR, 2012)

Equation 2.5 – Curvature function. $CF = D_0 - D_{200}$

Where:

- CF = curvature function
- D200 = deflection at a point 200 mm from the maximum rebound deflection
- D0 = maximum rebound deflection

The representative curvature function, CFr, for a section of pavement taken to be the mean CF. For granular pavements with thin bituminous surfacings, the curvature function is likely to be 25% to 35% of the maximum deflection. Values higher than this may indicate that the granular base course has low strength. (TMR, 2012)

High values of the CF (e.g. 0.4 mm for results derived using a FWD with a 40 kN loading, the Benkelman Beam or PAVDEF) may indicate a pavement that is lacking stiffness, a very thin pavement, or a pavement with a cracked asphalt surface. Low values of the CF (e.g. <0.2 mm for results derived using a FWD with a 40 kN loading, the Benkelman Beam or PAVDEF) indicate a stiff pavement. (TMR, 2012)

3.3.4 Subgrade Response

The deflection at a point 900 mm from the point of maximum deflection is referred to as the D_{900} value. For pavements without bound, thick asphalt or rigid layers, the D_{900} value has been found to reflect a subgrade response that remains essentially unaffected by the structure of the overlying pavement. This has been used to estimate the subgrade California Bearing Ratio (CBR) at the time of testing. This relationship is shown below in Figure 13, which shows results derived using a FWD with a 40 kN loading, the Benkelman Beam or PAVDEF. (TMR, 2012)

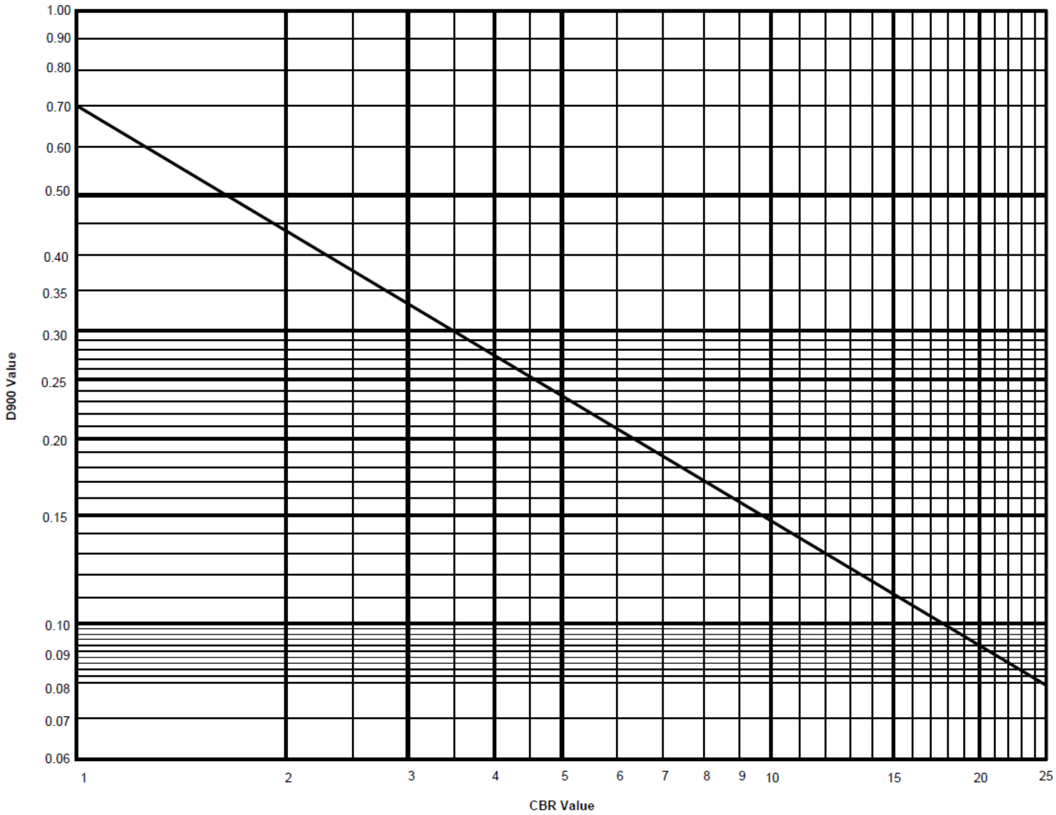

Figure 13 - D_{900} verse CBR for Benkelman Beam, PAVDEF and normalised 40 kN FWD results for granular pavement with a thin asphalt surfacing or seal (TMR, 2012)

The representative D900 value is taken as the 90% highest D_{900} (see Equation 2.6) (TMR, 2012).

Equation 2.6 – Representative D_{900} $$D_{900.r} = x + (1.28 \times \sigma)$$

Where:

- $D_{900.r}$ = representative D_{900}
- σ = standard deviation of D_{900} values for section under consideration
- x = mean of D_{900} values for section under consideration

3.4 Procedure to Determine Design ESA

Based on Townsville City Council Standards a theoretical Design ESA will be calculated with the following equation:

$$N_{DT} = 365 \times AADT \times DF \times \%HV/100 \times LDF \times CGF \text{ x } N_{HVAG,}$$

$$DESA = ESA/HVAG \times N_{DT,}$$

where

- AADT = Annual Average Daily Traffic in vehicles per day in the first year.
- DF = Direction Factor is the proportion of the two-way AADT travelling in the direction of the design lane.
- %HV = average percentage of heavy vehicles.
- LDF = Lane Distribution Factor, proportion of heavy vehicles in design lane.
- CGF = Cumulative Growth Factor.
- N_{HVAG} = average number of axle groups per heavy vehicle. (TMR, 2012)
- ESA/HVAG = average number of Equivalent Standard Axles per Heavy Vehicle Axle Group.
- N_{DT} = cumulative number of Heavy Vehicle Axle Groups over the design period

The variables for the Design ESA equation will be adopted from .

Table 3 below. This table has been adopted from Austroads ‘Guide to Pavement Technology Part 2 – Pavement Structural Design AGPT02.’ The five roads tested within this report are equivalent to ‘Local access with no busses’ in Table 3.

Table 3 - Design ESA Variables, Austroads 2012

Street type	AADT two-way	Heavy vehicles (%)	Design AADHV (single lane)	Design period (years)	Annual growth rate (%)	Cumulative growth factor (Table 7.4)	Axle groups per heavy vehicle	Cumulative HVAG over design period	ESA/HVAG	Indicative design traffic (ESA)
Minor with single lane traffic	30	3	0.9	20	0	20	2.0	13 140	0.2	3×10^3
				40	0	40	2.0	26 280	0.2	5×10^3
Minor with two lane traffic	90	3	1.35	20	0	20	2.0	19 710	0.2	4×10^3
				40	0	40	2.0	39 420	0.2	8×10^3
Local access with no buses	400	4	8	20	1	22.0	2.1	128 480	0.3	4×10^4
				40	1	48.9	2.1	285 576	0.3	9×10^4
Local access with buses	500	6	15	20	1	22.0	2.1	240 900	0.3	8×10^4
				40	1	48.9	2.1	535 455	0.3	1.5×10^5
Local access in industrial area	400	8	16	20	1	22.0	2.3	256 960	0.4	1.5×10^5
				40	1	48.9	2.3	571 152	0.4	3×10^5
Collector with no buses	1200	6	36	20	1.5	23.1	2.2	607 068	0.6	4×10^5
				40	1.5	54.3	2.2	1 427 004	0.6	10^6
Collector with buses	2000	7	70	20	1.5	23.1	2.2	1 180 410	0.6	8×10^5
				40	1.5	54.3	2.2	2 774 730	0.6	2×10^6

4.0 CHAPTER 4 – FWD Analysis & Discussion

4.1 Saba Street Analysis

Saba Street was selected due to the access street being developed within the peak residential market demands. Saba street has been in service for approximately five and a half years and is suggested to have been impacted by the construction traffic used in residential construction. Visual inspection of this street highlighted that there was minor subsidence at the lip of kerb which has already been repaired as seen below in Figure 14. There were also small amounts of aggregate within the asphalt which had been dislodged from the screwing effect of the heavy vehicles

Figure 14 - Saba Street Pavement Failure

Saba Street As-Constructed pavement design is detailed as per below:

- Design ESA – 5.86 x 10^3
- 25mm Asphalt Surfacing (DG10)
- 100mm Base Course (DMR Type 2.1)
- 195mm Sub-Base Course (DMR Type 2.3)
- Subgrade CBR = 4%

Normalised deflection results were received from Northern Pavement Consultants for each of the 10 tests performed on Saba Street. These deflection results are displayed in Table 4 below.

Table 4 - Saba Street FWD Test Results

Normalised to 40 kN Deflection Results																	
Road	Saba Street							**Start Date of Testing**			8/25/2016		**Target Load (kN)**			40	
Comments	Refer to Location Testing Diagram for further information							**Start Reference**			FWD 0.0 m = Kahana Avenue Kerb line						
FWD Chainage (m)	**Deflections (mm)**																
	Geophone Radius (mm)																
	0	200	300	400	500	600	700	800	900	1000	1200	1400	1600	1800	2000	2250	2500
19	1.374	0.936	0.639	0.442	0.294	0.202	0.141	0.108	0.084	0.074	0.059	0.055	0.048	0.045	0.038	0.033	0.026
38	1.231	0.850	0.598	0.432	0.302	0.221	0.164	0.128	0.103	0.087	0.067	0.057	0.049	0.046	0.038	0.034	0.027
59	1.671	1.147	0.800	0.555	0.363	0.245	0.169	0.127	0.102	0.088	0.074	0.066	0.055	0.051	0.042	0.035	0.030
76	1.738	1.184	0.811	0.550	0.348	0.226	0.147	0.110	0.087	0.080	0.068	0.061	0.049	0.044	0.036	0.031	0.027
94	1.258	0.803	0.520	0.330	0.191	0.107	0.052	0.026	0.015	0.019	0.020	0.025	0.022	0.022	0.017	0.015	0.014
103	1.625	1.086	0.750	0.514	0.322	0.204	0.120	0.073	0.046	0.039	0.036	0.037	0.032	0.030	0.023	0.020	0.020
125	1.396	0.966	0.682	0.478	0.320	0.220	0.153	0.116	0.090	0.076	0.061	0.056	0.045	0.042	0.034	0.029	0.024
138	1.192	0.785	0.520	0.338	0.197	0.112	0.055	0.029	0.017	0.017	0.020	0.022	0.019	0.019	0.016	0.014	0.014
158	1.823	1.233	0.855	0.588	0.372	0.240	0.149	0.104	0.078	0.070	0.063	0.055	0.045	0.041	0.033	0.028	0.025
171	1.988	1.339	0.904	0.605	0.367	0.227	0.133	0.088	0.064	0.055	0.048	0.044	0.036	0.034	0.027	0.024	0.022

4.1.1 Maximum Deflection

Figure 15 demonstrates that all 10 test points along Saba street exceed the tolerable deflection limit set by the TMR's Pavement Rehabilitation Manual. The representative Deflection for Saba Street is 1.883mm, with a mean and standard deviation of 1.530mm and 0.276mm respectively.

The maximum deflections of 1.823mm and 1.988mm at chainages 158 and 171 respectively. The full stretch of the road is an area of concern as the measured deflection averages 2.5 times the acceptable limit. The tolerable deflection as calculated from Figure 12 was determined as 0.680mm.

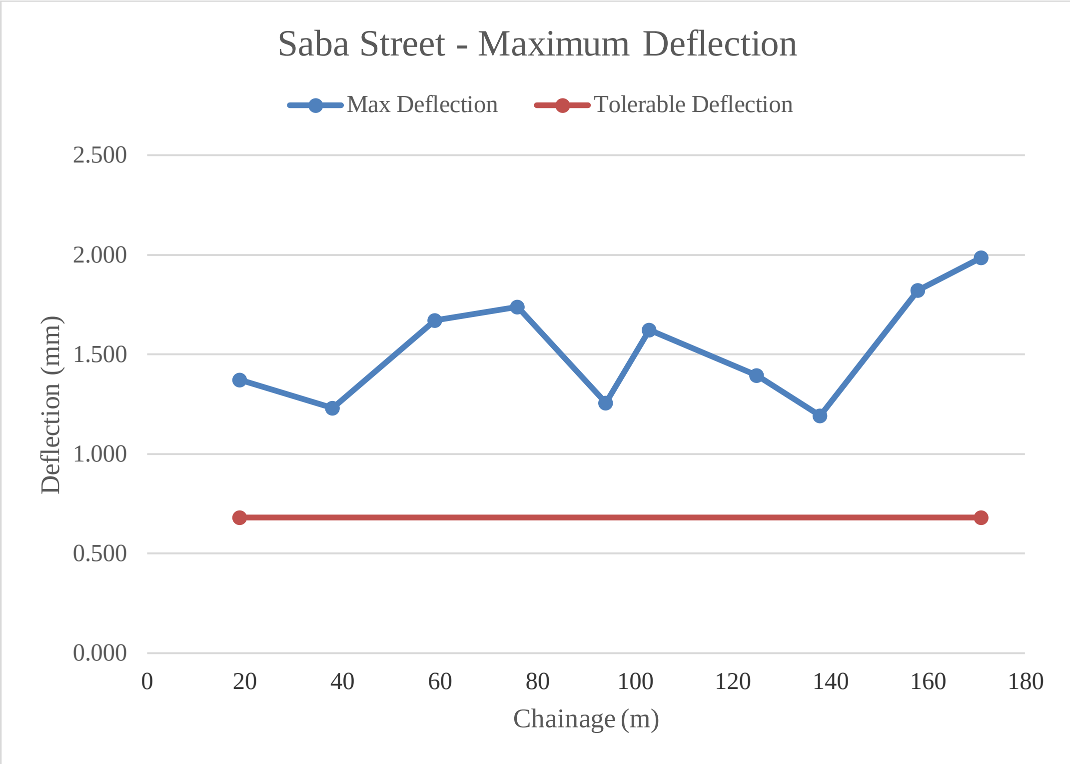

Figure 15 - Saba Street Maximum Deflections

4.1.2 Deflection Ratio

Saba Street has a representative deflection ratio of 0.543mm and with a mean and standard deviation of 0.568mm and 0.020mm respectively. As seen in Figure 16 below all the test locations fall within the zero to 0.600mm zone, which according to the Transport and Main Roads (2012) relates to a weak unbound pavement.

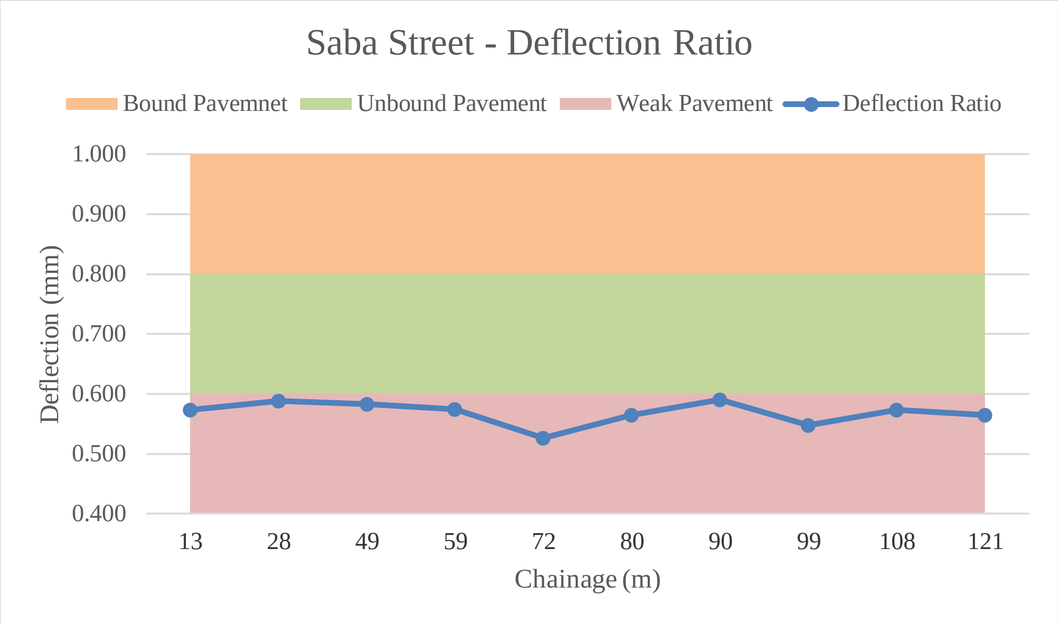

Figure 16 - Saba Street Deflection Ratio.

4.1.3 Curvature Function

Saba Street has an average relationship between the CF values and the maximum deflections of 32.5% for the length of the road. These percentages are consistent along the length of the road and are at the top of the expected limit set by Transport and Main Roads (2012) of 35%. Values higher than 35% may indicate that the granular base course has a low strength.

As seen below in Figure 17, the curvature function values range from 0.381mm to 0.649mm and have a mean of 0.497mm. Transport and Main Roads (2012) specify that roads with a curvature function greater than 0.4mm generally lack stiffness.

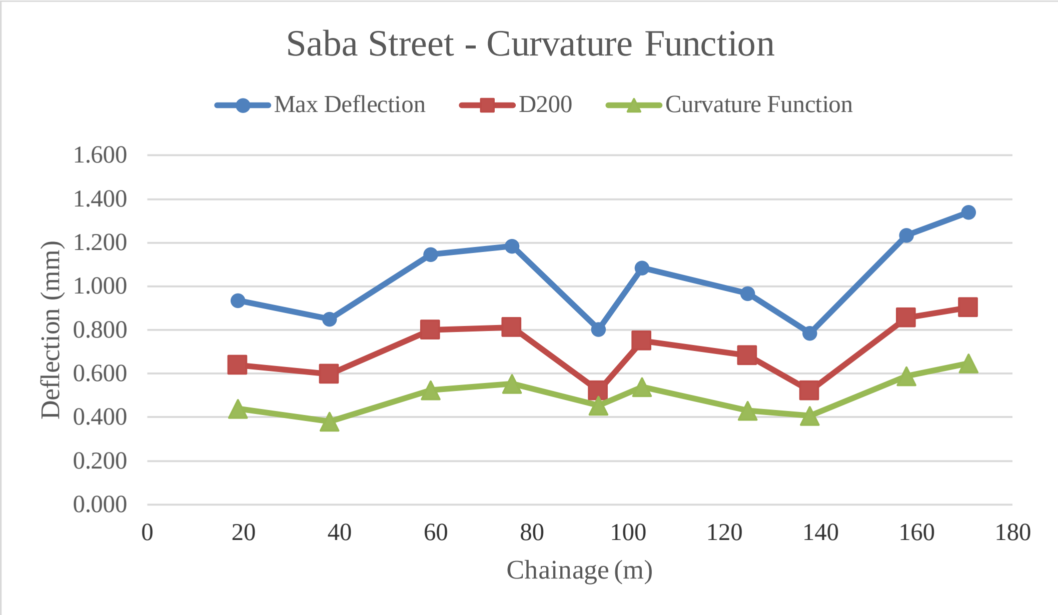

Figure 17 - Saba Street Curvature Function

4.1.4 Subgrade Response

Saba Street has a representative D_{900} value of 0.110mm and with a mean and standard deviation of 0.069mm and 0.032mm respectively. At the time of testing the CBR was estimated as 15.5% when the representative D_{900} value of 0.110mm was applied to Figure 13.

4.2 Oculina Street Analysis

Oculina Street was selected due to being developed within the peak residential market demands. This street has been in service for approximately six years and is suggested to have been impacted by the construction traffic used in residential construction. Visual inspection of this street highlighted that there was minor subsidence and the lip of kerb as seen below in Figure 18. There were also small amounts of aggregate within the asphalt, which had been dislodged from the screwing effect of the heavy vehicles

Figure 18 - Oculina Street Pavement Failure

Oculina Street As-Constructed pavement design is detailed as per below:

- Design ESA – 5.86 x 10^3
- 25mm Asphalt Surfacing (DG10)
- 100mm Base Course (DMR Type 2.1)
- 100mm Sub-Base Course (DMR Type 2.3)
- 340mm Sub-Base Course (DMR Type 2.5)
- Subgrade CBR = 1.5%

Normalised deflection results were received from Northern Pavement Consultants for each of the 10 tests performed on Oculina Street. These deflection results are displayed in Table 5 below.

Table 5 - Oculina Street FWD Test Results

Normalised to 40 kN Deflection Results																	
Road	Oculina Street							**Start Date of Testing**		8/25/2016		**Target Load (kN)**			40		
Comments	Refer to Location Testing Diagram for further information							**Start Reference**		FWD 0.0 m = Biscayne Street Kerb line							
FWD Chainage (m)	**Deflections (mm)**																
	Geophone Radius (mm)																
	0	200	300	400	500	600	700	800	900	1000	1200	1400	1600	1800	2000	2250	2500
19	0.509	0.312	0.196	0.129	0.086	0.063	0.049	0.042	0.034	0.032	0.024	0.022	0.019	0.018	0.015	0.012	0.011
30	0.630	0.403	0.263	0.183	0.129	0.100	0.078	0.067	0.056	0.050	0.038	0.033	0.027	0.025	0.020	0.016	0.013
49	0.692	0.435	0.277	0.182	0.116	0.081	0.058	0.046	0.036	0.032	0.026	0.023	0.021	0.020	0.018	0.015	0.012
66	0.728	0.477	0.318	0.219	0.148	0.108	0.082	0.067	0.054	0.047	0.035	0.030	0.026	0.026	0.021	0.017	0.014
80	0.673	0.428	0.281	0.196	0.137	0.105	0.081	0.067	0.054	0.047	0.034	0.029	0.024	0.023	0.018	0.015	0.013
94	0.901	0.583	0.376	0.257	0.178	0.131	0.103	0.087	0.072	0.064	0.051	0.043	0.036	0.033	0.028	0.024	0.018
110	1.066	0.737	0.510	0.363	0.252	0.185	0.139	0.113	0.092	0.082	0.060	0.052	0.044	0.040	0.032	0.027	0.023
128	0.943	0.632	0.432	0.301	0.209	0.153	0.119	0.100	0.080	0.070	0.053	0.046	0.036	0.034	0.028	0.024	0.019
153	0.788	0.533	0.371	0.264	0.186	0.138	0.104	0.083	0.065	0.055	0.037	0.031	0.025	0.025	0.019	0.017	0.014
169	0.843	0.554	0.370	0.263	0.190	0.151	0.126	0.111	0.097	0.087	0.068	0.059	0.050	0.047	0.039	0.033	0.026

4.2.1 Maximum Deflection

Figure 19 demonstrates that all but one test point along Oculina street exceeds the tolerable deflection limit set by the TMR's Pavement Rehabilitation Manual. The representative deflection for Oculina Street is 0.988mm, with a mean and standard deviation of 0.777mm and 0.165mm respectively.

The maximum deflections of 0.901mm, 1.066mm and 0.943mm occur at chainages 94, 110 and 128 respectively. From chainage 94 to the end of the road, it indicates an area of concern, as the measured deflection averages 1.8 times higher than the acceptable limit. The tolerable deflection as calculated from Figure 12 was determined as 0.580mm.

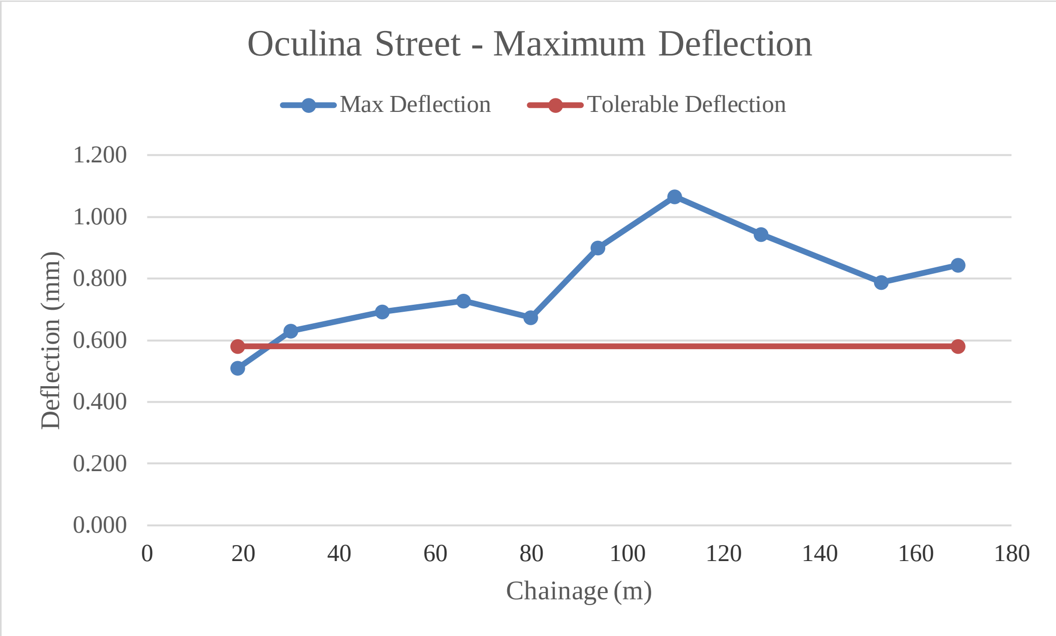

Figure 19 - Oculina Street Maximum Deflection

4.2.2 Deflection Ratio

Oculina Street has a representative deflection ratio of 0.507mm and with a mean and standard deviation of 0.542mm and 0.027mm respectively. As seen below in Figure 20, all the tests are consistently within the zero to 0.600mm zone, which as per the Transport and Main Roads (2012) relates to a weak unbound pavement.

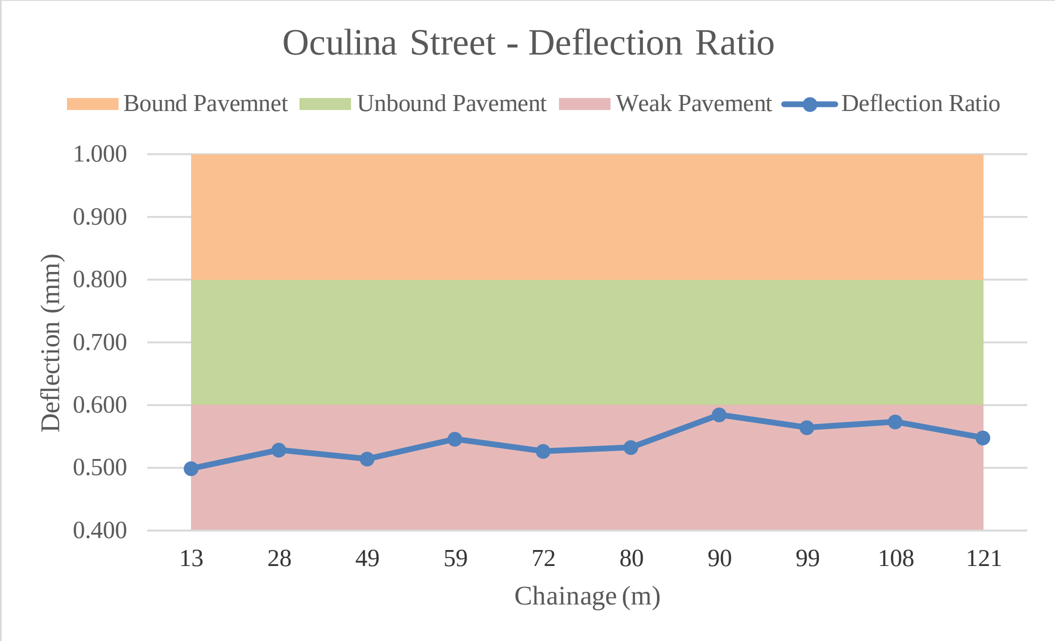

Figure 20 - Oculina Street Deflection Ratio

4.2.3 Curvature Function

Oculina Street has an average relationship between the CF values and the maximum deflections of 34.9% for the length of the road. These percentages are consistent along the length of the road and are at the top of the expected limit set by Transport and Main Roads (2012) of 35%. The FWD testing at chainage 19 shows this relationship as 38.7% which suggests that the granular base course has a low strength.

As seen in Figure 21 below the curvature function values range from 0.197mm to 0.329mm and have a mean of 0.268mm. Transport and Main Roads (2012) specify that roads with a curvature function greater than 0.4mm generally lack stiffness, and pavements with a curvature function of less than 0.2mm indicate a stiff pavement.

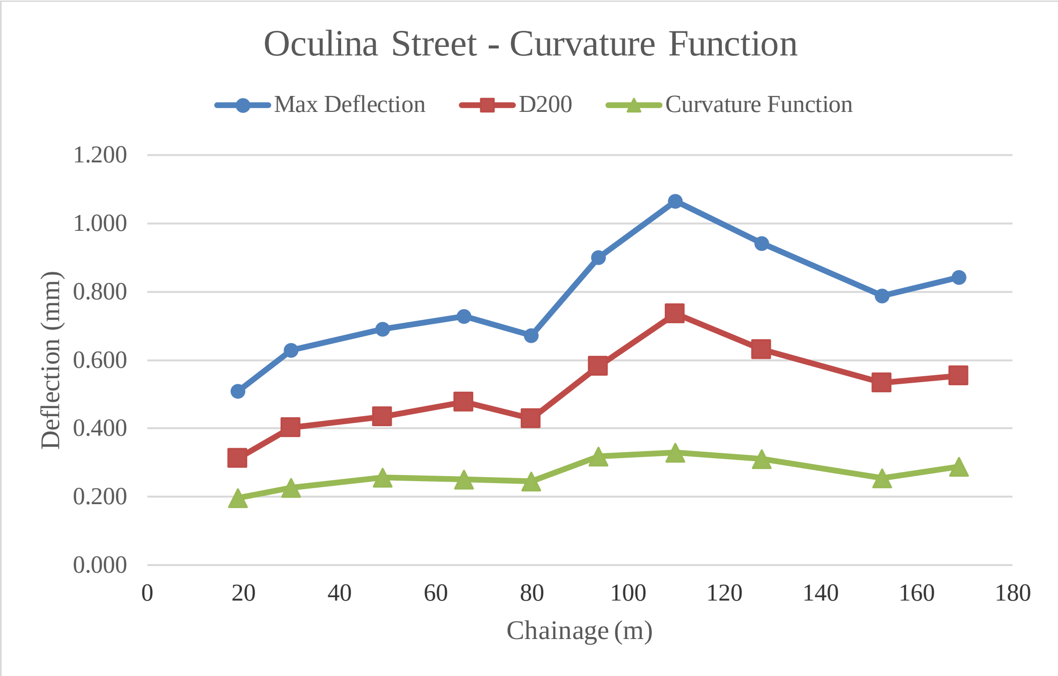

Figure 21 - Oculina Street Curvature Function

4.2.4 Subgrade Response

Oculina Street has a representative D_{900} value of 0.091mm and with a mean and standard deviation of 0.064mm and 0.021mm respectively. By applying the representative D_{900} value of 0.091mm to Figure 13 the estimated CBR at the time of testing was determined as 20.5%.

4.3 Laysan Street Analysis

Laysan Street was selected as it was not developed within the peak residential market demands. This street has been in service for approximately four and a half years and is suggested to have been structurally impacted less than Saba and Oculina Street by the construction traffic used in residential construction. Visual inspection of this street highlighted that small amounts of aggregate within the asphalt had been dislodged from the screwing effect of the heavy vehicles.

Laysan Street As-Constructed pavement design is detailed as per below:

- Design ESA – 5.86×10^3
- 30mm Asphalt Surfacing (DG10)
- 100mm Base Course (DMR Type 2.1)
- 100mm Sub-Base Course (DMR Type 2.3)
- 150mm Lower Sub-Base Course (DMR Type 2.5)
- 250mm Subgrade Improvement (CBR 10 Minimum)
- Subgrade CBR = 1.5%

Normalised deflection results were received from Northern Pavement Consultants for each of the 10 tests performed on Laysan Street. These deflection results are displayed in Table 6 below.

Table 6 - Laysan Street FWD Test Results

Normalised to 40 kN Deflection Results																	
Road	Laysan Street							**Start Date of Testing**			8/25/2016		**Target Load (kN)**			40	
Comments	Refer to Location Testing Diagram for further information							**Start Reference**			FWD 0.0 m = Sunhaven Boulevard Kerb line						
FWD Chainage (m)	**Deflections (microns)**																
	Geophone Radius (mm)																
	0	200	300	400	500	600	700	800	900	1000	1200	1400	1600	1800	2000	2250	2500
13	0.546	0.363	0.253	0.185	0.139	0.111	0.091	0.078	0.065	0.057	0.042	0.034	0.028	0.025	0.020	0.017	0.014
28	0.647	0.444	0.316	0.235	0.178	0.143	0.114	0.095	0.078	0.066	0.048	0.036	0.030	0.028	0.022	0.018	0.016
49	0.585	0.395	0.274	0.202	0.153	0.123	0.101	0.087	0.073	0.063	0.047	0.039	0.033	0.030	0.025	0.020	0.016
59	0.577	0.386	0.267	0.198	0.150	0.122	0.100	0.086	0.073	0.063	0.047	0.039	0.032	0.030	0.025	0.021	0.017
72	0.667	0.446	0.313	0.237	0.180	0.143	0.115	0.098	0.081	0.070	0.052	0.044	0.036	0.034	0.027	0.023	0.018
80	0.642	0.447	0.330	0.257	0.200	0.162	0.132	0.111	0.090	0.077	0.052	0.040	0.032	0.029	0.023	0.019	0.015
90	0.654	0.442	0.312	0.234	0.176	0.142	0.114	0.096	0.079	0.069	0.051	0.042	0.034	0.033	0.027	0.022	0.018
99	0.616	0.417	0.295	0.226	0.174	0.144	0.119	0.103	0.086	0.075	0.055	0.046	0.038	0.036	0.029	0.025	0.019
108	0.639	0.434	0.307	0.234	0.180	0.148	0.121	0.104	0.088	0.076	0.057	0.047	0.038	0.036	0.030	0.025	0.017
121	0.743	0.525	0.386	0.300	0.233	0.188	0.153	0.130	0.109	0.095	0.071	0.060	0.049	0.044	0.037	0.031	0.024

4.3.1 Maximum Deflection

Figure 22 demonstrates that all 10 test points along Laysan Street are within acceptable limits to the tolerable deflection as per the TMR's Pavement Rehabilitation Manual. The representative deflection for Laysan Street is 0.561mm, with a mean and standard deviation of 0.632mm and 0.055mm respectively. The maximum deflection of 0.743mm occurred at chainage 121. The tolerable deflection as calculated from Figure 12 was determined as 0.640mm.

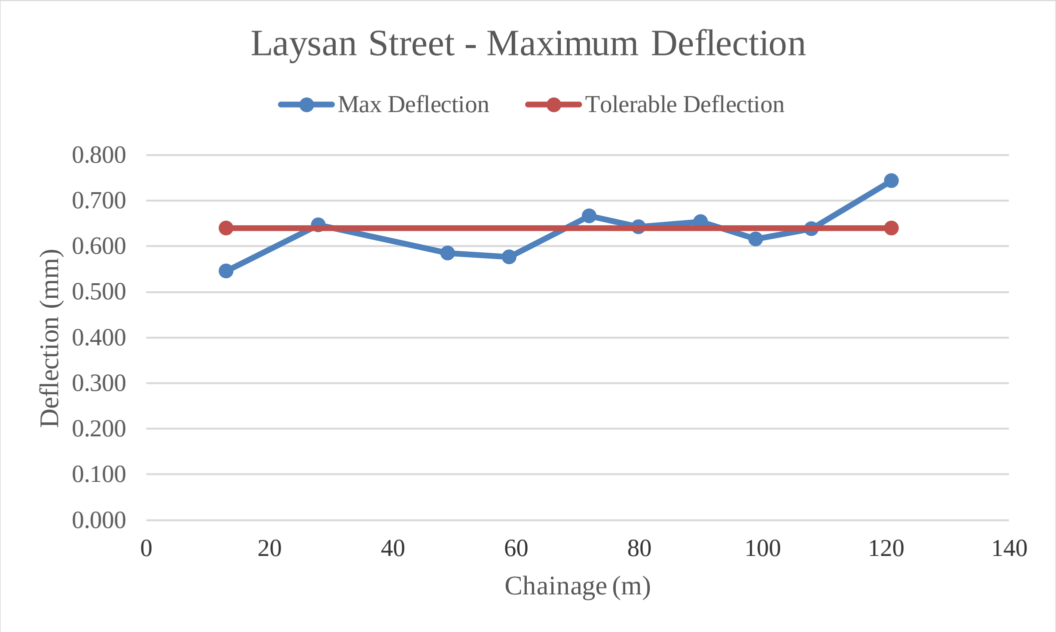

Figure 22 - Laysan Street Maximum Deflection

4.3.2 Deflection Ratio

Laysan Street has a representative deflection ratio of 0.581mm and with a mean and standard deviation of 0.581mm and 0.016mm respectively. As seen in Figure 23 below all the tests are consistently within the zero to 0.600mm zone, which as per the Transport and Main Roads (2012) relates to a weak unbound pavement. This however is only just outside of the 0.600mm to 0.800mm zone which relates to an unbound pavement in good condition.

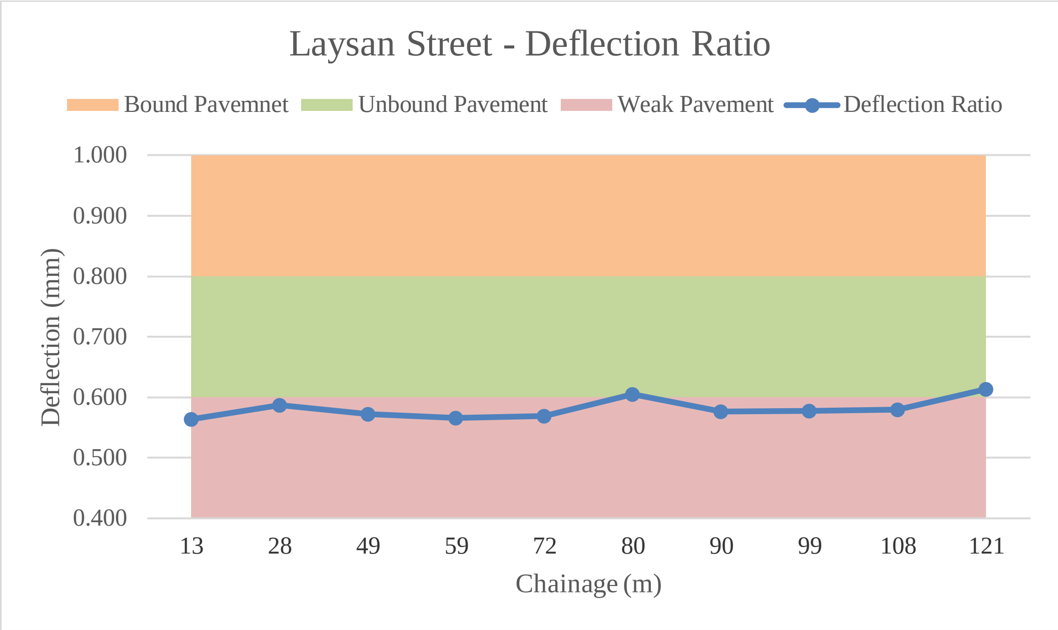

Figure 23 - Laysan Street Deflection Ratio

4.3.3 Curvature Function

Laysan Street has an average relationship between the CF values and the maximum deflections of 32.0% for the length of the road. These percentages are consistent along the length of the road and are at the top end of the expected limits set by Transport and Main Roads (2012) of 35%. Values higher than 35% may indicate that the granular base course has a low strength.

As seen in Figure 24 below the curvature function values range from 0.183mm to 0.221mm and have a mean of 0.202mm. Transport and Main Roads (2012) specifies that roads with a curvature function less than 0.200mm indicate a stiff pavement.

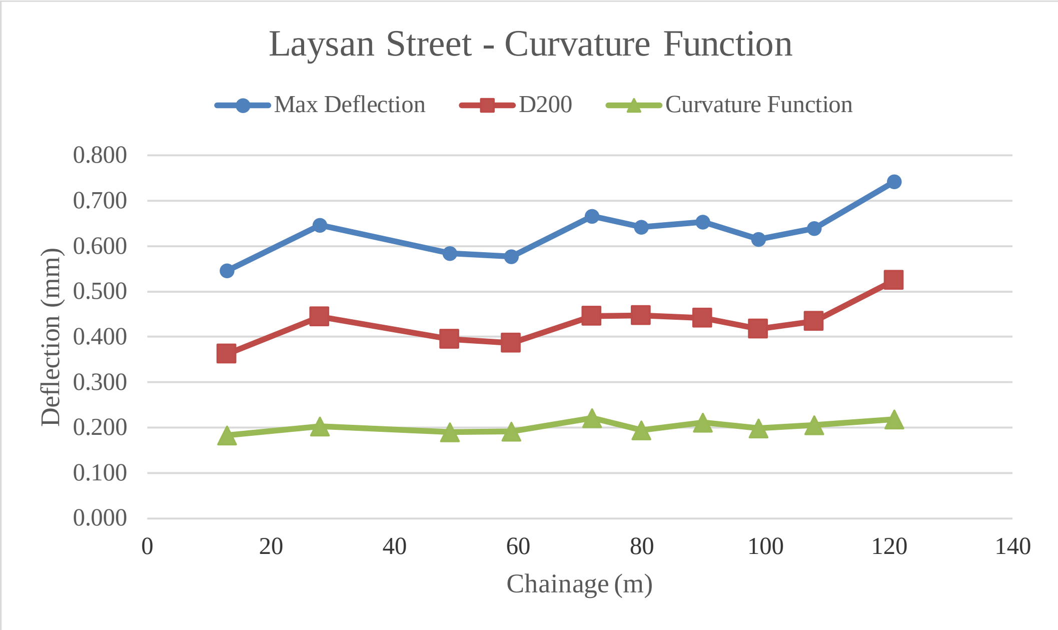

Figure 24 - Laysan Street Curvature Function

4.3.4 Subgrade Response

Laysan Street has a representative D_{900} value of 0.098mm and with a mean and standard deviation of 0.082mm and 0.012mm respectively. By applying the representative D_{900} value of 0.098mm to Figure 13 the estimated CBR at the time of testing was determined as 17.5%.

4.4 Yanuca Street Analysis

Yanuca Street was selected as it was not developed within the peak residential market demands. This street has been in service for approximately two years and testing suggests that the structural impact from the construction traffic used in residential construction is very minor. Visual inspection of this street highlighted that small amounts of aggregate within the asphalt had been dislodged from the screwing effect of the heavy vehicles.

Yanuca Street As-Constructed pavement design is detailed as per below:

- Design ESA – 6.00 x 10^4
- 30mm Asphalt Surfacing (DG10)
- 125mm Base Course (DMR Type 2.1)
- 200mm Sub-Base Course (DMR Type 2.3, 1.5% Cement)
- Subgrade CBR = 4%

It was decided at the time of construction by the engineers, that a small percentage of cement be added to the sub-base course layers. According to the Austroads design charts this was not required, however it was added in an effort to reduce the initial pavement failure within the access streets.

Normalised deflection results were received from Northern Pavement Consultants for each of the 10 tests performed on Yanuca Street. These deflection results are displayed in Table 7 below.

Table 7 - Yanuca Street FWD Test Results

Normalised to 40 kN Deflection Results																	
Road	Yanuca Street							**Start Date of Testing**			8/25/2016		**Target Load (kN)**			40	
Comments	Refer to Location Testing Diagram for further information							**Start Reference**			FWD 0.0 m = Nyota Street Kerb line						
FWD Chainage (m)	**Deflections (mm)**																
	Geophone Radius (mm)																
	0	200	300	400	500	600	700	800	900	1000	1200	1400	1600	1800	2000	2250	2500
19	0.442	0.293	0.212	0.168	0.135	0.114	0.096	0.084	0.069	0.060	0.041	0.035	0.029	0.026	0.021	0.016	0.013
36	0.791	0.527	0.360	0.262	0.191	0.149	0.117	0.098	0.080	0.068	0.049	0.039	0.031	0.029	0.024	0.020	0.017
47	0.615	0.434	0.319	0.248	0.194	0.158	0.126	0.107	0.089	0.076	0.059	0.047	0.038	0.035	0.029	0.024	0.018
70	0.475	0.353	0.268	0.217	0.177	0.148	0.120	0.104	0.087	0.076	0.058	0.047	0.037	0.034	0.027	0.022	0.017
92	0.511	0.365	0.269	0.215	0.174	0.148	0.125	0.109	0.094	0.082	0.063	0.051	0.042	0.038	0.031	0.026	0.019
108	0.627	0.404	0.285	0.224	0.181	0.154	0.130	0.114	0.097	0.084	0.062	0.054	0.042	0.039	0.032	0.027	0.020
129	0.336	0.237	0.182	0.156	0.137	0.122	0.107	0.095	0.081	0.071	0.052	0.043	0.036	0.033	0.027	0.022	0.016
146	0.515	0.363	0.268	0.210	0.166	0.137	0.112	0.095	0.079	0.067	0.051	0.041	0.033	0.030	0.025	0.021	0.017
161	0.653	0.456	0.332	0.263	0.206	0.165	0.128	0.105	0.085	0.070	0.050	0.039	0.032	0.029	0.024	0.020	0.015
174	0.217	0.160	0.123	0.099	0.081	0.072	0.062	0.057	0.049	0.045	0.035	0.030	0.025	0.023	0.019	0.015	0.011

4.4.1 Maximum Deflection

Yanuca street is the first road pavement that incorporated the additional 1.5% cement in the bottom layer of the pavement. Figure 25 demonstrates that all but one test result was within acceptable limits when compared to the tolerable deflection limit set by the TMR's Pavement Rehabilitation Manual.

The representative deflection for Saba Street is 0.730mm, with a mean and standard deviation of 0.518mm and 0.165mm respectively. The maximum deflection of 0.791mm occurs at chainage 36, where no noticeable pavement defects were located at and around chainage 36. Overall Yanuca Streets deflections were at an acceptable limit and there is no area of concerns. The tolerable deflection as calculated from Figure 12 was determined as 0.640mm.

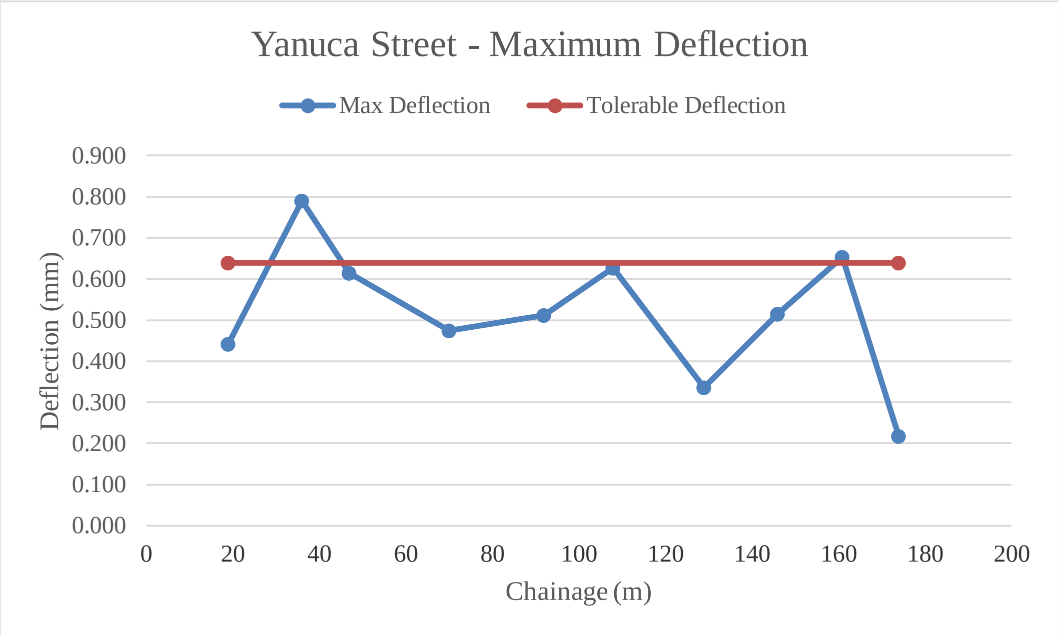

Figure 25 - Yanuca Street Maximum Deflection

4.4.2 Deflection Ratio

Yanuca Street has a representative deflection ratio of 0.560mm and with a mean and standard deviation of 0.606mm and 0.036mm respectively. As seen in Figure 26 below the test results fluctuate between the unbound pavement in good condition and the weak unbound pavement zones. However as per the Transport and Main Roads (2012) the deflection ratio is taken as the representative deflection ratio. This is the 10th percentile lowest deflection ratio assuming a 'normal' statistical distribution which places Yanuca Street within the weak unbound pavement zone.

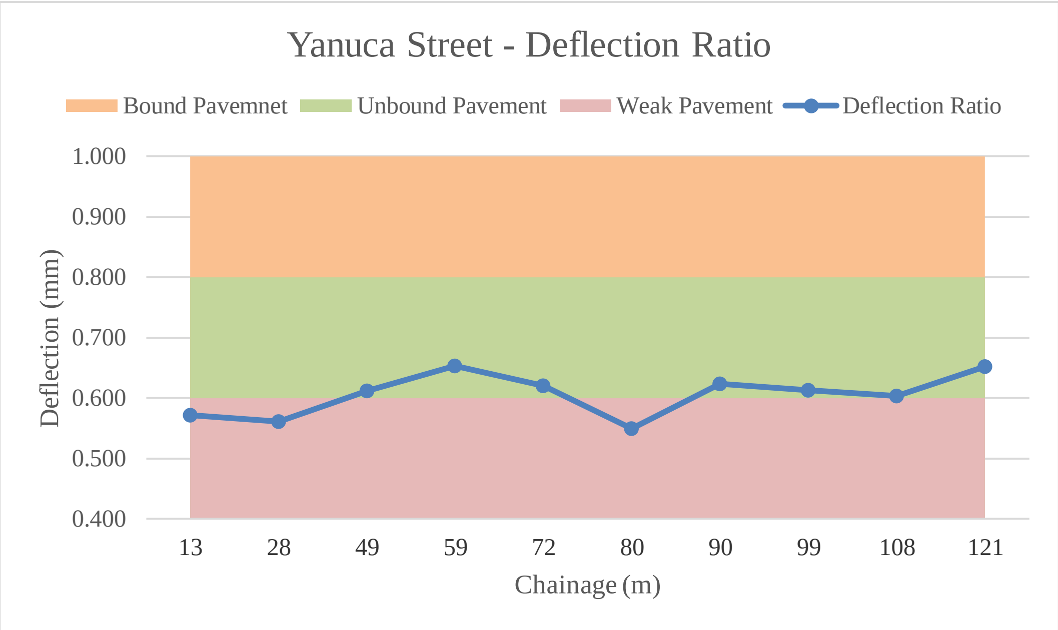

Figure 26 - Yanuca Street Deflection Ratio

4.4.3 Curvature Function

Yanuca Street has an average relationship between the CF values and the maximum deflections of 30.2% for the length of the road. This percentage is in the middle of the accepted range of 25% - 35% as set by Transport and Main Roads (2012). The FWD testing at chainage 108 shows this relationship as 35.6%, which suggests at this location the granular base course has a low strength.

As seen in Figure 27 below the curvature function values range from 0.122mm to 0.264mm and have a mean of 0.159mm. According to Transport and Main Roads (2012) a pavement with a curvature function of less than 0.2mm indicates a stiff pavement.

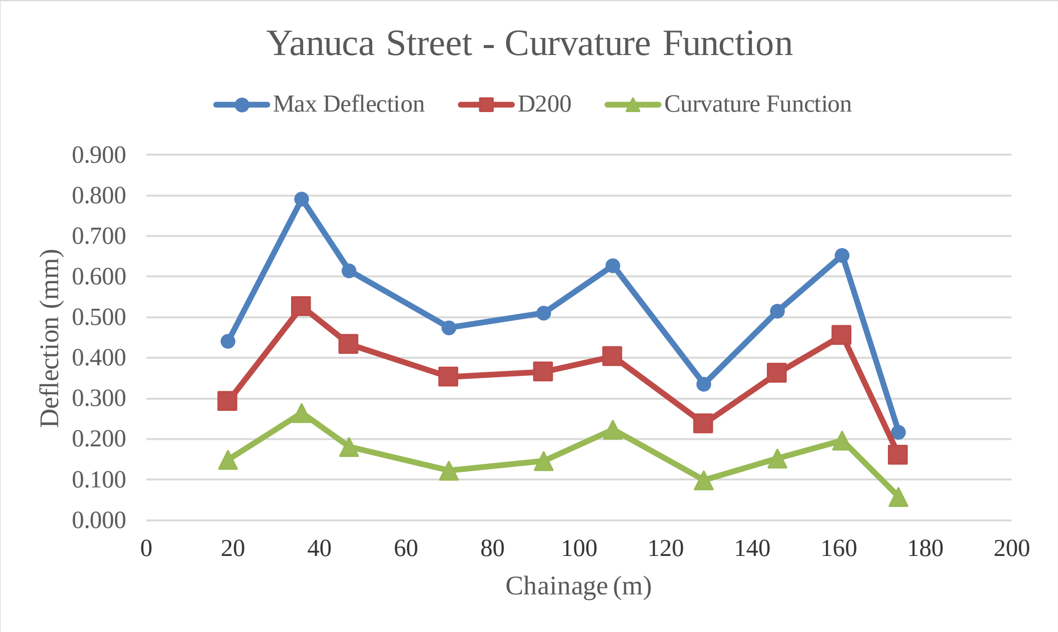

Figure 27 - Yanuca Street Curvature Function

4.4.4 Subgrade Response

Yanuca Street has a representative D_{900} value of 0.099mm and with a mean and standard deviation of 0.081mm and 0.014mm respectively. By applying the representative D_{900} value of 0.099mm to Figure 13 the estimated CBR at the time of testing was determined as 17.5%.

4.5 Columbus Street Analysis

Columbus Street was selected as it was just completed and has had very limited impact from the construction traffic used in residential construction.

Columbus Street As-Constructed pavement design is detailed as per below:

- Design ESA – 1.20 x 10^5
- 30mm Asphalt Surfacing (DG10)
- 125mm Base Course (DMR Type 2.1)
- 175mm Sub-Base Course (DMR Type 2.3, 1.5% Cement)
- Subgrade CBR = 5%

It was decided at the time of construction by the engineers, that a small percentage of cement be added to the sub-base course layers. According to the Austroads design charts this was not required, however it was added in an effort to reduce the initial pavement failure within the access streets.

Normalised deflection results were received from Northern Pavement Consultants for each of the 11 tests performed on Columbus Street. These deflection results are displayed in Table 8 below.

Table 8 - Columbus Street FWD Test Results

Normalised to 40 kN Deflection Results																	
Road	Columbus Street							**Start Date of Testing**		8/25/2016		**Target Load (kN)**			40		
Comments	Refer to Location Testing Diagram for further information							**Start Reference**		FWD 0.0 m = Emperor Boulevard Kerbline							
FWD Chainage (m)	**Deflections (mm)**																
	Geophone Radius (mm)																
	0	200	300	400	500	600	700	800	900	1000	1200	1400	1600	1800	2000	2250	2500
37	0.638	0.393	0.267	0.200	0.155	0.127	0.103	0.088	0.072	0.061	0.044	0.034	0.026	0.024	0.019	0.015	0.012
51	0.462	0.262	0.165	0.119	0.087	0.069	0.055	0.047	0.038	0.035	0.026	0.023	0.020	0.020	0.016	0.013	0.011
75	0.667	0.421	0.286	0.205	0.146	0.107	0.076	0.059	0.042	0.032	0.019	0.015	0.011	0.012	0.010	0.008	0.007
92	1.205	0.833	0.581	0.424	0.305	0.226	0.164	0.128	0.100	0.080	0.054	0.044	0.035	0.033	0.027	0.023	0.020
100	0.813	0.552	0.372	0.269	0.194	0.147	0.108	0.086	0.066	0.053	0.033	0.025	0.019	0.019	0.015	0.014	0.012
108	0.576	0.352	0.240	0.173	0.122	0.090	0.065	0.052	0.040	0.033	0.021	0.017	0.014	0.015	0.012	0.011	0.010
117	0.540	0.385	0.294	0.248	0.210	0.184	0.160	0.140	0.120	0.104	0.075	0.056	0.040	0.033	0.024	0.018	0.013
128	0.426	0.271	0.201	0.166	0.138	0.121	0.104	0.092	0.079	0.069	0.051	0.038	0.026	0.021	0.014	0.010	0.007
137	0.394	0.235	0.165	0.129	0.103	0.087	0.072	0.061	0.051	0.043	0.028	0.021	0.015	0.014	0.011	0.009	0.009
148	0.370	0.216	0.150	0.118	0.095	0.079	0.063	0.054	0.044	0.036	0.024	0.018	0.014	0.014	0.011	0.010	0.009
158	0.393	0.247	0.183	0.152	0.126	0.109	0.090	0.077	0.065	0.055	0.039	0.030	0.021	0.019	0.014	0.011	0.009

4.5.1 Maximum Deflection

Figure 28 demonstrates that the majority of Columbus street is within acceptable tolerable deflection limits set by the TMR's Pavement Rehabilitation Manual. There is however an isolated pocket in the middle of the testing between chainages 75 and 100 that is outside the accepted limits. The maximum deflection occurs at chainage 92 with a deflection of 1.205mm. The deflection at this location is twice the acceptable limit.

The representative deflection for Columbus Street was 0.917mm, with a mean and standard deviation of 0.585mm and 0.259mm respectively. The tolerable deflection as calculated from Figure 12 was determined as 0.640mm.

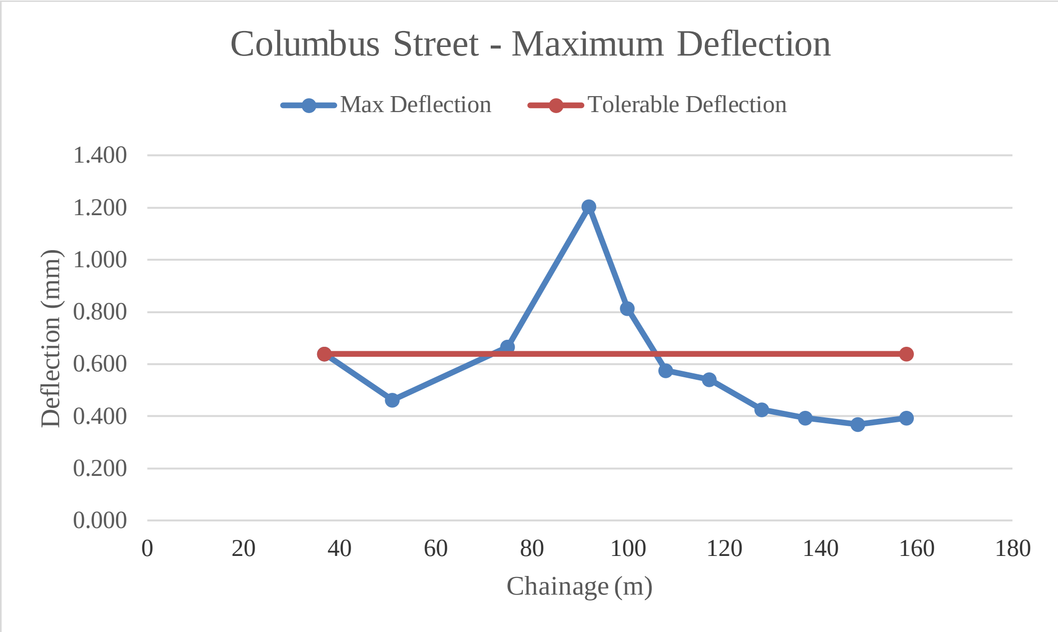

Figure 28 - Columbus Street Maximum Deflection

4.5.2 Deflection Ratio

Columbus Street has a representative deflection ratio of 0.475mm and with a mean and standard deviation of 0.537mm and 0.049mm respectively. As seen in Figure 29 below all but one test result is located within the weak unbound pavement zone.

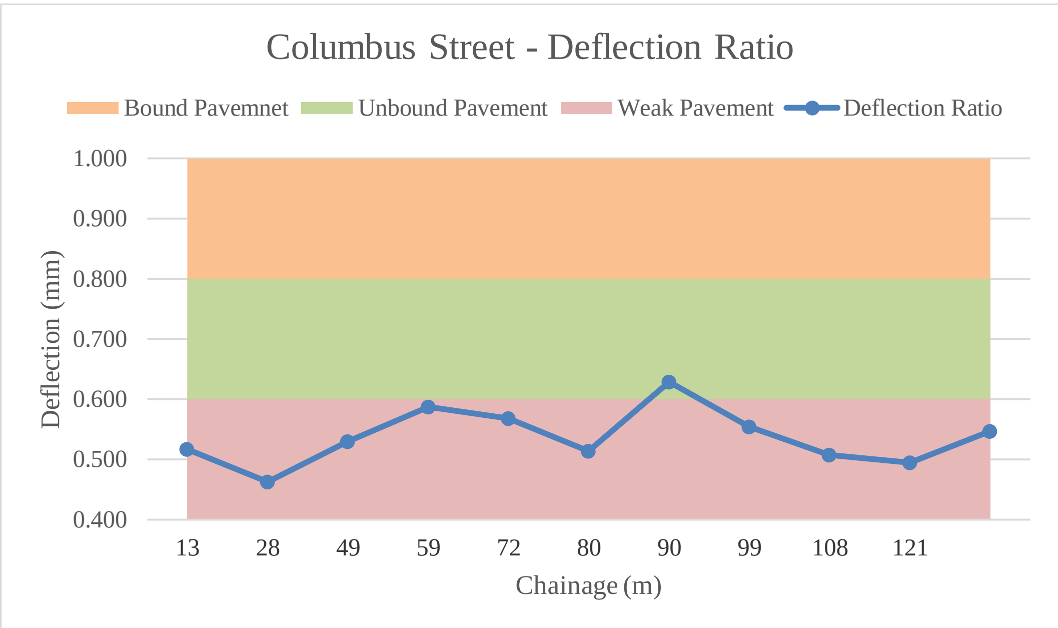

Figure 29 - Columbus Street Deflection Ratio

4.5.3 Curvature Function

Columbus Street has an average relationship between the CF values and the maximum deflections of 36.8% for the length of the road. This percentage exceeds the accepted range of 25% - 35% as set by Transport and Main Roads (2012). The FWD testing at chainages 51, 137 and 148 shows this relationship as 43.3%, 40.4% and 41.6% respectively which suggests at these locations the granular base course has a low strength.

As seen in Figure 30 below the curvature function values range from 0.146mm to 0.372mm and have a mean of 0.211mm and standard deviation of 0.069mm. According to Transport and Main Roads (2012) a pavement with a curvature function of less than 0.2mm indicates a stiff pavement. It can clearly be seen in Figure 30 that between chainages 75 to 100 the pavement deflects twice as much as the remaining test points. This would suggest there is an isolated pavement issue in this area.

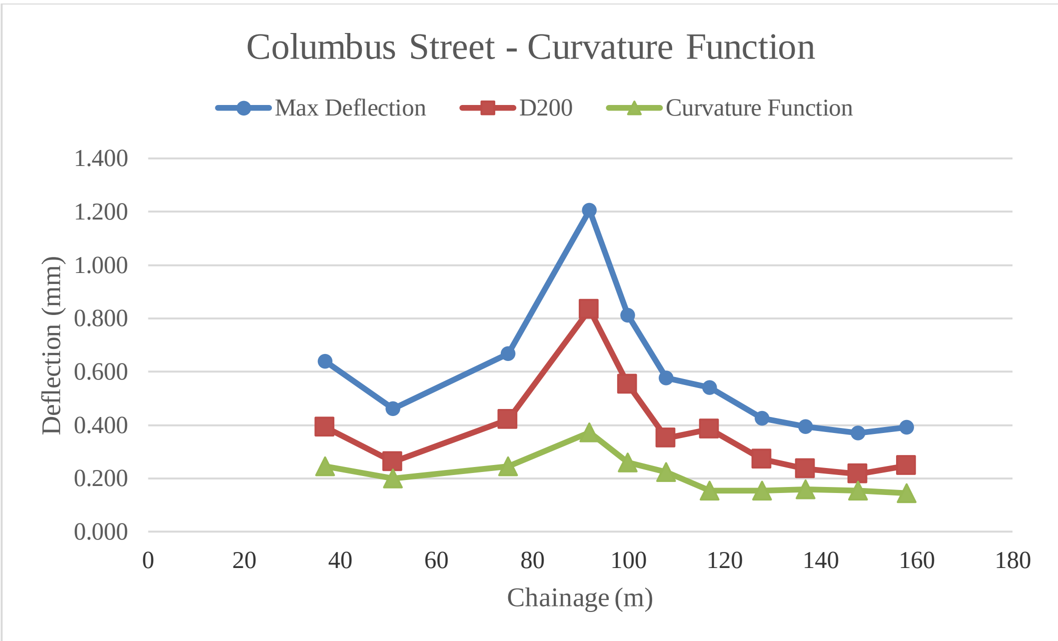

Figure 30 - Columbus Street Curvature Function

4.5.4 Subgrade Response

Columbus Street has a representative D_{900} value of 0.099mm and with a mean and standard deviation of 0.065mm and 0.026mm respectively. By apply the representative D_{900} value of 0.099mm to Figure 13 the estimated CBR at the time of testing was determined as 17.5%.

4.6 Summary of FWD Analysis

The aim of the falling weight deflectometer testing was to establish if the relativity new roads were showing signs of premature failure. These five roads are between 5% and 30% through their expected design life. This section performs an analysis of the following criteria to establish possible relationships:

- Maximum Deflections
- Deflection Ratio
- Curvature Function
- Subgrade Response

Figure 31 summarises the representative deflection of all five roads tested with the falling weight deflectometer. It was expected that under normal traffic loads, each road would rank from the oldest too youngest and highest to lowest deflections respectively. This however was not the case as Saba Street has deflected almost twice as much as the next highest road Oculina Street, while the remanding three roads all have similar deflection limits. The tolerable deflection limits as detailed by section 3.3.1 were achieved by Laysan, Yanuca and Columbus Street. Saba and Oculina Street however did not meet the tolerable deflection criteria set by Transport and Main Roads (2012).

As Columbus Street was constructed last it should have the lowest representative deflection value of the five roads. But in fact, of the three roads meeting the tolerable deflection requirement, Columbus Street had the highest deflection. From the analysis of Columbus

Street two outliers where identified at chainages 92 and 100. It was clear that there is a major issue with the small portion of the road. Removing these values and recalculating the representative deflection value resulted in a better representation of the condition of Columbus Street. This can be seen in Figure 31 Columbus Street (Adjusted)

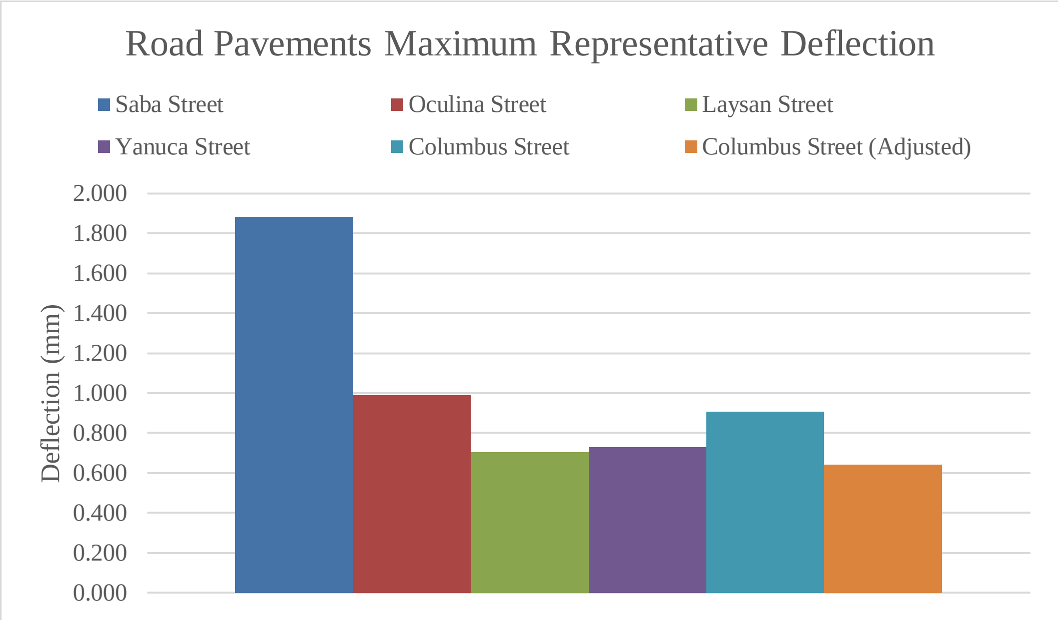

Figure 31 - Road Pavements Representative Maximum Deflection

The representative deflection ratios for all five roads are displayed below in Figure 32. It is unusual that all roads fail to comply with Transport and Main Roads (2012) criteria, seeing as these roads are between 5% and 30% through their expected design life. For deflection ratio results derived using a FWD with a 40 kN loading the following ranges can be adopted;

- A deflection ratio of greater than 0.8 would indicate a bound pavement;
- A deflection ratio of between 0.6 and 0.8 would be expected for a good quality unbound pavement with a thin asphalt surfacing or seal; and
- A deflection ratio of less than 0.6 would indicate a possible weakness for an unbound pavement with a thin asphalt surfacing or seal (TMR, 2012).

Therefore, ideally for a new road you would expect values for a good quality unbound pavement to range from 0.6 to 0.8. For these five roads the values range from 0.475mm to 0.560mm, with Columbus Street having the lowest ratio. This indicates a possibility that these unbound pavements are structurally weak.

It is very unusual that the newest road constructed has the lowest deflection ratio. This would indicate that Columbus Street is not distributing the load outwards from the impact location and is suffering more from a point load. This indicates that the cement added to the lower layers of the pavement has not aided in distributing the loads laterally through the pavement layers, but is however reducing the maximum deflections.

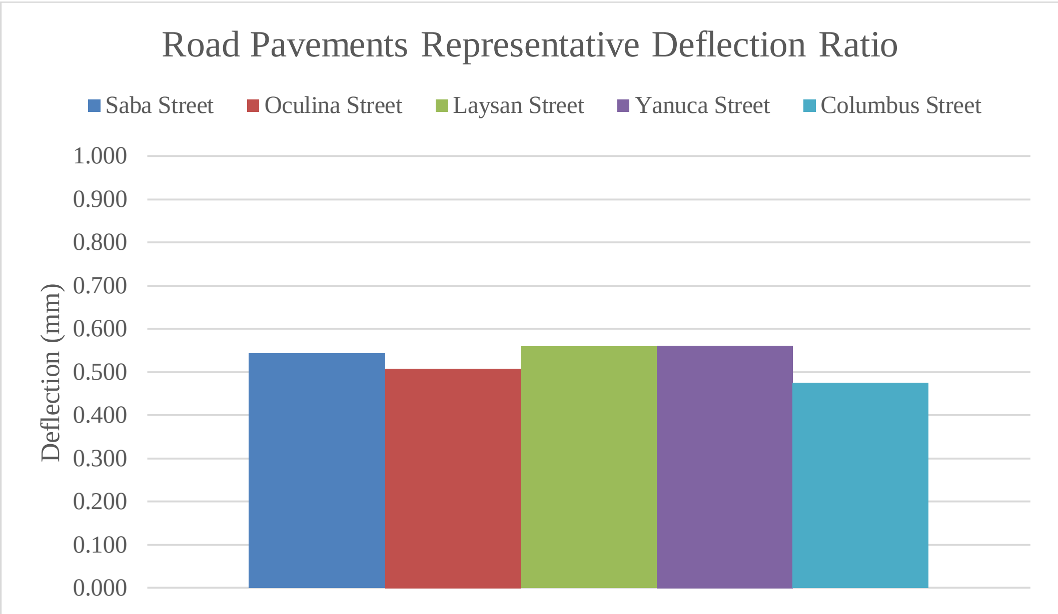

Figure 32 - Road Pavements Representative Deflection Ratio

The average relationship between the curvature function values and the maximum deflections should range between 25% and 35% as specified by Transport and Main Roads (2012). All roads were at the top end of this range, which indicates that the granular base course has possibly a low strength. It should be noted that the base course for all roads was constructed with the same material and very similar thickness, therefore it is expected that each road would result is a similar percentage.

As seen in Figure 33 below the curvature function values for the five roads range from 0.159mm to 0.497mm. Values higher than 0.400mm for the curvature function may indicate that the pavement is lacking stiffness, is a very thin pavement, or is a pavement with a cracked asphalt surface. While values lower than 0.200mm indicate a stiff pavement.

Both Saba and Oculina Street were constructed within the peak of the residential demand in the market. This has been reflected in the curvature function as they are outside the acceptable limits set by Transport and Main Roads (2012). Yanuca and Columbus Street both had the inclusion of cement in the bottom layer of the pavement and have withstood the impacts of the construction traffic used in the construction of residential dwellings.

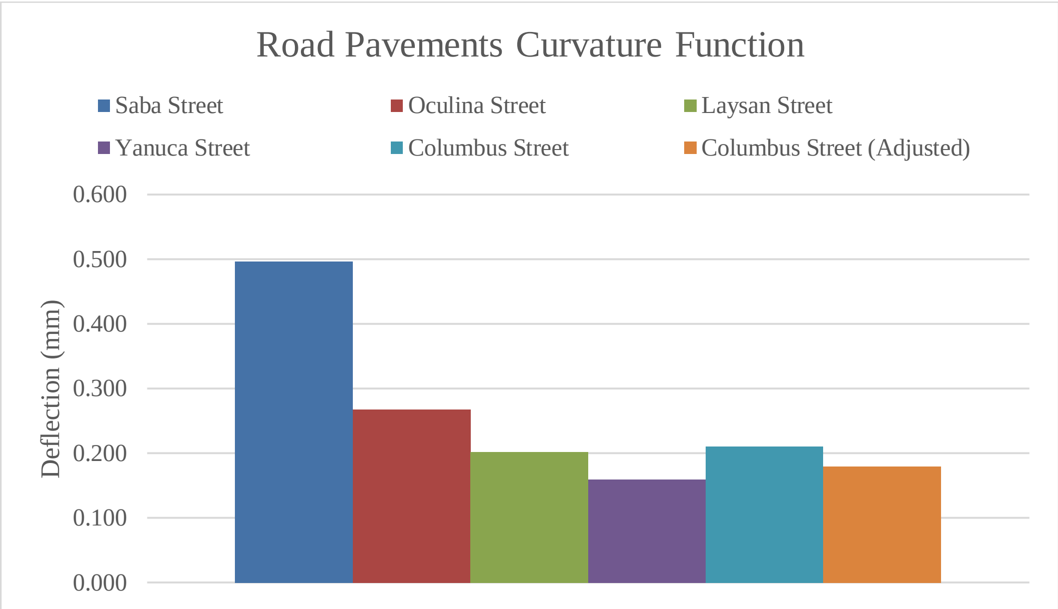

Figure 33 - Road Pavements Curvature Function

The subgrade response CBR value at the time of the testing was calculated by using the representative D_{900} value and Figure 13. When the estimated CBR values were compared to the original CBR results they were 4 times higher than when originally tested. As there is such a difference in results the estimated CBR values will be used with caution.

5.0 CHAPTER 5 – Pavement Design Process

In order to assess the current Townsville City Council method and the proposed alternative method, a number of assumptions need to be made to ensure the basis of the designs are consistent. Refer to the following assumptions for the trial pavement Design:

- Number of lot contributing to the traffic volumes = 40 Lots
- In-sit-u Subgrade CBR value = 3.0%
- 30mm Asphalt Surfacing (DG10)
- 150mm Base Course Layer (TMR Type 2.1)
- Sub-Base Course thickness varies for each design (TMR Type 2.3)

5.1 Current Design ESA Method

Based on the Design ESA equations in section 3.4 and Table 3 the current design ESA method was calculated with the following values:

- AADT = 400 (40 dwellings times 10 vehicles per day).
- DF = 1.0 (due to the narrow pavement width all vehicles traveling along the road are assumed to take the same path)
- %HV = 4% (as specified in Table 3 for ‘Local access with no busses’)
- LDF = 1.0 (only one lane).
- CGF = 22 (as specified in Table 3 for ‘Local access with no busses’).
- N_{HVAG} = 2.1 (as specified in Table 3 for ‘Local access with no busses’).
- ESA/HVAG = 0.3 (as specified in Table 3 for ‘Local access with no busses’).
- $N_{DT} = 2.70 \times 10^5$
- <u>Design ESA = 8.10×10^4</u>

5.2 Alternative Design ESA Method

Based on the Design ESA equations in section 3.4 and Table 3. The alternative design ESA method takes into the initial year of increase heavy vehicles, the following values were used:

- AADT = 200 (40 dwellings times 5 heavy vehicles per day).
- DF = 1.0 (due to the narrow pavement width all vehicles traveling along the road are assumed to take the same path)
- %HV = 100% (As the traffic loadings are considered seasonal the following assumptions were made in accordance with Section 12.7.1 of the Austroads Guide to Pavement Technology Part 2 Manual, 2012. In such situations where the design traffic needs to be adjusted for the short-term heavy loadings, the maximum daily heavy vehicle traffic per annum is used. Therefore, the 100% relates to the peak %HV of five concrete trucks during the pouring of the base slab of a residential dwelling.)
- LDF = 1.0 (only one lane).
- CGF = 1.0 (as this calculation is only for one year while the dwellings are constructed).
- N_{HVAG} = 2.3 (as specified in Table 3 for 'Local access in industrial area,' this increased value was adopted as during this first year of service the pavement is subject to much higher heavy vehicle loads).
- ESA/HVAG = 0.6 (as specified in Table 3 for 'Local access in industrial area,' this increased value was adopted as during this first year of service the pavement is subject to much higher heavy vehicle loads).
- $N_{DT} = 1.68 \times 10^5$
- <u>Design ESA = 1.01×10^5 (Initial year of service)</u>

To calculate the total Design ESA for the trial road the initial loading must be added to the current design ESA for the access street with a 20-year design life. Therefore;

Total Design ESA = $8.10 \times 10^4 + 1.01 \times 10^5 = \underline{1.82 \times 10^5}$

This new Design ESA method will enable the pavement depth to be designed adequately to handle the peak traffic load and therefore perform as desired.

5.3 Comparison of Pavement Designs

The Design ESA, is the only value that changes between the current and alternative ESA methods. Therefore, it was necessary to calculate the pavement depth for the two trial pavements and also recalculate the ESA and pavement for the five selected roads using the alternative method. Refer to Appendix C for the pavement design calculations and to Table 9 for a comparison of the calculated pavement designs.

Table 9 - Comparison of Pavement Design

Road Name	**CBR Design**	**ESA's**	**Asphalt Surface (mm)**	**Base Course (mm)**	**Sub Base (mm)**	**Lower Sub Base (mm)**	**Sub Grade Replacement (mm)**	**Total Pavement (mm)**
Trial Pavement (Current Method)	**3.0**	8.10E+04	30	Type 2.1	Type 2.3	N/A	N/A	**410**
				150	230	0	0	
Trial Pavement (Alternative Method)	**3.0**	1.82E+05	30	Type 2.1	Type 2.3	N/A	N/A	**450**
				150	270	0	0	
Saba Street (Current Method)	**4.0**	5.86E+03	25	Type 2.1	Type 2.3	N/A	N/A	**320**
				100	195	0	0	
Saba Street (Alternative Method)	**4.0**	4.62E+04	30	Type 2.1	Type 2.3	N/A	N/A	**395**
				150	215	0	0	
Oculina Street (Current Method)	**1.5**	5.86E+03	25	Type 2.1	Type 2.3	Type 2.5	N/A	**565**
				100	100	340	0	
Oculina Street (Alternative Method)	**1.5**	6.13E+04	30	Type 2.1	Type 2.3	N/A	CBR 20	**605**
				150	225	0	200	
Laysan Street (Current Method)	**1.5**	5.86E+03	30	Type 2.1	Type 2.3	Type 2.5	CBR 10	**630**
				100	100	150	250	

Laysan Street (Alternative Method)	**1.5**	4.62E+04	30	Type 2.1	Type 2.3	N/A	CBR 20	**595**
				150	215	0	200	
Yanuca Street (Current Method)	**4.0**	6.00E+04	30	Type 2.1	Type 2.3 (1.5% cement)	Type 2.5	N/A	**355**
				125	200	0	0	
Yanuca Street (Alternative Method)	**4.0**	1.18E+05	30	Type 2.1	Type 2.3	N/A	N/A	**370**
				150	190	0	0	
Columbus Street (Current Method)	**5.0**	1.20E+05	30	Type 2.1	Type 2.3 (1.5% cement)	Type 2.5	N/A	**330**
				125	175	0	0	
Columbus Street (Alternative Method)	**5.0**	1.70E+05	30	Type 2.1	Type 2.3	N/A	N/A	**345**
				150	165	0	0	

All pavement designs except for Laysan Street increased in depth by an average of 30mm. For Laysan Street the materials did however change when compared to the alternative design. Stronger materials were used to achieve a higher total strength with thinner pavement. This is relatively a small increase in upfront pavement costs when compared to possible rehabilitation costs that could be incurred. Using local rates, we can calculate the increase in pavements costs for the trial pavement. The following rates, length and width of road will be used to calculate the total pavement costs:

- Asphalt = $18/m^2
- Type 2.1 Gravel = $91/m^3
- Type 2.3 Gravel = $85/m^3
- Length of Road = 400m (20 lots each side at 20m each)
- Width of Road = 6m

The trial pavement based on current methods is valued at $122,880 and for the alternative method $131,040. The difference of $8,160 relates to $204/lot for the trial 40 lot stage.

6.0 CHAPTER 6 – Recommendations and Conclusion

6.1 Recommendations

The following recommendations are the result of this research and can be further developed or adopted to ensure that the pavement design for an access street can meet the required design life.

- It is recommended that by using the alternative design method for calculating the ESA, the pavement can be designed to cater for the initial peak in traffic demands created by the construction vehicles used in the construction of residential dwellings.
- As even the newly constructed Columbus Street failed to meet the accepted deflection ratio range as set by Transport and Main Roads (2012). It is suggested to increase the base course layer too a minimum 150mm thickness.

6.2 Achievement of Objectives

The primary objectives of the project have been achieved and are summarized below:

1. Research was conducted on road pavements with respect to the design ESA values adopted for the design of access streets. It was found that heavy vehicles play a major role in the failure of road pavements and the impacts of these heavy vehicles on access streets had previously not been explored.
2. North Shore estate was selected as the test site, and five roads were selected for testing. These roads all had similar design ESA's and subgrade CBR values, however each road was constructed within different peak residential periods and had different alternative design approaches.

3. Numerous test points were selected within each road pavement. Each location was selected to ensure that it did not coincide with an underground service.
4. Several different Transport and Main Roads testing methods were conducted with the raw FWD test data. This resulted in the ability to compare each road and to assess if the pavement had been effected by the construction vehicles used in the construction of residential dwellings.
5. The initial peak design ESA was established and was found to exceed the previously calculated design ESA for an expected 20-year life. This then relates to the weak and poor quality pavements found in the test data.
6. An alternative pavement design method was created which caters for the initial peak in traffic loads experienced within the first year the pavement is in service.
7. This objective was to repeat the FWD testing on Columbus Street after the construction traffic had finished. This would then be compared with the base line data to established possible impacts. This objective was to be completed if time and resources permitted, unfortunately more than half of the dwellings within the Stage 570 are yet to start and the second round of testings at this time would have not provided any useful data.

6.3 Further Research

Continued research is required to ensure better engineering solutions and assets are developed for our ever evolving community. Further research and development of new procedures on how pavements are designed and constructed could include the following:

- Conduct field tests by constructing road pavements with the alternative design method during a peak residential property demand period. This pavement can then be tested once completed and then again after a year, to test if it withstood the impacts of construction vehicles used in dwelling construction

- Visual inspections of the pavement conducted during the site selection analysis indicated that there were numerous locations where asphalt damage had occurred. Further investigations are required into the screwing effect on the asphalt surface from heavy vehicles turning off the road and performing u-turns. Possible solutions may include poly modified asphalt surfacing to assist with the turning manoeuvres of heavy vehicles on access streets.
- Using Circly or equivalent software to preform backwards calculations to determine the number of service years remaining in the pavement.
- Re conduct the FWD testing on Columbus Street once all residential dwellings are completed. These new test results can then be compared to the initial test results as detailed in this report.

6.4 Conclusion

Numerous literature reviews have all specified that the volume of heavy vehicles contributing to traffic on roads have played a major role in the structural failure of pavements. The majority of past pavement research has focused on high order roads with large traffic demands. It is believed that these design methods have then been scaled down for the design of access streets.

Testing of five existing access streets within the residential subdivision ‘North Shore Estate’ indicated that the pavements designed with traditional methods were not in good structural condition. The two roads that had the modified design responded well to the testing and only failed to meet the Transport and Main Roads criteria on one occasion. The modified design had reduced the maximum deflections recorded, however did not aid in distributing the loads laterally through the pavement layers.

As some of the tested pavements were designed with traditional methods, these have resulted in poor structural strength considering they are only 5% to 30% through their intended design life. It has then been assumed that this was due the initial increase in traffic loads caused by the heavy vehicles used to construct residential dwellings.

The initial peak in the design ESA calculated, exceeds the traditional design method which is based off the expected traffic over a 20 year deign period. This ultimately results in the pavement design not having the strength to withstand the initial impacts of the heavy vehicles used in the construction of dwellings. As the speed in which dwellings are to be constructed within the subdivision is not known, the alternative design ESA has been calculated for the worst case. The slight increase in initial pavement costs are very small when compared to possible rehabilitation costs that could be incurred by the local council, therefore this is seen as a viable solution.

There is sufficient literature and past cases where new residential pavements have failed prematurely which would allow for the continue this research. This is a real world application that can reduce future rehabilitation costs for local councils and these methods can be applied now rather than later. Field testing and further analysis needs to be conducted so that design manuals can confidently be amended.

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APPENDIX A – Project Specification

NG4111/4112 Research Project

Project Specification

For: Adam Pease

Title: The Impacts Construction Traffic has on Pavements in Residential Subdivisions

Major: Civil Engineering

Supervisor: Soma Somasundaraswaran

Enrolment: ENG4111 – EXT S1 2016
ENG4112 – EXT S2 2016

Project Aim: To investigate the differences between the theoretical pavement life and in-situ pavements located in residential streets, establish the major factors affecting the variations in pavement life and techniques to improve the pavement design process.

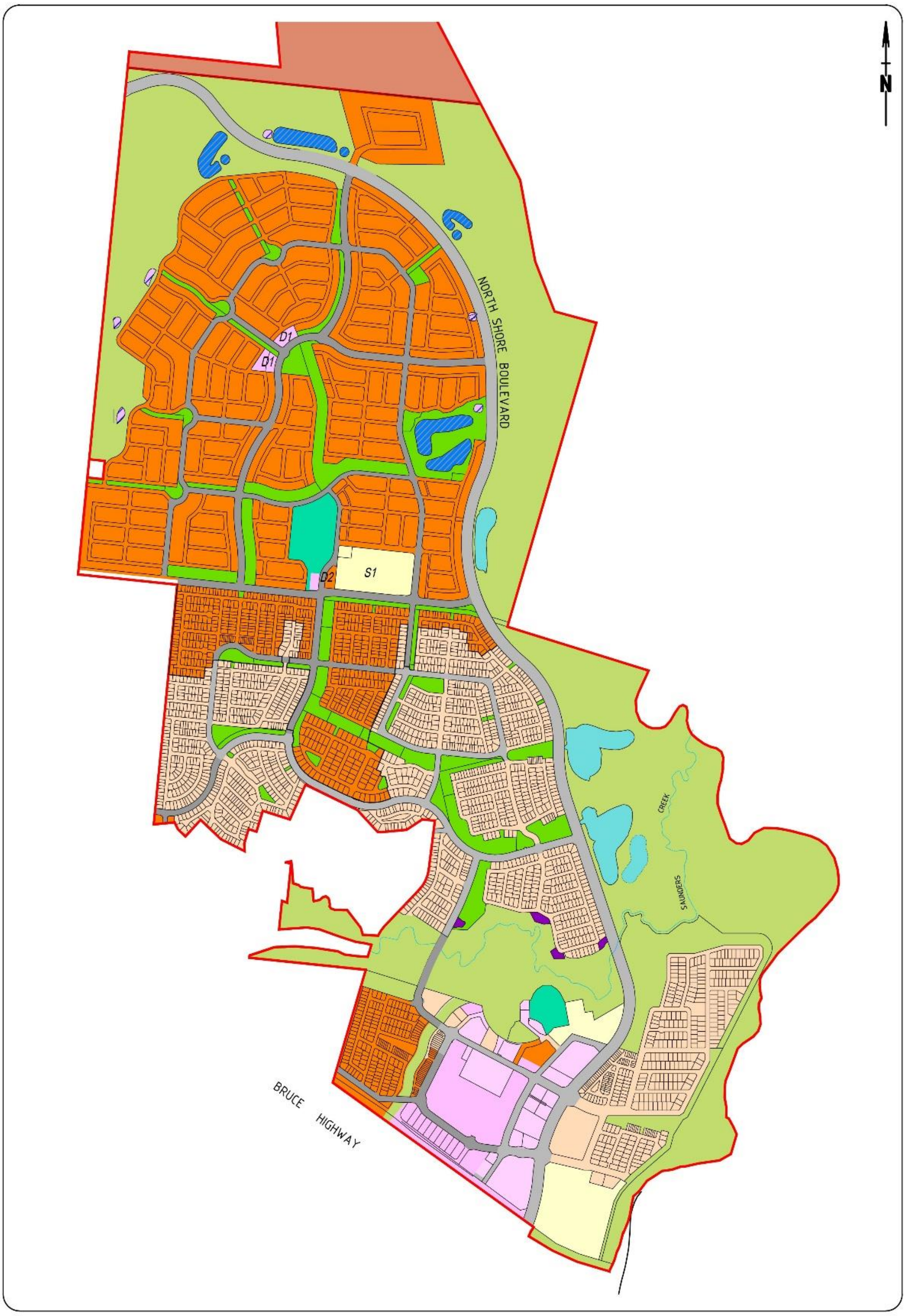
Programme: Issue A, 16th March 2016

1. Research background information relating to the design of road pavements with respect to the design equivalent standard axles and percentage of heavy vehicles.
2. Gather as-constructed information from numerous residential subdivision stages and analyse the data to establish appropriate and comparable stages for non-destructive testing.
3. Establish appropriate test locations within the chosen residential stages and to conduct enough non-destructive tests to ensure a good spread of data is achieved.
4. Convert the raw data extracted from the non-destructive testing into a format in which the amount of life remaining in the pavement can be calculated.
5. Analyse theoretical versus actual design equivalent standard axles and establishes whether the residential pavement has exceeded or failed to meet the theoretical design calculations.
6. Establish an alternative design technique that accounts for construction traffic and enables the residential pavement to reach its intended design life.

If time and resources permits:

7. Repeat non-destructive testing after construction traffic has impacted the new residential pavement and analyse new results with base line data.

APPENDIX B – Site Selection Analysis


N
NORTH SHORE BOULEVARD
D1
D1
D2
S1
SAUNDERS CREEK
BRUCE HIGHWAY

Stage Number	Number of Lots	Placed On Maintenance	Years in Service	Road Classification		Adopted Design CBR's	Access Street DESA	Cement Treated	Visual Inspection	Applicable for Testing	Comments
				Access Streets	Minor Collectors						
502	48	4/05/2010	6.05	6	1	2.0 - 6.0	5.86E+03	No		Yes	Possible stage for testing
503	26	24/05/2011	4.99	2	0	1.5 & 2.0	5.86E+03	No		Yes	Possible stage for testing
504	27	28/05/2010	5.98	1	0	2.0 - 3.0	5.86E+03	No		Yes	Possible stage for testing
505	42	3/08/2010	5.80	2	0	3.5 & 4.0	5.86E+03	No		Yes	Possible stage for testing
506	41	14/06/2011	4.94	1	0	2.0	5.86E+03	No		Yes	Possible stage for testing
507	40	21/07/2011	4.84	3	0	1.5 & 2.0	5.86E+03	No	Yes	Yes	Possible stage for testing
508	38	22/06/2011	4.92	1	1	2.0 & 2.5	5.86E+03	No	Yes	Yes	Possible stage for testing
509	35	13/10/2010	5.61	2	1	4.0	5.86E+03	No	Yes	Yes	Possible stage for testing
510	20	2/08/2010	5.81	3	1	-	-	-	No	No	Access Streets within stage are very short and not ideal for testing
511	35	3/08/2010	5.80	2	0	2.0 & 3.5	5.86E+03	No	Yes	Yes	Possible stage for testing
512	35	14/09/2010	5.69	3	0	1.0 - 3.0	5.86E+03	No	Yes	Yes	Possible stage for testing
513	22	14/03/2010	6.19	3	0	1.5 & 2.5	5.86E+03	No	Yes	Yes	Possible stage for testing
514	28	4/10/2010	5.63	1	0	1.5 & 3.0	5.86E+03	Yes	No	No	Subsoil Drainage failure and Pavement was re-constructed with a cement treatment
515	32	25/10/2011	4.58	2	0	2.5 & 3.0	5.86E+03	No	Yes	Yes	Possible stage for testing
516	26	21/12/2011	4.42	2	1	1.0 & 1.5	5.86E+03	Yes		Yes	Possible stage for testing

517	23	21/12/2011	4.42	2	1	1.0 & 1.5	5.86E+03	No	Yes	Yes	Possible stage for testing
518	26	3/10/2012	3.64	0	1	-	-	-	No	No	No Access Streets within stage
519	30	7/07/2012	3.88	1	1	2.5	5.86E+03	No		Yes	Possible stage for testing
520	24	23/10/2012	3.58	2	1	1.0	6.00E+04	Yes		Yes	Possible stage for testing
521	23	22/02/2013	3.25	1	1	1.0	6.00E+04	Yes		Yes	Possible stage for testing
522	40	17/12/2013	2.43	4	1	2.0	6.00E+04	Yes		No	Access Streets within stage are very short and not ideal for testing
523	28	12/05/2014	2.03	3	0	4.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
524	24	2/09/2015	0.72	2	0	3.5	6.00E+04	Yes		Yes	Possible stage for testing
527	0	13/01/2015	1.36	0	1	-	-	-	No	No	No Access Streets within stage
528	26	17/07/2014	1.85	4	1	-	-	-	No	No	Access Streets within stage are very short and not ideal for testing
529	14	10/10/2014	1.62	1	0	2.5	6.00E+04	Yes		Yes	Possible stage for testing
530	19	18/09/2014	1.68	2	0	1.5 & 2.0	6.00E+04	Yes		Yes	Possible stage for testing
531	8	12/05/2014	2.03	0	1	-	-	-	No	No	No Access Streets within stage
539	29	17/07/2012	3.85	2	0	1.5	2.86E+03	No		No	Single loaded roads
540	13	18/09/2014	1.68	2	0	3.0	6.00E+04	Yes	No	No	Premium area with slow dwelling construction
541	14	20/02/2015	1.26	3	0	2.0	6.00E+04	Yes	No	No	Premium area with slow dwelling construction
548	37	18/10/2012	3.59	3	1	3.0	5.86E+03	No		Yes	Possible stage for testing
549	25	18/06/2012	3.93	2	0	3.0	5.86E+03	No		Yes	Possible stage for testing
550	25	18/06/2012	3.93	2	0	3.0	5.86E+03	No		Yes	Possible stage for testing
551	31	19/12/2012	3.43	4	1	1.5	6.00E+04	Yes	No	No	Access Streets within stage are very short and not ideal for testing
552	21	20/11/2012	3.51	2	0	1.0	6.00E+04	Yes		Yes	Possible stage for testing
553	29	20/11/2012	3.51	2	0	1.0	6.00E+04	Yes		Yes	Possible stage for testing
554	11	19/12/2012	3.43	1	0	2.0	6.00E+04	Yes		Yes	Possible stage for testing
555	20	11/12/2013	2.45	0	1	2.5	1.00E+06	Yes	No	No	No Access Streets within stage
556	12	11/12/2013	2.45	1	0	4.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing

557	9	17/06/2013	2.93	1	2	2.5	6.00E+04	Yes	No	No	Access Streets within stage are very short and not ideal for testing
558	18	11/12/2013	2.45	1	0	4.0	6.00E+04	Yes	No	No	Nonstandard staging
559	11	13/05/2013	3.03	0	1	3.5	1.00E+06	Yes	No	No	No Access Streets within stage
560	29	16/05/2014	2.02	5	0	3.5	6.00E+04	Yes	Yes	Yes	Possible stage for testing
561	35	16/05/2014	2.02	1	0	3.5	6.00E+04	Yes	Yes	Yes	Possible stage for testing
562	20	30/06/2014	1.89	2	0	8.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
563	30	21/07/2015	0.84	3	1	5.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
564	43	21/07/2015	0.84	2	1	3.0	7.90E+04	Yes	Yes	Yes	Possible stage for testing
566	26	25/11/2014	1.49	1	1	8.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
567	19	25/11/2014	1.49	1	0	7.0	6.00E+04	Yes	Yes	Yes	Possible stage for testing
570	20	22/05/2016	0.00	3	1	5.0	1.20E+05	Yes	Yes	Yes	Possible stage for testing
581	18	20/05/2013	3.01	1	1	2.5	6.00E+04	Yes	Yes	Yes	Possible stage for testing
582	19	20/05/2013	3.01	2	1	10.0	6.00E+04	Yes	Yes	No	Possible stage for testing
583	14	11/12/2013	2.45	1	0	4.0	6.00E+04	Yes	Yes	No	Possible stage for testing
584	16	11/12/2013	2.45	1	0	4.0	6.00E+04	Yes	Yes	No	Possible stage for testing
585	10	31/05/2013	2.98	0	0	-	-	-	No	No	No Roadworks within stage
591	5	12/11/2014	1.53	0	1	-	-	-	No	No	No Access Streets within stage
592	7	13/01/2015	1.36	0	1	-	-	-	No	No	No Access Streets within stage

APPENDIX C – Pavement Design Calculations

Road Name		Trial Road - Current Method					
N_{DT}	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF					
AADT	=	H x 10 vehicles per day					
		H	=	No. Houses	=	40	each
AADT	=	400					
DF	=	1.00					
%HV	=	4	%				
N_{HVAG}	=	2.1					
LDF	=	1.0					
CGF	=	$\frac{(1 + 0.01 \times R)^P - 1}{0.01 \times R}$					
	where	R	=	Annual Growth Rate	=	1.0	%
		P	=	Design Period	=	20	years
CGF	=	22					
N_{DT}	=	2.70E+05					

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.3
DESA	=	8.10E+04				

Road Name		Trial Road - Alternative Method				
N_{DT}	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF				
AADT	=	H x 5 vehicles per day				
		H	=	No. Houses	= 40	each
AADT	=	**200**				
DF	=	1.00				
%HV	=	100	%			
N_{HVAG}	=	2.3				
LDF	=	1.0				
CGF	=	$\frac{(1 + 0.01 \times R)^{P} - 1}{0.01 \times R}$				
	where	R	=	Annual Growth Rate	= 0.0	%
		P	=	Design Period	= 1	years
CGF	=	**1.0**				
N_{DT}	=	**1.68E+05**				

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	**1.01E+05**				
DESA	=	**8.10E+04**		Design ESA as calculated with Current methods		
Total DESA	=	**1.82E+05**				

Road Name		Saba Street - Alternative Method					
$\mathbf{N_{DT}}$	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF					
AADT	=	H x 5 vehicles per day					
		H	=	No. Houses	=	16	each
AADT	=	80					
DF	=	1.00					
%HV	=	100	%				
N_{HVAG}	=	2.3					
LDF	=	1.0					
CGF	=	$\dfrac{(1 + 0.01 \times R)^{P} - 1}{0.01 \times R}$					
	where	R	=	Annual Growth Rate	=	0.0	%
		P	=	Design Period	=	1	years
CGF	=	1.0					
$\mathbf{N_{DT}}$	=	6.72E+04					

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	4.03E+04				
DESA	=	5.86E+03		Design ESA as calculated with Current methods		
Total DESA	=	4.62E+04				

Road Name		Oculina Street - Alternative Method					
N_{DT}	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF					
AADT	=	H x 5 vehicles per day					
		H	=	No. Houses	=	22	each
AADT	=	110					
DF	=	1.00					
%HV	=	100	%				
N_{HVAG}	=	2.3					
LDF	=	1.0					
CGF	=	$\frac{(1 + 0.01 \times R)^P - 1}{0.01 \times R}$					
	where	R	=	Annual Growth Rate	=	0.0	%
		P	=	Design Period	=	1	years
CGF	=	1.0					
N_{DT}	=	9.23E+04					

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	5.54E+04				
DESA	=	5.86E+03		Design ESA as calculated with Current methods		
Total DESA	=	6.13E+04				

Road Name		Laysan Street - Alternative Method				
$\mathbf{N_{DT}}$	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF				
AADT	=	H x 5 vehicles per day				
		H	=	No. Houses	= 16	each
AADT	=	80				
DF	=	1.00				
%HV	=	100	%			
N_{HVAG}	=	1.0				
LDF	=	1.0				
CGF	=	$\dfrac{(1 + 0.01 \times R)^{P} - 1}{0.01 \times R}$				
	where	R	=	Annual Growth Rate	= 0.0	%
		P	=	Design Period	= 1	years
CGF	=	1.0				
$\mathbf{N_{DT}}$	=	6.72E+04				

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	4.03E+04				
DESA	=	5.86E+03		Design ESA as calculated with Current methods		
Total DESA	=	4.62E+04				

Road Name		Yanuca Street - Alternative Method					
N_{DT}	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF					
AADT	=	H x 5 vehicles per day					
		H	=	No. Houses	=	23	each
AADT	=	115					
DF	=	1.00					
%HV	=	100	%				
N_{HVAG}	=	2.3					
LDF	=	1.0					
CGF	=	$\frac{(1 + 0.01 \times R)^P - 1}{0.01 \times R}$					
	where	R	=	Annual Growth Rate	=	0.0	%
		P	=	Design Period	=	1	years
CGF	=	1.0					
N_{DT}	=	9.65E+04					

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	5.79E+04				
DESA	=	6.00E+04		Design ESA as calculated with Current methods		
Total DESA	=	1.18E+05				

Road Name		Columbus Street - Alternative Method					
N_{DT}	=	365 x AADT x DF x (%HV / 100) x NHVAG x LDF x CGF					
AADT	=	H x 5 vehicles per day					
		H	=	No. Houses	=	20	each
AADT	=	100					
DF	=	1.00					
%HV	=	100	%				
N_{HVAG}	=	2.3					
LDF	=	1.0					
CGF	=	$\frac{(1 + 0.01 \times R)^P - 1}{0.01 \times R}$					
	where	R	=	Annual Growth Rate	=	0.0	%
		P	=	Design Period	=	1	years
CGF	=	1.0					
N_{DT}	=	8.39E+04					

Design Equivalent Standard Axles (DESA)

DESA	=	(ESA / HVAG) x N_{DT}				
	where	ESA / HVAG	=	Damage Index	=	0.6
DESA	=	5.04E+04				
DESA	=	1.20E+05		Design ESA as calculated with Current methods		
Total DESA	=	1.70E+05				

GUIDE TO PAVEMENT TECHNOLOGY PART 2: PAVEMENT STRUCTURAL DESIGN

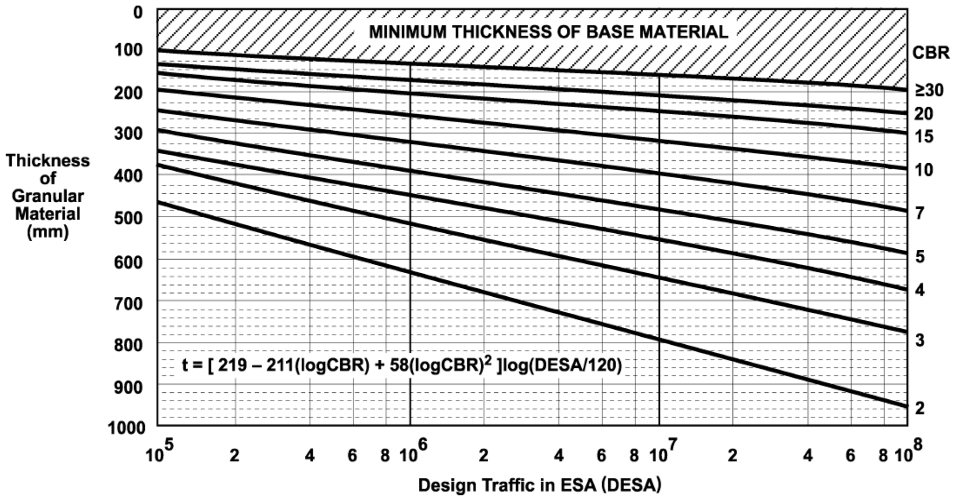
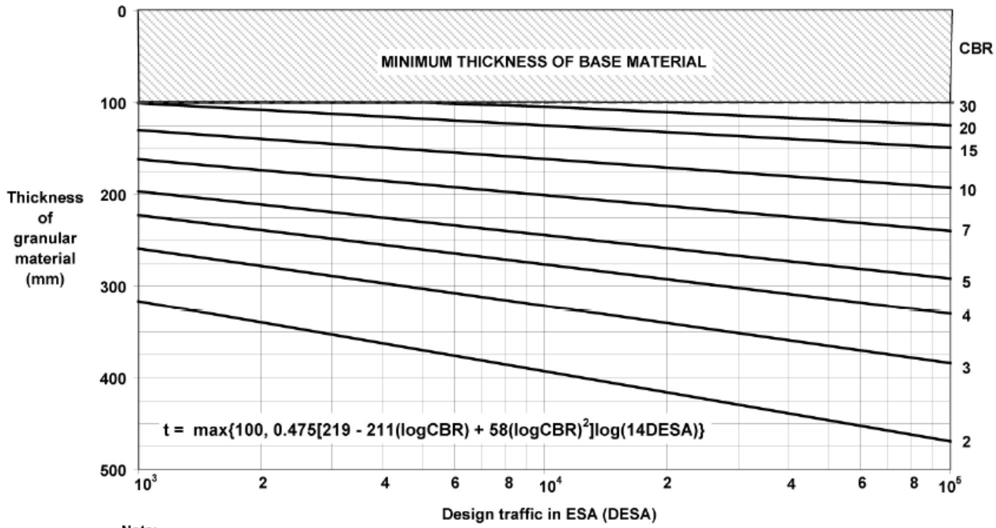


Figure 8.4: Design chart for granular pavements with thin bituminous surfacing

GUIDE TO PAVEMENT TECHNOLOGY PART 2: PAVEMENT STRUCTURAL DESIGN



Note:
1. Appropriate local conditions, environmental and drainage issues must be considered in using these design curves.
2. Thin asphalt surfacings may be included in total granular thickness. However, the minimum thickness of the granular base is 100 mm.

Figure 12.2: Example design chart for lightly-trafficked granular pavements with thin bituminous surfacings

Austroads 2010 Fig 12.2	
Light Traffic < 1E5 ESAs	
Trial Pavement (Current Method)	
CBR =	3.0
DESA =	8.10E+04
T min. =	378.27
T adopted =	**380**

Austroads 2010 Fig 8.4	
Heavy Traffic > 1E5 ESAs	
Trial Pavement (Alternative Method)	
CBR =	3.0
DESA =	1.82E+05
T min. =	418.39
T adopted =	**420**

Austroads 2010 Fig 12.2	
Light Traffic < 1E5 ESAs	
Saba Street (Alternative Method)	
CBR =	3.0
DESA =	4.62E+04
T min. =	363.04
T adopted =	**365**

Austroads 2010 Fig 12.2	
Light Traffic < 1E5 ESAs	
Oculina Street (Alternative Method)	
CBR =	3.0
DESA =	6.13E+04
T min. =	370.71
T adopted =	**375**

Austroads 2010 Fig 12.2	
Light Traffic < 1E5 ESAs	
Laysan Street (Alternative Method)	
CBR =	3.0
DESA =	4.62E+04
T min. =	363.04
T adopted =	**365**

Austroads 2010 Fig 8.4	
Heavy Traffic > 1E5 ESAs	
Trial Pavement (Alternative Method)	
CBR =	4.0
DESA =	1.18E+05
T min. =	338.14
T adopted =	**340**

Austroads 2010 Fig 8.4	
Heavy Traffic > 1E5 ESAs	
Trial Pavement (Alternative Method)	
CBR =	5.0
DESA =	1.70E+05
T min. =	314.67
T adopted =	**315**