University of Southern Queensland

Faculty of Engineering and Surveying

The Effect of Compaction on the Design Life of Rehabilitated Insitu Cement Powder Stabilised Pavements.

A dissertation submitted by

Mark Geoffrey Weatherley
BETech StudIEAust

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ABSTRACT

This project involves conducting visual inspections and Falling Weight Deflectometer (FWD) tests on a number of road pavements which have been reconstructed over the last eighteen months by cement powder insitu stabilisation. The roads are within the area serviced by the Mackay District of the Department of Main Roads, Queensland.

The results of these tests are compared with the compaction dry density test results, taken for quality control purposes at the time of construction, to ascertain whether there is a correlation between the two values and whether failure to meet the specified requirement of 100 percent standard compaction affects the “cured” pavement strength.

The results indicate that there is no correlation between the field dry density and the modulus of the pavement found by the FWD tests. While plots of modulus and Relative Dry Density (RDD) suggest a similarity where the higher field density results often correspond to high modulus values, many of the comparisons exhibit the opposite behaviour.

The investigation identifies that the modulus valves used in the design of pavements often appear to be relatively conservative with some of the tests achieving modulus values up to twenty times the targeted value. Of the 21 lots investigated only 2 lots passed the requirement of 100% standard compaction, however, using the same statistical analysis method on the moduli values, 19 of the 21 lots passed. Failure to meet the specified 100% RDD requirement does not mean that the required strength has not been obtained.

It is concluded that the current processes for the design and construction of cement powder insitu-stabilisation are providing satisfactory results, however there appears to be a need for more controlled investigations into obtaining the design data and forecasting the resulting modulus of the stabilised layer after treatment.

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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 or 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.

Mark Geoffrey Weatherley

Student Number: Q9723871X

Signature: ....

Date: ....

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ABBREVIATIONS

AADT	Average annual daily traffic (vehicles per day)
ARDD	Average Relative Dry Density %
AS	Australian Standard Specification
AustStab	Australian Stabilisation Industry Association
AustRoads	Association of Australian and New Zealand Road Transport and Traffic Authorities
CBR	Californian Bearing Ratio
CIRCDEF	An analysis engine based on CIRCLY to interpret deflections measured by the FWD test.
CIRCLY	A computer program developed by CSIRO for linear elastic isotropic and non-isotropic multi-layer stress analysis.
CSIRO	Commonwealth Scientific and Industrial Research Organisation
CV	Characteristic Value (a) for RDD: ARDD – (SD x F_{CV}) (b) for FWD: (Mean / SD) x 100
DCP	Dynamic Cone Penetrometer
DF	Direction Factor
ESA	Equivalent Standard Axles
F_{CV}	Statistical Factor for the determination of Characteristic Value, depending on the number of samples – refer MRS 11.01.
FWD	Falling Weight Deflectometer
GB	General Purpose Blended Cement (AS 3972 - 1997)
GP	General Purpose Portland Cement (AS 3972 - 1997)
hp	Horsepower
LL	Liquid Limit

Mean	Arithmetic mean of a list of values
MDD	Maximum Dry Density
MR	Main Roads
MRD	Queensland Government Department of Main Roads
MRS	Main Roads Standard Specification
MUTCD	Manual of Uniform Traffic Control Devices
NAASRA	National Association of Australian State Road Authorities
PI	Plasticity Index
PL	Plastic Limit
RDD	Relative Dry Density
SAR	Standard Axle Repetitions
SD	Standard Deviation
SL	Shrinkage Limit
UCS	Unconfined Compressive Strength
vpd	Vehicles per day
WIM	Weigh-in-Motion traffic station

CHAPTER 1 – INTRODUCTION

1.1 Outline

This project analyses field compaction dry density test results for nine insitu-stabilised road pavement reconstruction works carried out in the Mackay District of the Department of Main Roads, Queensland, during the period January 2005 – June 2007.

Visual assessments of the condition of the pavements were made to assess whether the pavements showed any initial signs of failure, although the in-service time was much shorter than the design service life and consequently the applied axle loading to date was well below the design loading.

Falling Weight Deflectometer (FWD) testing were also carried out to obtain the modulus at each “site specific” test location where as-constructed compaction density tests were carried out during construction. These “cured” moduli values are compared with the compaction density results to determine whether there is a correlation between the two values and whether failure to meet the specified requirement of 100 percent standard compaction affects the “cured” pavement strength.

Additionally, sequential FWD testing was also carried out at regular intervals (every 50 or 100 metres) in the outer wheel track on both sides of the road and the results analysed to provide a measure of the standard of the pavement and its remaining service life.

Following on from these results, the moduli were reviewed to assess whether the moduli assumed in design could have been increased with a consequent improvement in the forecast design life. If so, it may be possible to extend the use of the less expensive insitu stabilisation process to situations where this method might not have appeared to economically produce a satisfactory design life.

1.2 Objective

The objectives of this project are:

- to assess and record the visible performance of a sample of pavements rehabilitated by in-situ stabilisation over the previous eighteen months;
- to determine a correlation (if any) between construction compaction density tests and “cured” moduli;

- to determine whether failure to meet the specified 100% compaction during construction has a deleterious effect on the service life of the pavement and whether the reduced service life can be estimated from the results;
- to recommend whether a reduction in the compaction requirements of the specification should be considered based on the analysis of the results, or alternatively whether the construction process for insitu stabilisation should be modified to ensure that the 100 precent standard compaction is achieved more consistently; and
- to determine whether the design modulus calculated and assumed for the insitu stabilised pavement material reflects actual “cured” results and to recommend whether the value of the modulus should be updated to reflect actual results.

1.3 Background

RoadTek Mackay is the construction arm of the Queensland Government’s Department of Main Roads in the Mackay District, and undertakes approximately ten road rehabilitation projects each year involving the in-situ stabilisation of pavement material with general blend cement. Each project is subdivided into half-road width lots of approximately 700 - 1000 metres. The annual budget for these projects is approximately \$5.4 million and accounts for approximately 35% of the infrastructure construction/reconstruction carried out by RoadTek in the District.

Cement stabilisation requires that compaction be completed within a specified time after the introduction of the cement powder, so the size of each “lot” rehabilitated at one time is determined by the available machinery. Normally, mixing and compaction must be completed within four hours. Once the compaction is completed, soil compaction density tests using either the sand replacements (MR Test Method - Q111A) or nuclear gauges (MR Test Method - Q112) are carried out for each lot. Tests are generally taken at a rate of 1 per 1000 m^2 with a minimum of 3 per lot, or 1 per 800 m^2 with a minimum of 5 per lot. These results are then compared with the density of a reference sample which has been compacted in the laboratory to its maximum dry density (MR Test Method - Q110A or Q110F) and the Relative Dry Density ratio (RDD) determined. The standard construction specification requires that, for each lot, the Characteristic Value (CV) of the RDD (a statistical average of the several RDD test results) attain

100 percent compaction. If 100 percent compaction is not achieved, the service life of the pavement is considered to be less than required. Although not strictly permitted, the whole lot can be re-stabilised and re-compacted in an attempt to achieve specified compaction, which obviously increases the construction cost.

In a significant number of projects the standard stabilisation process did not produce compaction results which met specification. The standard specification (MRS 11.07) provides for a reduced level of payment to compensate for the reduced level of service inferred because of failure to meet compaction specifications. On average, the typical reduction in payment for the reduced level of service is approximately $14,000 per project, ie an estimated $150,000 annually. The alternative of re-working a lot is generally more costly than accepting the reduced level of service payment.

There is anecdotal evidence that despite not meeting specification there is no appreciable degradation of service for compactions above about 93% standard compaction and the expense of meeting specification is unnecessary. If so, it may be appropriate to relax the specification requirement with no detriment to the pavement performance.

This project was designed to investigate whether there is any factual basis for the anecdotal inferences about in-service performance and if so, to recommend changes to the requirements of the MRD standard specification, or alternatively, to recommend changes to the standard procedure for in-situ stabilisation to ensure the compaction standard is met.

It has been suggested that this problem is not unique to Mackay and it is possible that the results of this study may be applicable on a state wide basis.

There are a number of possible reasons why the compaction test may fail to achieve 100% RDD:

- the material within the project is not uniform and homogeneous and may react differently from the material sampled for the pre-construction pavement investigation;
- poor subgrade materials over which the pavement is supported can cause inconsistencies in the compaction of the pavement layer;

- the most appropriate compaction equipment may not be available outside the limits of the major centres (such as South East Queensland); and
- poor workmanship and poor knowledge of the construction process for the stabilisation of different materials may produce inconsistence results.

For design purposes, a modulus in the order of 600 MPa is often targeted for rehabilitation work where only a small percentage of grade-correcting gravel is added to the existing pavement material. Where the existing material is of a higher strength, or a significant amount of high strength material is added, the target modulus may be in the region of 1000 – 2000 MPa. Ad-hoc Falling Weight Deflectometer testing around the Mackay District on insitu stabilised pavements which have had small percentages of cement powder added (0.5 - 2.5% by mass) have shown moduli well in excess of that assumed for design, often exceeding 1000 - 1500 MPa and sometimes into or above the Category 2 level 2000 – 5000 MPa.

Approximately 18 months ago, the Department's Materials Testing branch in Mackay identified this issue and implemented a more rigorous system of documenting the field and laboratory test results for every project involving in-situ stabilisation. Documentation for a total of 9 projects is available for analysis and are summarised in Appendix D. These results have been reviewed against the design documents and 23 lots have been selected for further analysis, a total of 89 test locations, as detailed in Chapter 6.

Arrangements were made for the MRD’s Falling Weight Deflectometer team to test at these locations and using a computer program based on the CIRCLY pavement design program, estimates of the moduli of each pavement layer were obtained.

By the very nature of the process, the material properties for an insitu stabilised pavement are likely to be show more variation over the extent of the work, compared with a new construction where the properties of all layers are more controlled. Hence it may be appropriate to repeat the analysis with data from other districts to verify the findings over a greater number of sites and test locations.

To carry out insitu stabilisation rehabilitation work, the Mackay District has available a 350 hp Stabiliser capable of mixing a layer not greater than 300 mm thick, hence designs are limited to this thickness. Multi-layer construction is not normally carried out as bonding problems are experienced at the interfaces.

The six projects selected for analysis include two sections on the Peak Downs Highway, one section on the Fitzroy Development Road, one section on the Dysart-Middlemount Road, one section on the Sarina-Homebush Road and one section on the Marian-Eton Road. Traffic volumes range from approximately 500 to 2000 vehicles per day per lane. The more heavily trafficked roads carry a significant volume of coal-mine related heavy vehicle traffic.

CHAPTER 2 – ROAD PAVEMENT CONSTRUCTION

Most of the rural roads constructed in the Mackay District of the Queensland Department of Main Roads (MRD) over the past twenty years have been designed and constructed using unbound pavement material. Unbound pavement material refers to mixtures of crushed rock, fine clays and similar material combined in such a way that, when properly compacted, minimal air voids are present. The strength of the material for transmitting traffic loadings is attained basically from mechanical friction and mechanical interlock of the particles.

These pavements would have been designed using various empirical methods which have been developed from Australian and overseas experience and knowledge of the performance of previously constructed pavements. The most common empirical method currently used by Australian road authorities is described in more detail in Chapter 3.

As the older pavements reach the end of their useful service life, the approach taken by many authorities and in particular by the rural districts of the MRD is, where possible, to rejuvenate the existing pavement through the use of cement insitu stabilisation rather than reconstructing a new pavement. The suitability of the existing pavement material will determine whether insitu stabilisation is appropriate, as well as other factors such as the need to improve the vertical or horizontal alignment.

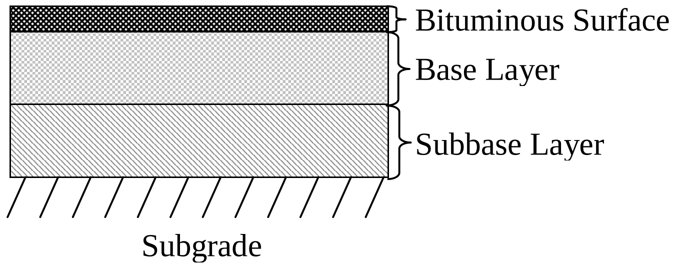
The cement insitu stabilisation process can, if the properties are appropriate, reduce moisture susceptibility and improve the interparticle bonds in granular materials giving the stabilised material a useful tensile strength and higher elastic modulus. It has been shown in the past that a pavement rehabilitated with cement can achieve more than 80% of a newly constructed pavement life at a considerable cost saving.

This chapter provides a brief overview of the construction of flexible pavements, the history and theory of the cement stabilisation process, the construction of insitu cement stabilised pavements, and the testing carried out to verify the quality of the construction process.

2.1 What is a pavement

The natural soil on which a road is to be constructed is often not strong enough to support the repeated application of even relatively light wheel loads without significant deformation. It is therefore necessary to cushion the natural soil by the use of a structure capable of bearing the applied loads and distributing them over the natural soil to prevent excessive deformations (Municipal Services Study Book 2000, p. 4.1). This structure is called a pavement. Figure 2-1 displays the composition of a pavement.

Figure 2-1 - Typical Pavement



The subgrade is the base of the construction and is typically the existing soil. The main purpose of the overlying layers is to distribute the traffic load so the subgrade can support the loads without damage.

The base and subbase are the main load-bearing layers of a pavement. The materials used to construct the base and subbase are typically made up of crushed rock of various sizes up to 19 mm interspersed with finer rock and fine clay material.

The bituminous surfacing provides a seal to minimise the amount of water infiltrating the pavement and contains bound rock aggregate which provides the wearing surface to resist the wear of the traffic and prevent the bitumen being worn away. The pavements that are the subject of this report have been surfaced by a conventional bituminous aggregate mix, typically a seal layer with 7 mm aggregate followed by a wearing layer with 16 mm aggregate.

The Austroads *Guide to the Structural Design of Road Pavements* (Austroads 2004) divides pavements into three groups - flexible pavements, consisting solely of unbound pavement materials; flexible pavements that contains one or more bound layers; and rigid pavements.

Rigid pavements consist of layers of plain or reinforced concrete constructed on top of the subgrade and are not considered further.

The original pavements, prior to being insitu stabilised as the subject of this project, are classified as unbound flexible pavements, whereas the rehabilitated insitu stabilised pavements are classified as bound flexible (modified) pavements.

Bound flexible pavements having small quantities of binders such as cement, bitumen, polymers and other similar additives have come to the forefront in recent years in response to the increasing demands placed on the performance of the pavement with increasing traffic intensity and loading. They are constructed from natural manufactured material with a small percentage of the binding material added, typically 1% to 4% of the additive. Although still classified as flexible pavements their failure mechanisms are complex and design of these pavements requires detailed analysis rather than the empirical approach which can be used for unbound pavements.

2.2 In-situ Cement Stabilisation

In-situ cement stabilisation is a construction process that mixes a predetermined portion of cement or a blend of cementitious materials (such as cement, flyash and blast furnace slag) with existing materials to achieve:

- a reduction in moisture susceptibility, resulting in improved volume and strength stability under variable moisture conditions.
- the development of inter-particle bonds in granular materials, giving the stabilised material a useful tensile strength and higher elastic modulus.

2.2.1 History of Stabilisation

The first recorded modern use of insitu stabilisation was in 1944 by the UK Ministry of Transport (Williams 1986). The first specialised contractor, *Stabilisers Limited*, entered the Australian market in 1952, with the P&H triple rotor stabiliser. The process was continually used during the 1960's, however as more contractors entered the market, competition became fierce and work started to be carried out by cheaper machines leading to poor quality mixing, at lower prices and with less attention to quality. This led to unacceptable pavements of inadequately mixed materials with localised failures appearing during the service life of the pavement. This poor performance led to a number of companies closing (Wilmot 1996). Road authorities moved away from insitu-stabilisation due to the loss of confidence in this method in the late 1960's.

The 1970's saw a resurgence in the use of stabilisation in Victoria and New South Wales, which soon spread to the other states. The process was then being performed in a more controlled manner with improved construction success. In 1976, many articles on completed cement stabilisation projects were seen in technical publications. The P&H triple-rotor machine was replaced by the single rotor stabiliser in the late 1970s, which is still in use today.

Until recently, the major restriction on pavement stabilisation was the depth to which the road pulveriser and compaction equipment could operate effectively, usually about 250 mm compacted depth. However, in 1992 the CMI RS 500 deep-lift stabilisation equipment, capable of stabilising a layer up to 400 mm in depth, became available in Australia (Vorobieff 1998a). To achieve these greater depths it was apparent that more research was required into the cement binder products to delay the set time to allow for full compaction. Hence, blended binders consisting of cement with other waste products such as slag and fly ash which have properties that delay the hydration process were developed (Wilmot 1996). Together with the development of accurate cement spreading equipment, these new capabilities have led to the extension of stabilisation to roads ranging from local government low-traffic roads through to major roads and highway construction.

2.2.2 Rationale and Benefits

When a flexible pavement is nearing the end of its service life it shows signs of distress as a loss of structural capacity or a deterioration in ride quality. Methods of treatment of deteriorating pavements are:

- Reconstruction - completely rebuild the road with new materials which involves a large initial cost, but potentially low ongoing maintenance costs equivalent to a new pavement;
- Overlay Failing Pavement - overlay the existing pavement with a new 100 mm base layer of high quality pavement material. This has a lower cost of reconstruction initially, but high future maintenance costs.

- Recycle/Rehabilitate Existing Pavement with Cement Powder - Typically incorporate a cement powder mix ranging from 1 to 4% by mass into the top 150 - 250 mm of the pavement, re-compact and seal.

The advantages of recycling the original pavement are as follows.

- Insitu stabilised pavements are less expensive to construct than a full reconstruction. A saving of up to 40% can be achieved with a service life typically exceeding 80% of that of a traditionally reconstructed pavement (Hodgkinson. G.F. 1991).
- Because the depth of disturbance is restricted to the existing pavement vertical alignment there will be minimal interference with existing kerb, drainage and underground service levels .
- The time limit on cement binder workability calls for sections to be sized in daily manageable portions. Therefore, a section of road is not normally closed to traffic overnight (reopened at end of working day).
- The construction process usually requires very little change to the existing vertical alignment, therefore with care and under the direction of the stabilisation crew, access to adjacent properties can be given through the work site with only temporary discomfort.
- There is very little, if any, material needed to be carted to or removed from site other than small quantities to correct or improve surface crossfall. The only new material is the cement powder.
- The recycling of pavement material reduces the amount of quarry material used, directly extending the life of quarry sources, and thus reducing the need to develop new quarry sites with the associated costs and environmental harm.
- Recycling reduces the amount of cartage required in transporting material to site, contributing to a reduction in atmospheric pollution from the heavy vehicle emissions, as well as reducing fuel requirements.
- Less material transport reduces the damage caused to existing adjoining pavements along the haul route to the project site (Smith & Vorobieff 2007).

- Recycling also reduces the requirements for storage and disposal of the excess material produced by reconstruction. Existing excess pavement material and unsuitable material must be removed, temporarily stored then dumped. Where possible, some of this material is used as embankment material for widening the new road reconstruction. However, in many situations there is an excess of material at the completion of construction, with the attendant risks of erosion and sediment problems.

2.2.3 Theory of Cement Stabilisation

To achieve the most desirable results with cement stabilisation the cementitious binder and pavement material is to be intimately mixed and then water added. The primary hydration process begins immediately between the cementitious binder and the water in the soil forming calcium silicate and aluminium hydrates. This reaction occurs independently of the nature of the soil.

A secondary hydration reaction also occurs releasing hydrated lime which will react with any pozzolans within the soil. Similar by-products to the primary reaction will be produced.

The primary reaction with the calcium silicate and aluminium hydrates will cause significant strength gains in the first day. The secondary reaction will proceed slowly but continue over a long period provided that adequate moisture is present. Reactions are also temperature sensitive, the rate of reaction increasing with the increasing temperature. Organic materials and sulphates may cause retardation of the reaction.

2.2.4 Correction Course and Grade Correction

Pavements that require rehabilitation have usually lost shape due to rutting and shoving, so shape correction is often required to recover the profile and superelevation. Hence it is common to apply a correction course before mixing. The common depth averages between 50 and 75 mm. Gravel designated Type 2 (MRS 11.05) by the MRD, as described below, is commonly used in Queensland.

Well used pavements also exhibit a loss of strength and grading of the material due to wear and crushing over time. In these cases, a grade correction layer may be required to improve the grading and the structural strength of the insitu material. A layer of up to

100 mm of Type 2 granular material maybe used and incorporated into the existing pavement when pulverising and mixing.

Type 2 C Grade granular material is commonly used by MRD in North Queensland. This material has been developed for a range of traffic ESA loadings up to 10^7 and includes up to approximately 45% of rock between 9.5 mm and 37.5 mm, replacing the lost coarse material. It was specially developed for use in wet environments.

The CBR values for Type 2 material can vary in the range of 20 - 80%. The commonest subtypes are 2.1 and 2.2 with CBR values approximately 80% and 60% respectively.

2.2.5 The Stabilisation Construction Process

The in-situ cement stabilisation process involves the intimate mixing of a binder and existing reclaimed pavement material, adding water, compacting and trimming, and then curing to complete the process.

A specialised recycling machine (Figures 2-2 & 2-3) is used to perform the process. The recycling machine consists of a mixing box with a rotating shaft that has teeth attached to pulverise and mix the pavement material and the binder (Morton 1993). Typically two passes are required, the first to intimately mix the binder throughout the pavement material and second to add water to achieve the optimum moisture content.

Figure 2-2 - 350 HP Stabiliser
Capable of stabilising to a depth of 250 mm.

Figure 2-3 - T.R.N. Camden Stabiliser
Capable of stabilising to a depth of 500 mm.

The quantity of cement binder is measured accurately and applied by a purpose built spreading machine (Figure 2-4), with spread rate data being stored electronically by a spreader-mounted computer (Wilmont 1993). This produces an accurate and consistent distribution of the binder over the pavement surface for the stabilising equipment to then mix throughout the pavement material.

Figure 2-4 - 14 Tonne Cement Spreader

Compaction must commence as soon as practicable after mixing. The binder has an allowable working time, typically four hours for cement powder binder. The allowable working time is usually specified in the project contract documents, commencing at the start of mixing of the binder and finishing after full compaction has been completed. The common types of rollers used are the pad foot vibratory roller (21 tonne), the smooth drum vibratory roller (21 tonne) and the multi-tyred roller.

The pad foot roller assists in the compaction of the lower portion of the pavement layer and the smooth drum is effective in compacting the upper portion. A multi-tyred roller is used to knead the surface and to close the surface pores.

Curing follows compaction and involves frequent fine spraying of the surface with water so that the surface remains visibly damp, until the bitumen seal is applied or the next layer is constructed. The surface must be sealed within seven days. Typically, a water truck would water the surface at the rate of approximately 1 litre per sq metre every 30 minutes. Sealing is normally carried out on the reclaimed sections every four days. Experience has shown that the lack of proper curing will result in surface cracking and subsequent ravelling under traffic if only a thin wearing surface is applied on top of the stabilised layer. (Austroads 2003).

2.3 Acceptance Testing

Construction of an insitu stabilised section is always chosen so that the section or lot can be completed in the one day, as there is a limited time for compaction once the cement powder has been added. Quality control and acceptance testing of the final product is done by measuring the Relative Dry Density at sample locations.

Once the pavement material has been pulverised, the binder added and the material completely mixed (before compaction), samples are taken so that the Maximum Dry Density can be determined to provide the benchmark for the quality of the construction compaction of the pavement. This test (MR Test Method - Q110A) must be completed within 45 – 65 minutes from the time the cement is incorporated, otherwise the density measured decreases and will not provide the correct reference density. (Hall 2005).

Compaction of this soil sample is carried out over a range of moisture contents, and compacted in three layers by dropping a 2.7 kg standard rammer typically 25 times from a height of 300 mm. The densities are plotted, the maximum measured and the Maximum Dry Density and the optimum moisture content recorded.

The samples are taken at random positions along each lot (distance and offset, selected in accordance with MR Test Method - Q050).

Once the road pavement has been compacted, sample tests are taken at the same positions along each lot to determine the in-field Dry Density and subsequently the

Relative Dry Density (RDD) - the ratio of the compacted density to the benchmark laboratory Maximum Dry Density.

Dry Density testing (MR Test Method - Q111A) is carried out by collecting, drying and weighing a soil sample, measuring the volume by the sand replacement method and calculating the dry density. The RDD values obtained for each test in a lot are combined to produce a Characteristic Value for the lot, as defined in Main Roads Standard Specification 11.01:

$$CV = ARDD - (SD \times F_{CV})$$

where:	CV	=	Characteristic Value for the Lot
	$ARDD$	=	Average Relative Dry Density (%)
	SD	=	Standard deviation of the sample ARDDs
	F_{CV}	=	A factor depending on the number of samples in the set determined from Table 6 of MRS 11.01

A lot is deemed to have passed if the Characteristic Value is 100% or greater. If a Characteristic Value of less than 100% is obtained, the lot may be re-worked or otherwise accepted at a reduced level of service, ie there is an assumption that the result indicates a pavement that will not carry the design traffic required and will fail before its design life.

2.4 Types of binders

There is a wide variety of cementitous binders suitable for use in the stabilisation process. The tendency is away from General Purpose Portland Cement (GP) which tends to provide only about a one hour working window to achieve compaction, towards the General Purpose Blended Cement (GB) because of the improved working time limits created by the addition of additives. This increases the length of the section of road that can be rehabilitated in the one day.

Fly Ash is the most common additive used and is a by-product of the power industry created by the burning of black coal. It is generally high in silica and alumina. In the presence of moisture and at ordinary room temperature it reacts with calcium hydroxide released by the hydration of Portland cement to form compounds possessing cementitious properties.

With the blending facilities available today there is no limit to the proportioning of the various additives and as the proportion of cement decreases the price of the blended binder reduces, although suppliers produce standard mixes such as 70% GP cement / 30% fly ash (known as 70/30). It should be appreciated that the cost will be related to the proximity of the material source and blending plant to the stabilised site.

Recent research into triple blending (e.g. cement, fly ash and slag) are showing extended working times of up to 8 hours for specific soil types with reduced susceptibility to rapid reductions in strength gains as a result of compaction delays outside the limits.

2.5 Visual Signs of Pavement Failure

Pavements distress can be visually assessed by checking for:

- Deformation
- Cracks
- Edge defects
- Potholes and patches
- Loss of aggregate from bitumen surfacing

Source: *NAASRA (1987)*

Deformation

Deformation is a change in the road surface caused by traffic conditions, environmental conditions, inadequate quality control during construction or a combination of the above. The deformation may reflect either as structural inadequacies in the pavement, subgrade or both. The main attribute is vertical displacement and is measured by the maximum depth obtained under a 1.2 m straight edge.

The four main types of deformation are:

- Depressions - A localised section in the pavement that is lower than the surrounding area. It may be caused by either settlement of a service trench, embankment consolidation or volume change in the subgrade.
- Rutting - Longitudinal deformation usually contained in either the outer or inner wheel paths of the pavement (Figure 2-5).

Figure 2-5 -
Rutting in Road Surface

- Shoving - The bulging of the road surface caused by braking, accelerating or turning motions of vehicles. Shoving is usually prevalent at most heavily traffic intersections (Figure 2-6).

Figure 2-6 -
Shoving in Road Surface

- Corrugations - Transverse undulations which are regularly spaced usually caused by an unstable base layer in the pavement.

Cracks

Cracks are fissures from partial or complete fractures of the pavement surface. They can appear in a wide variety of patterns from single cracks to complex interconnected cracks extending over the pavement surface. If cracks are left untreated they can lead to premature failure of the pavement caused by the ingress of water to the underlying layers. The main types of cracks are:

- Meandering / Diagonal Cracks - These cracks can be caused by reflection from underlying layers, tree roots or differential settlement (Figure 2-7).
- Transverse Cracks - Cracks running transversely across the pavement. They can be reflecting to the surface from underlying layers, shrinkage cracking or along a construction joint (Figure 2-8).
- Longitudinal Cracks - A single crack or a series of cracks running parallel longitudinal along the pavement. They can be caused by poorly constructed construction joints, differential settlement or reflection cracking from underlying layers (Figure 2-9).

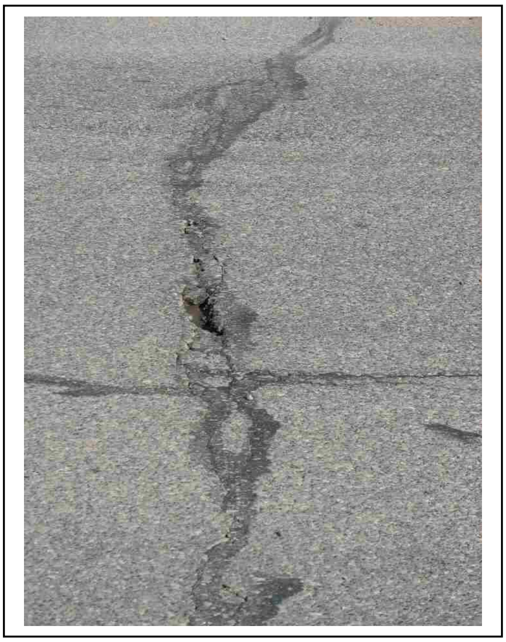


Figure 2-7 - Meandering Crack in Road Surface

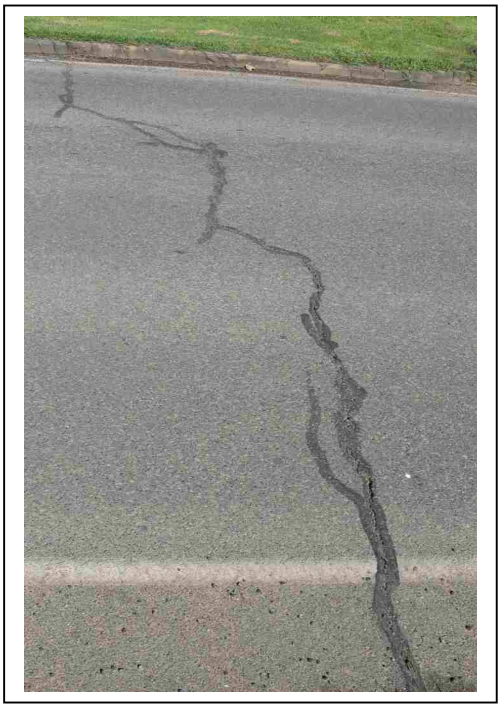


Figure 2-8
Transverse Crack in Road Surface

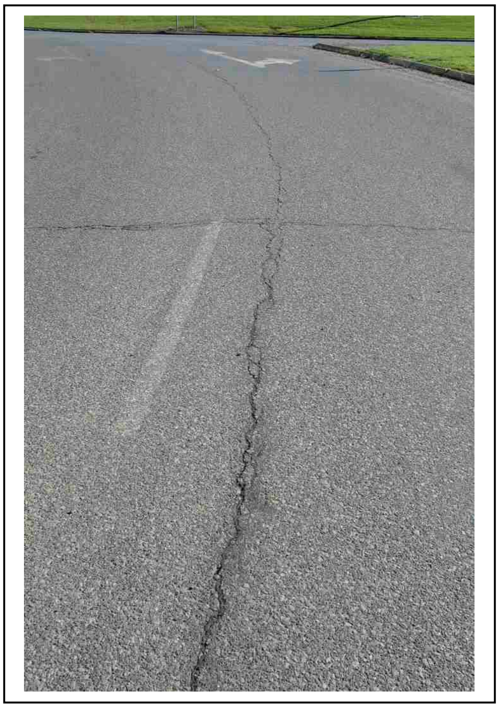


Figure 2-9
Longitudinal Crack in Road Surface

- Block Cracks - Interconnecting cracks that form a series of blocks in the pavement ranging in size from 200 mm to 2000 mm square. The cracks will usually occur due to shrinkage cracking in the underlying cement modified pavement layer or in more rigid pavements.
- Crocodile Cracks - A series of interconnecting cracks that resemble the back of a crocodile. They are usually caused by fatigue failure in an aging flexible pavement or due to inadequate thickness in the base layer (Figure 2-10).

Figure 2-10 - Crocodile Cracks in Road Surface

- Crescent (Shear) Cracks - Half moon shaped cracks which occur because of a poor bond between the wearing surface and the base layer. They usually occur because of high horizontal shear stresses due to braking and cornering.

Edge Defects

Edge defects occur at the interface between the bitumen surface and the unsealed shoulder material (Figure 2-12). The reasons for edge defects are:

- inadequate pavement thickness and width;
- erodible shoulder material causing poor edge support; and
- traffic travelling on shoulder edge.

Figure 2-12 - Edge Failure along Road

Figure 2-11 - Pothole in Road

Potholes and Patches

Potholes are depressions in the pavement created by traffic abrading surface imperfections which allow the ingress of water. The ingress of water causes the fine components in the base layer to go plastic, the subsequent loss of in mechanical interlock between the particles causes the pothole to propagated (Figure 2-11).

Loss of Aggregate

Loss of aggregate on the sprayed bitumen surface can significantly impact on the serviceability of the pavement. Aggregate loss can result from excessively hot weather which reactivates the bitumen in the seal, poor surface preparation prior to the sealing operation can leave loose fine material on the surface of the base layer preventing the binding of the bitumen to the surface. If recognised early, it can be rectified by resurfacing the affected areas before damage to underlying layers.

CHAPTER 3 – ROAD PAVEMENT DESIGN

The design of pavements has altered over the years partly necessitated by the increasing use of binding materials in the pavement layers both for new construction and for rejuvenation. Firstly, there is more experience available for assessing the success of design using empirical methods, and secondly the advent of computers has made possible the widespread use of elastic modelling. When pavement layers are bound using additives, the increased stiffness means that the structure is outside the bounds of the empirical design methods and mechanistic design procedures should be used. Mechanistic design procedures consider pavement failure by tensile strain at the bottom of asphalt, tensile strain at the bottom of cemented material and by compressive strain at the top of the subgrade layer and attempt to calculate the ability of the design to prevent these stresses exceeding the material capability.

Austroads, the Association of Australian and New Zealand Road Transport and Traffic Authorities is a body with a membership comprising the eight State and Territory road transport and traffic authorities, the Commonwealth Department of Transport and Regional Services in Australia, the Australian Local Government Association and Transit New Zealand. This body has a stated purpose of contributing to the achievement of improved transport related outcomes and is considered to be the primary authority on pavement design and construction in Australia. Expert panels maintain a watching brief on new research, provide a peer review of published work and publish a number of manuals recommending the current best practice for road pavement design and construction.

It aims to provide strategic direction for the integrated development, management, and operation of the Australian and New Zealand road system - through the promotion of national uniformity and harmony, elimination of unnecessary duplication and the identification and application of world best practice (Austroads 2004 p iv).

As well as accumulating the knowledge base relating to road pavement design and construction into a series of manuals, Austroads also initiates research and publishes technical notes expanding on the information contained in the manuals. The information is regularly reviewed by expert panels from the membership and the manuals/technical notes are updated to encompass the latest thinking and findings.

A number of these manuals and technical notes have been used for the background for this project and are listed in the References and Bibliography. Other major sources of information are the manuals and technical notes published by AustStab, The Australian Stabilisation Industry Association and the Department of Main Roads Queensland also produces design manuals and technical notes specifically for use for designing and constructing main roads within Queensland.

The standard mechanistic design method recommended by Austroads is the CIRCLY program which uses linear elastic multi-layer theory, and is more fully described in Chapter 5. The program requires the material properties of elastic modulus and Poisson's ratio for each layer as well as values relating to number of heavy vehicles which are expected to travel over the road during its useful life.

This chapter provides a brief overview of the techniques currently in use for the design of flexible pavements, and the testing methodologies applicable to ascertain design information used to obtain data for this project. Some of the design topics mentioned in this chapter are described in greater detail in Appendix B.

3.1 Flexible Pavement Design

There are two methods for the design of flexible pavements currently in use:

a) The Empirical Method is a traditional method which requires the knowledge of the CBR and the total number of equivalent standard axles over its design life. The method is based on observed performance of pavements in-service. The only failure method considered is the failure of the subgrade (Figure 3-1) causing rutting and tables and charts are provided to determine the required thickness of the subbase and base to prevent the high stresses reaching the subgrade and causing failure. This method is applicable to the design of unbound flexible pavements, but has limited application for flexible pavements with bound layers as they have different failure modes.

b) The Mechanistic Method attempts to ascertain the point of failure by calculating the critical stresses and strains that occur throughout the multi-layered structure based on the linear elastic multi-layer theory. The CIRCLY program uses the linear elastic multi-layer theory adopted by Austroads. The program requires the material properties of elastic modulus and Poisson's ratio for each layer as well as values relating to the standard axle repetitions for each of the failure modes.

The failure modes considered applicable are:

- tensile strain at the bottom of asphalt;
- tensile strain at the bottom of cemented material; and
- compressive strain at top of subgrade.

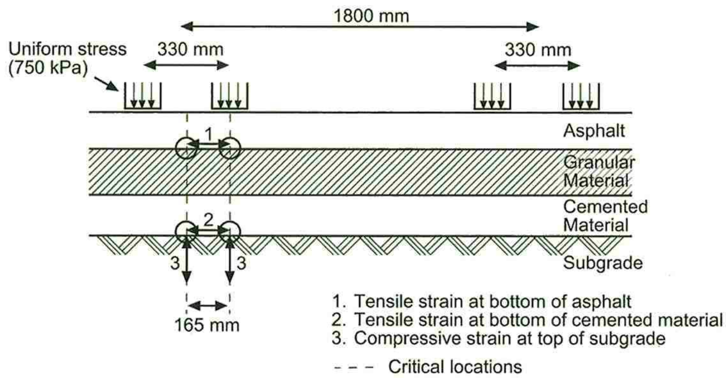


Figure 3-1 - Failure Modes in Pavement Design
Source: *Austroads Pavement Design Guide (2004)*

This method is applicable for flexible pavement design using both unbound and bound pavement material and rigid pavement design and a combination of these.

Both these methods require an estimate of the total number of compressive actions caused by the wheels of vehicles to successfully design the pavement.

The empirical method uses the estimated value of "Equivalent Standard Axles" (ESA) while the mechanistic method uses "Standard Axle Repetitions" (SAR) for each failure mode and would normally be different for each failure mode. These values are taken over the design life of the pavement.

The calculation of these figures requires an estimate of volume of traffic traversing the pavement. Because the damage caused is a power relationship to the applied load, the damage caused by light passenger and similar vehicles is negligible, so an estimate of heavy vehicle traffic only is required. Commonly (and historically, where only simplistic traffic counters were available), the loadings are based on short-term total counts and a small number of manual counts to estimate the percentage of heavy vehicles, their assumed loadings and the distribution of different heavy vehicle types. Hence, it is common to estimate a value designated heavy vehicle axle groups, N_{DT}, as the first step in calculating the required traffic parameters.

3.2 Design Traffic

Both the empirical and the mechanistic design methods relate pavement capability to the number of passes of a standardised axle loading which will be experienced by the pavement over its useful life. The empirical method uses this value as an input to the design whereas the mechanistic method calculates the capacity of the proposed design which is then compared with the expected values to determine if the design is adequate.

The basic method for calculating N_{DT} as proposed by AustRoads is the following formula:

$$N_{DT} = 365 \times (AADT \times DF) \times \frac{\%HV}{100} \times N_{HVAG} \times LDF \times CGF$$

where AADT = Average annual daily traffic (vehicles per day)

DF = Direction Factor - the proportion of the two-way AADT travelling in the direction of the design lane.

%HV = Average percentage of all traffic comprising heavy vehicles.

N_{HVAG} = Average number of axle groups per heavy vehicles

LDF = Lane Distribution Factor

CGF = Cumulative Growth Factor

The determination of the above parameters is discussed in more detail in Appendix B. It should be noted that as the damage caused has a power relationship to the load, damage caused by light commercial and passenger vehicles is insignificant compared to that caused by heavy vehicles, so only heavy vehicles are considered in the above formula.

This formula for estimating N_{DT} forms the basis for obtaining the design figures required for the relevant pavement design method, viz Equivalent Standard Axles

(ESA) for the Empirical Method and the three values of the Standard Axle Repetitions (SAR) for the Mechanistic Method. Some of the issues in determining these values are detailed in Sections 3.3 and 3.4 below.

3.3 Traffic Data Collection

The methods for collecting traffic data range from the simplistic manual traffic counting to the advanced weigh-in motion systems. The common methods are described in more detail in Appendix B.

The usual approach to obtain a traffic count is to install a twin-tube Vehicle Classification Counter for approximately two weeks, often repeating the count after an interval of one or two months. The counter data coupled with information from weigh-in-motion systems and manual observations can provide a reasonable estimate of the number and probable loading of the heavy vehicles using the road.

A forecast of the likely growth in traffic volume over the following 25 – 30 years is again based on historical data and a view of the expected economic growth in the region affecting traffic for the road in question. A good crystal ball is an advantage.

3.4 Imposed Axle Loadings

Both the empirical and mechanistic design methods require the heavy vehicle axle groups to be converted to Equivalent Standard Axles (ESA) or Standard Axle Repetitions (SAR).

Equivalent Standard Axles (ESA) / Standard Axle Repetitions (SAR)

The Standard Axle is defined in the Austroads manual as:

> *" a single axle with dual wheels carrying a load of 80 kN. The circular contact stress being applied to the pavement at 330 mm centres over each dual wheel is 750 kPa for highway traffic"* (Figure 3-2).

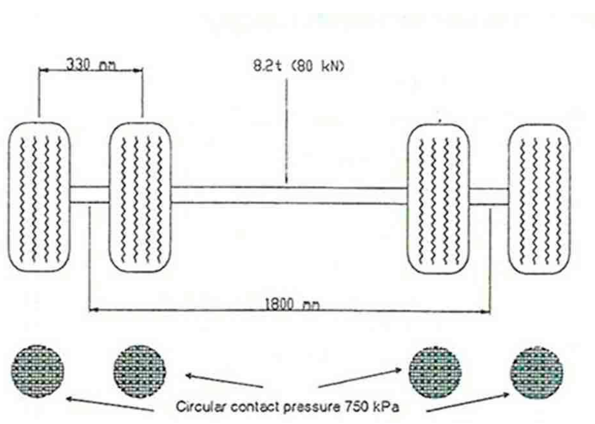


Figure 3-2 - Standard Axle
Source: *Pavement Design Training Manual (MRD)*

Experimental work has determined that different axle profiles can carry different loads to cause the same amount of damage as a standard axle (Table 3.1).

Table 3.1 - Axle Load Values Equivalent to a Standard Axle

Axle Group Type	Load (kN)
Single Axle with Single Tyres (SAST)	53
Single Axle with Dual Tyres (SADT)	80
Tandem Axle with Single Tyres (TAST)	90
Tandem Axle with Dual Tyres (TADT)	135
Triaxle with Dual Tyres (TRDT)	181
Quad-axle with Dual Tyres (QADT)	221

Source: *Austroads Pavement Design Guide (2004)*

If an axle group is loaded to a different loading from that shown in Table 3.1 it is necessary to calculate the equivalence in terms of the standard axle. Experiments have shown that the equivalence obeys the following formula:

$$EA = \left(\frac{L}{SL}\right)^m$$

where: EA = Equivalent number of standard axles

L = Actual load of axle group

SL	=	Standard load for that axle group
m	=	An exponent depending on the method of failure

For the empirical method, design is based on the strength of the subgrade and the exponent is 4. The mechanistic method uses three failure modes - fatigue of the asphalt layer (exponent of 5), rutting/shape loss (exponent of 7) and fatigue of cement material layer (exponent of 12).

Where counts of the number of different axle group and the axle group loads have been estimated, the above formula can be applied to the percentage of each vehicle type and its load, then summated to provide the design ESAs or SARs. Obviously this is a very onerous calculation which requires an extensive knowledge of the traffic volumes for the forecast period as well as accurate details of the heavy vehicle loadings and axle types. However, in practice, the future traffic volume can only be at best an estimate, based on current data and a forecast of the development of the economic activity of the surrounding area and its impact on the traffic volume and vehicle loads. Consequently, most organisations tend to use predetermined average factors for each of the calculations.

3.5 Empirical and Mechanistic Pavement Design Methods

The empirical method of design uses a design chart to enable determination of pavement layer thicknesses based on the strength of the underlying layer represented by its Californian Bearing Ratio (CBR). The chart currently used in Australia is contained in the Austroads Pavement Design Manual (Figure 8.4). It may be used solely for pavements comprised of unbound layers of granular material which are surfaced with either a bituminous seal or thin asphalt layer (less than 40mm).

The design chart is reproduced as Figure 3-3.

The mechanistic method of design uses a computer program to analyse the performance of pavement layers based on a structural model of the pavement.

Each layer is considered to be comprised of a homogeneous linearly elastic material characterised by its elastic stiffness properties ie modulus and Poisson’s ratio. The program most commonly used in Australia is the CIRCLY program written in 1977 by Dr Leigh Wardle at CSIRO and further described in Chapter 5.

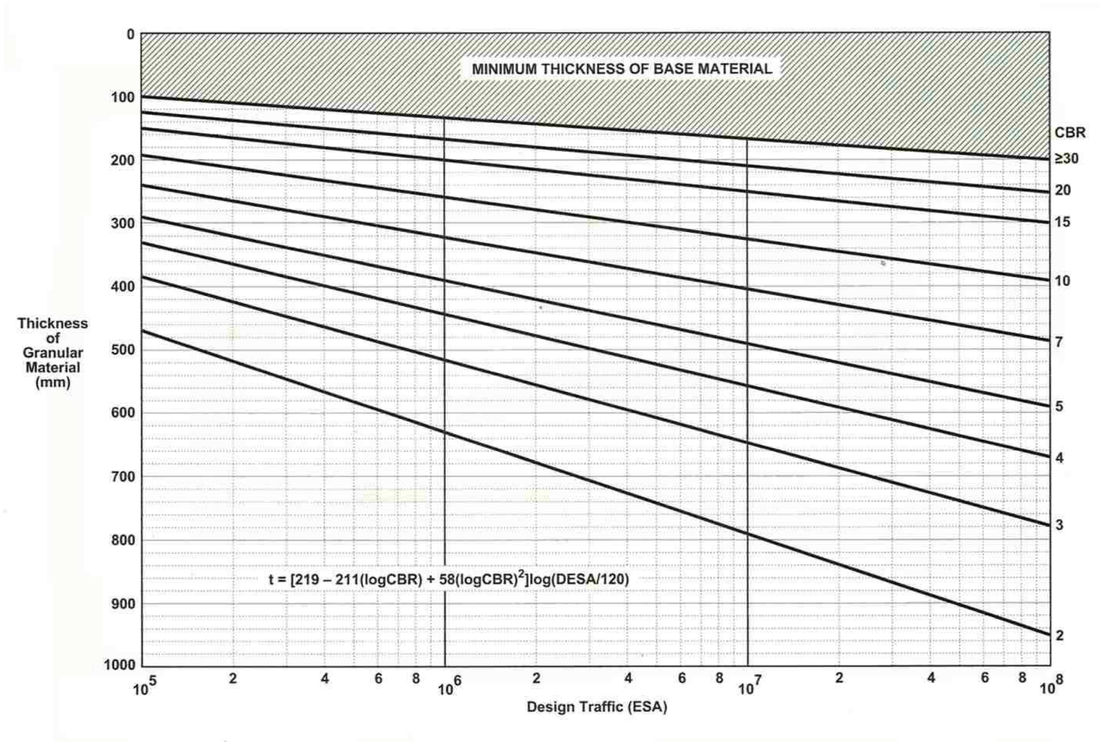


Figure 3-3 - Empirical Road Pavement Design Chart
Source: *Austroads Pavement Design Guide (2004)*

The empirical method has limited use for stabilised pavements, so CIRCLY design is generally used to design the insitu stabilised pavements. The capacity of a design is evaluated and compared against the required SARs for the three failure modes, the design being modified and re-analysed until a satisfactory solution is attained.

3.6 Pavement Design for Insitu Stabilisation

3.6.1 Selecting the Stabilisation Additive

There are numerous products available on the market today which can be used as additives for the stabilisation of existing road pavements. The tests used to determine the most appropriate product to use are the particle size distribution and the Atterberg limits.

The desirable particle size distribution is achieved when each smaller particle size can fit into the void formed by the larger sized particles in close contact. This provides transfer of mechanical strength through the pavement. A well-graded mix with favourable particle shapes and texture can be compacted to a state in which it has adequate stability, low permeability and good wear resistance.

The particle size distribution of the material is determined by passing a sample through a series of standard sieves and weighing the portion retained on each sieve. The size distribution is described in terms of the cumulative percentage mass of the particles passing each sieve.

The Plasticity Index (see Appendix B) is useful to give an indication of the bindability and workability of gravel mixes and their suitability as pavement material. Typical PI values will depend on the position of the layer in the pavement. Top base layers are normally constructed of high strength material, low in fines, relying mainly on internal friction between particles for its load bearing capacity and stability. The PI for this layer will generally have a maximum of 4%. Subbase material which is lower in strength and higher in fines relies on both internal friction and cohesion properties to achieve the required strength and stability. The PI will increase to 12% because of the increased percentage in fines.

Researchers at the University of South Australia investigated various binders on 20 types of Australian soils and developed a chart for the determination of the most suitable binder based on its plasticity index and the size of material passing a 75μm sieve (Symons, M.G. and Poli, D.C. 1998). Based on this research Austroads offers a guide for selecting a method of stabilisation as displayed in Table 3.2.

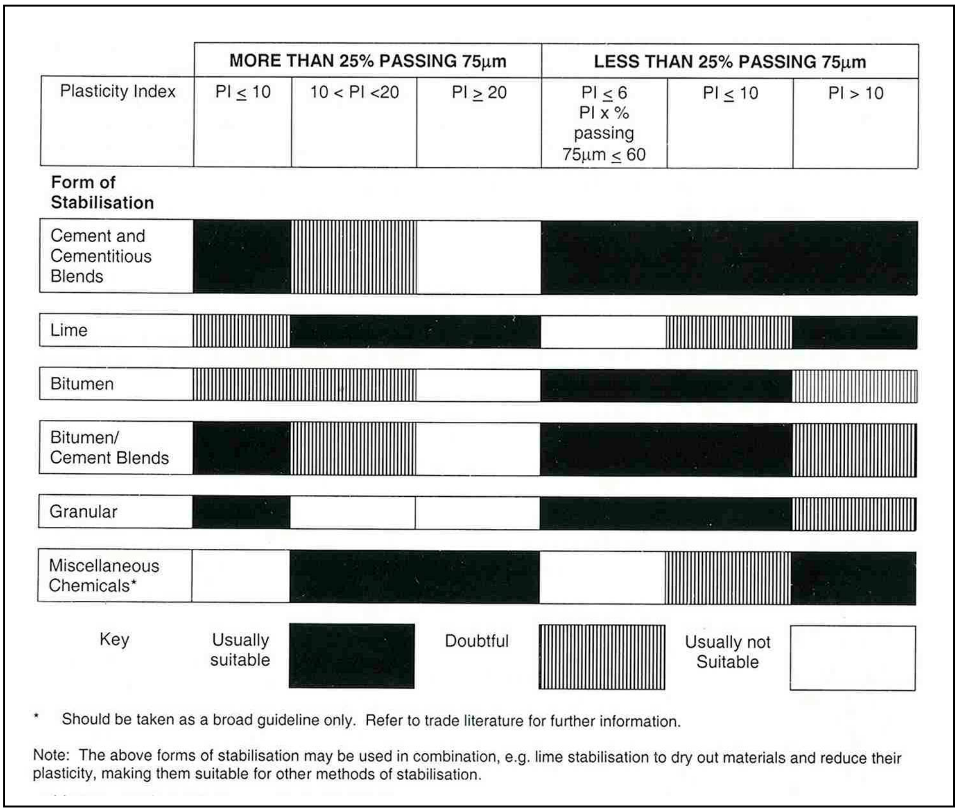
Cement stabilisation can be successfully used on any material which has less than 25% passing a 75μm sieve (coarser material). However, for material with more than 25% passing a 75μm sieve (finer material) the PI must be less than 10 for cement stabilisation to be appropriate.

3.6.2 Design Properties of the Stabilised Mix

Before designing the re-construction of a pavement, the quality of the existing pavement needs to be determined, and in particular, the modulus and Poissons Ratio of the stabilised mix must be determined.

The grading of the existing road pavement also needs to be checked as grading as well as the strength can deteriorate over time and traffic. Hence, physical laboratory testing of samples of the material in the existing pavement is required to determine the amount of cement material required to produce a suitable mix.

Table 3.2 - Selection Guide for Different Stabilisation Methods



Source: *Austroads Guide to Stabilisation in Roadworks (1998)*

If the grading is not suitable, new material must be brought in to mix with the material in the recycled layer. Various percentages of cement powder are then added to samples of the final mix and the unconfined compression strengths (UCS) determined. As a guide, a mix with a modulus between 600 and 1500 MPa is sought for normal country roads (Austroads 1998).

Note also that, although machinery for deep layer re-construction is available, the machinery normally available in the Mackay district limits the layer depth to 250 mm, which must be accounted for in the design process.

In brief, some or all of the following sampling and tests may be carried out to provide the data necessary to develop a design for the insitu stabilisation of an existing pavement.

- Measurement of pavement deflections by a Falling Weight Deflectometer (FWD).
- Assess subgrade bearing capacity using a Dynamic Cone Penetrometer (DCP), moisture content and soaked Californian Bearing Ratio (CBR)
- Excavation of test pits for measuring material properties and sampling materials.
- Laboratory based materials mix design.

3.6.3 Testing the Existing Pavement

Soil properties of the existing pavement and subgrade are required so that the most appropriate design can be produced. The mechanistic design method requires the modulus and depth of each layer in the final design, hence the modulus of the subgrade and of each layer which is not disturbed by the rehabilitation must be found. This is done by a series of field and laboratory tests.

The moduli are not usually found directly, unless a Falling Weight Deflectometer or equivalent device is available. In most regional areas, the properties are found by determining the moisture content, grading, Californian Bearing Ratio, Liquid Limit, Plastic Limit, Linear Shrinkage and the Plasticity Index.

Investigation pits are dug at predetermined locations, layer thickness recorded and material samples taken for each layer.

Dynamic Cone Penetrometer

The strength of the natural subgrade material layer is the starting point for the evaluation process. The Dynamic Cone Penetrometer (DCP) test (MR Test Method - Q114B) is used to determine the in-situ bearing capacity of the underlying subgrade. The penetrometer is a two metres long steel rod with a standard size hardened steel cone at the penetrating end. The upper end of the steel bar has a captive weight surrounding the rod, the weight is able to fall freely through a given drop height to achieve a standard amount of penetrative effort at every drop. By plotting the penetration of the cone against the number of drops of the weight the approximate CBR figure can be found by applying the formula:

$$LogCBR = 2.628 - 1.273 \log(DCP)$$

where: CBR = Californian Bearing Ratio

DCP = penetration mm per blow

Moisture Content and Soaked CBR

The moisture content (MR Test Method - Q102A) is determined at the time the DCP is performed. The moisture content indicates the level of saturation of the subgrade so that a determination can be made on whether the DCP result is the worse case scenario or whether a soaked CBR test is required in the laboratory.

Soaked CBR tests (MR Test Method - Q113C) are performed in wet coastal regions because the subgrade is likely to be saturated for a substantial period and saturated CBR value is more relevant.

Californian Bearing Ratio

Where the modulus cannot be measured directly, it can be estimated from the Californian Bearing Ratio of the material. The Californian Bearing Ratio (CBR) test measures the force needed to cause a 50 mm diameter plunger to penetrate 2.5 mm into a sample. It was developed by the US Corps of Engineers. The original test material was a Californian crushed rock, which was given a CBR value of 100. The strength of other materials is proportionally related to that bearing capacity and expressed as a percentage (typical values for subgrade range between 2 - 10%).

For each layer of the pavement material layers found during the investigation a CBR value needs to be determined. The standard soaked CBR test is used for granular material (MR Test Method - Q113A) which is slightly different to the CBR test for the subgrade.

Where the modulus is determined from CBR test results, the empirical relationship adopted by the Austroads Pavement Design Guide is used to convert the CBR strength to an elastic modulus (E). For the subgrade the relationship is:

$$E = 10 \times CBR$$

where: E = Elastic Modulus (MPa)

CBR = Californian Bearing Ratio (%)

For the subbase and base, in the absence of better information, the same relationship may be used, however, a more accurate (but still approximate) value based on research is used by some authorities. The MRD has adopted the relationship shown in Figure B-2 in Appendix B.

Particle Distribution

To ensure mechanical interlock between the particles in the pavement layer a particle distribution test (MR Test Method - Q103A) is performed. This test will determine whether additional grading correction material is required to achieve a greater mechanical interlock between particles.

Atterberg Limits

The Atterberg Limits - Liquid Limit (MR Test Method - Q104A), Linear Shrinkage (MR Test Method - Q106) and Plastic Limit (MR Test Method - Q105) - is conducted for each layer of the pavement. These test are useful to give an indication of the bindability and workability of gravel mixes and their suitability as pavement material. The plastic index is also used as a guide for the determination of the type of stabilisation to use.

Cement Additive Percentage

Once a decision has been made on the grading of the mix for the insitu stabilised layer, the appropriate cement content to achieve a resilient modulus between 600 and 1500 MPa is determined. This is achieved through a series of Unconfined Compressive Strength (UCS) tests (MR Test Method Q115C).

This test entails the addition of a range of cement contents ranging from 1% to 4% in 0.5% intervals to material samples which are the same as the final composition of the rehabilitated pavement layer. The material is compacted into test moulds, removed from the mould, sealed in an airtight container and placed in a curing room at approximately 100 percent humidity and 24°C for a period of 7 days. The cylinder is then immersed in water for 4 hours, removed and placed in the compression testing machine and loaded to failure at a constant rate of stress of 1.0 mm/minute, the maximum load being recorded. The compressive strength is determined from the maximum load applied divided by the cross sectional area of the sample.

To be suitable for normal country main roads, the unconfined compressive strength is generally required to be in the range of 0.6 to 1.0 MPa. The lowest cement content which fulfils this requirement will be selected as the cement powder is the most expensive component of the mix. This strength can be empirically converted to a modulus for mechanistic design purposes. The most generally used relationship for conversion is as follows:

$$E = k \times UCS$$

where:	*E*	=	Elastic Modulus (MPa)
	UCS	=	Unconfined Compressive Strength of laboratory specimen at 28 days (MPA).
	k	=	values of 1000 to 1250 are typically used for General Purpose Cements. The value of 1000 is adopted in Mackay.

Sources: *Austroads (2004)*

3.7 Project Testing

As previously described, a number of recently in-situ stabilised projects have been selected for investigation. These lots contain a significant proportion of test sites where the Relative Dry Density was below 100%, and also where the Characteristic Value is less than 100%. As each of these projects have been completed for several months or more, it can be assumed that the pavement is now well cured, and little additional strength will be developed in the future. Hence, if the strength of the pavement can be measured, it may be possible to draw comparisons between the construction tests and the developed strength, and draw conclusions about the validity of using the construction testing to determine the working life of the rehabilitated pavement.

Two tests will be applied, firstly a visual inspection of the lots in the selected projects for signs of distress, and secondly, a non-destructive Falling Weight Deflectometer test at 89 of the sample sites to evaluate the "cured" modulus and subsequently an estimate of the pavement's capacity and life.

3.7.1 Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD) uses a falling mass to generate a load pulse of similar magnitude and duration to an Equivalent Standard Axle travelling at high speed. When released from a specified height, the mass falls onto buffers mounted on a rigid circular plate lowered onto the pavement. Geophones placed on the pavement at 0, 200, 300, 450, 600, 900 and 1500 mm intervals from the load measure the resultant velocity. The data can then be processed to produce individual bowl shapes.

Back analysis of the deflection bowls are then carried out using the Queensland Department of Main Roads developed program called CIRDEF (CIRCLY based iterative elastic analysis program).

For a given pavement layer configuration, the combination of pavement and subgrade moduli that produces a theoretical deflection bowl that matches the measured deflection bowl shape is determined. The procedure involves the selection of initial seed moduli values for the pavement and subgrade layers. The program then computes the theoretical bowl shape, calculates the absolute sum of the differences between measured and computed bowl shapes, adjusts the layer moduli based on the initial results and repeats the procedure until an acceptable fit is obtained or the limiting number of iterations is reached. The combination of pavement and subgrade moduli that produce the 'best fit' are reported as the calculated insitu moduli. The insitu moduli will then be substituted for the design moduli in CIRCLY and the design life determined.

CHAPTER 4 – RECENT RESEARCH

For many years, Austroads, the Association of Australian and New Zealand Road Transport and Traffic Authorities, has been providing a source of the accumulated Australian and overseas knowledge, experience and research relating to road pavement design and construction. The collected information, including the adoption of the results of new research following peer review by expert panels, is promulgated in a series of manuals and technical notes.

The most comprehensive of these manuals related to the topic of this project are:

- Austroads, Pavement Design, A Guide to the Structural Design of Road Pavement, 2004;
- Austroads, Guide to Pavement Technology, Part 4D: Stabilised Materials, 2006;
- Austroads, Mix Design for Stabilised Pavement Materials, 2002;
- Austroads, Guide to Stabilisation of Roadworks, 1998.

With the rise in popularity of stabilised pavement material, another body, AustStab, The Australian Stabilisation Industry Association was formed in mid-1995. It was initiated by the major contractors and charged with promoting the stabilisation and road recycling industry, setting national standards of performance, assisting in and coordinating research, and educating and training people in the industry.

The AustStab website contains guidelines, technical notes and research publications promoting the proper use of the insitu stabilisation process for civil construction projects. Examples of published technical notes are:

- Smith. W. and Vorobieff. G. (2007), Recognition of sustainability by using stabilisation in road rehabilitation, ASA Sustainability & Slag Conference;
- AustStab (1999), Australian Binders used for the Stabilisation and Road Recycling Industry, National AustStab Guidelines; and
- AustStab (1999), Site investigation for the rehabilitation of low trafficked roads using insitu recycling, National AustStab Guidelines.

In the interest of corporate uniformity, each road authority in Australia maintains a series of manuals, which, while based on the Austroads recommendations, includes organisation-specific information. The Department of Main Roads Queensland has produced a number of manuals and workshops including:

- MRD Pavement Design Manual;
- MRD Pavement Rehabilitation Manual;
- MRD Workshop on Low Volume Roads; and
- MRD Material Testing Manual.

These sources have been heavily relied upon for much of the information contained in this document.

In 1997, Lake Macquarie City Council conducted research on the performance of ten roads which had been rehabilitated by insitu stabilisation over a seven year period (Pike 1997). Pike carried out multiple Benkleman Beam tests on each of these pavements to find the average deflection for each, and where pre-rehabilitation data was available (on six of the ten), compared the before and after results (Table 4.1).

Table 4.1 - Description of Traffic and Benkleman Beam Data

Street Name	Pave-ment Age (years)	Stabilised Depth (mm)	Cumulative Traffic to Date (ESA's)	20 Year Design Traffic (ESA's)	Ratio of actual to design traffic	Benkleman Beam Deflections	
						Before	June 1997
Gradburn & Curdie St	6.7	180	$9.70E^{4}$	$2.80E^{5}$	0.35	0.62	0.45
Statham St	6.2	180	$5.90E^{4}$	$1.90E^{5}$	0.31	0.78	0.48
The Groves	6.2	180	$1.20E^{5}$	$3.90E^{5}$	0.31	1	0.22
Ian St	5.4	180	$6.00E^{4}$	$2.20E^{5}$	0.27	1.03	0.72
Dalwood Crt	4.7	180	$3.80E^{3}$	$1.60E^{4}$	0.24	N/A	0.61
Tahlee St	2.9	180	$4.20E^{2}$	$2.90E^{3}$	0.14	N/A	0.51
Jame1.4s St	2.9	180	$3.70E^{3}$	$2.60E^{4}$	0.14	1.21	1.13
Tennent Rd	2.2	200	$1.70E^{4}$	$1.60E^{5}$	0.11	N/A	0.38
Albert St	1.4	200	$4.60E^{3}$	$6.70E^{4}$	0.07	N/A	0.36
Robiina Dr	0.3	180	$2.60E^{3}$	$1.60E^{5}$	0.02	0.72	0.49

Source: *Pike (1997)*

He also back-calculated from the Benkleman beam deflection bowls (using the program EfromD2 which is similar in function to CIRCDEF) to find the average moduli for each road. This then allowed him to "re-design" the road using CIRCLY and estimate the maximum base and subgrade strains to predict the remaining useful service life. Typical moduli obtained ranged from 1100 to 2800 MPa.

The pavement material used for all the roads tested was from the same quarry so that the "make-up" material used was consistent. A General Blend cement binder was used, 80% GP cement and 20% fly ash, at a rate of between 4% and 5%. This percentage is relatively high compared with Mackay District practice, where a maximum content of 3% is used, more commonly around 2%. The depth of stabilisation varied from 150mm to 200mm, similar to the stabilisation depth for this study.

The roads studied were relatively lightly trafficked, the cumulative design traffic for a 20 year life of each pavement being between 3×10^3 and 4×10^5 ESA's. This is an order of magnitude lighter than the typical design traffic applied to the pavement designs for the Mackay study.

Pike concluded that the results indicated that the insitu stabilisation process carried out on these ten roads produced a rehabilitation of the road that could be expected to provide a service life of at least the 20 year design life. All deflection comparisons showed a substantial decrease in deflection under test (ie an improvement in strength), and the strength was maintained over at least the five year testing period.

Although several pavements exhibited reflective cracking, (probably a symptom of the higher binder percentage, thin base and weak sub-structure), Pike found that there was no indication from the test results that the service life would be adversely affected.

Although Pike's work is similar to that being undertaken for this project, there are significant differences in the pavement design parameters, and the main thrust of the work is to evaluate the effectiveness of using Relative Dry Density as a measure of the pavement's ability to last for the design period. Also, the Falling Weight Deflectometer has replaced the Benkleman Beam, and it is expected that the reliability of the moduli calculation will be somewhat better.

In 1995, Fairfield City Council carried out research on the performance of its local road network. The Council had carried out insitu stabilisation on many of its road pavements since 1965. The depth of stabilisation varied from 150 to 225 mm and the percentage of cement binder varied from 3% to 6%. The Council used a pavement condition index (PCI) designed by SMEC to rate the pavement. The data was analysed in three traffic ranges, AADT < 500, 500 ≤ AADT ≤ 2000 and AADT ≥ 2000 (Meijer 1995). The depths, cement contents and ranges are similar to this project.

The data indicated that a majority of cement stabilised pavements had performed reasonably well at ages up to about 25 years for the two traffic categories less than 2000 AADT. Very few roads were stabilised where the traffic volumes exceeded 2000 AADT and the results showed mixed performances, although a 20 year design life appeared achievable.

The Township of Payneham used cement insitu stabilisation for six streets in the early 1970's. In keeping with practise in that period the cement contents were higher than used today and were typically 6%. This high percentage introduced early shrinkage cracking in the pavement which required replacement of the surfacing. Despite the cracking the pavement has not lost shape (Amey 1987). Similar lessons where learnt in the Mackay District through the late 1970's and early 1980's. The common practice today, which is detailed in the local design testing brief, is to limit cement contents to 3% which in most circumstances eliminates early cracking and reduces the need for additional sealing requirements.

In western Sydney in 2004, five rehabilitation options for a typical pavement where assessed against the direct cost, social and environmental benefits (Smith & Vorobieff 2007).

The five rehabilitation options considered which provide a similar pavement life based on a set traffic volume is displayed in Table 4.2.

The direct costs of each alternative in Table 4.3 were calculated using typical Sydney urban construction costs. As can be seen from the table the stabilisation treatments have the lowest construction rates.

Table 4.2 - Pavement Rehabilitation Options

No	Option	Details	Depth (mm)
1.	Granular pavement with seal (Reconstruction)	Mill out existing pavement to depth. Replace with quality granular material. Bitumen 2 coat seal wearing surface	520 520
2.	Granular pavement with asphalt surfacing. (Reconstruction)	Mill out existing pavement to depth. Replace with quality granular material. Asphalt wearing surface	520 470 50
3.	Stabilised Base Course with asphalt surfacing	Mill out blend material, remove for given final level. Cement Stabilise Asphalt wearing surface	60 335 50
4.	Deep asphalt Base Course	Mill out existing pavement to depth Replace with asphalt	180 180
5.	Stabilised subgrade, stabilised base with asphalt surfacing	Mill out blend material, remove for given final level. Mill and side cast base course Subgrade stabilise with lime Reinstate base course and stabilise Asphalt wearing surface	60 250 200 250 50

Source: *Smith & Vorobieff (2007)*

Table 4.3 - Direct cost estimate of each pavement rehabilitation option

No	Option	Direct Cost ($/m²)
1.	Granular pavement with seal (Reconstruction)	78
2.	Granular pavement with asphalt surfacing. (Reconstruction)	84
3.	Stabilised Base Course with asphalt surfacing	29
4.	Deep asphalt Base Course	65
5.	Stabilised subgrade, stabilised base with asphalt surfacing	39

Source: *Smith & Vorobieff (2007)*

An important social consideration when considering rehabilitation options is the expected duration of works, particularly at sites with high traffic flows. The disruption caused by the roadworks is assigned a value in order to compare options and establish which is the best. A value is difficult to quantify when taking account of the disruption to economic activity/business, personal activity, public services, emergency services and political cost for governing authorities dealing with community concerns as a result of the disruption. Table 4.4 displays the expected duration for each of the options and the corresponding road occupancy cost. As can be seen from the table the stabilisation treatments again have the lowest rates.

Table 4.4 - Duration of construction and road occupancy costs

No	Duration (day)	Lane Occupancy Rate ($/day)	Lane Occupancy Cost ($)	Lane Occupancy Cost ($/m²)
1.	12	1000	12 000	6.00
2.	12	1000	12 000	6.00
3.	3	1000	3 000	1.50
4.	5	1000	5 000	2.50
5.	5	1000	5 000	2.50

Source: *Smith & Vorobieff (2007)*

A number of the previously listed environmental advantages (2.2.2 Rationale and Benefits) can be quantified for each of the options. Table 4.5 displays the cost for various environmental elements for each of the options.

Table 4.5 - Cost for various environmental elements for each of the options.

No	Loss of Material Asset Cost	Disposal Cost	CO² Cost	Noise Cost	Road Injury Cost	Quarried Materials 'Levy'	Total	Total ($/m²)
1.	$2 100	$65 520	$343	6.00	$134	$4 368	$72 584	36.30
2.	$2 100	$65 520	$346	6.00	$135	$4 428	$72 650	36.30
3.	$500	$7 560	$40	1.50	$16	$480	$8 609	4.30
4.	$1 500	$22 680	$128	2.50	$50	$1 728	$26 130	13.10
5.	$500	$7 560	$40	2.50	$16	$480	$8 609	4.30

Source: *Smith & Vorobieff (2007)*

Summary values for direct, social and environmental costs per square metre are listed in Table 4.6. As can be seen, the benefits of the stabilisation based options on a direct cost basis are further emphasised with the additional consideration of social and environmental costs. It is important to outline at this point the significant environmental costs of the other options. This is why cement insitu stabilisation is growing as a rehabilitation and reconstruction technique.

Table 4.6 - Cost for various environmental elements for each of the options.

No	Direct Cost ($/m²)	Social Cost ($/m²)	Envir. Cost ($/m²)	Total Cost ($/m²)
1.	78.00	6.00	36.30	120.30
2.	84.00	6.00	36.30	126.30
3.	29.00	1.50	4.30	34.80
4.	65.00	2.50	13.10	80.60
5.	39.00	2.50	4.30	45.80

The question of the strength improvement achieved by the addition of binders was addressed by Vorobieff in a paper presented to the NZIHT Stabilisation of Road Pavements Seminar in 2004 (Vorobieff 2004). Figure 4-1 taken from that paper shows typical UCS values that could be expected by adding from 1% to 6% of cementitious binder to two typical road base materials. Typically, small amounts of additive (1% to 2%) would be expected to result in a UCS up to approximately 1 MPa, and is defined as Modified pavement material. Material with greater percentages are classified Lightly Bound until a UCS around 4 MPa, after which the material is classified as Heavily Bound.

Vorobieff also notes that it is risky to use heavily bound thin layers (100 to 250mm) over a flexible base as such a layer has insufficient strength to act as a beam to carry the load, and is likely to fail by flexural (fatigue) cracking, but also notes that more research data is necessary to build confidence in designing with various amounts of binder.

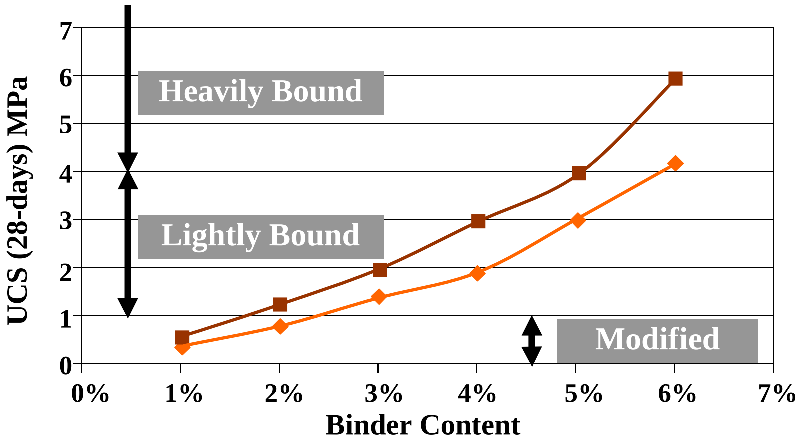


Figure 4-1 - Effect of Cement Content on Strength
Showing typical strength relationships for two different pavement materials with increasing binder content.
Source: *Vorobieff (2004)*

These remarks support the approach which was taken with the design of the rehabilitation of the pavements being considered for this project. All the pavements before rehabilitation were relatively shallow and a maximum cut of 250mm could be used. The range of binder additive used - between 1.5% and 3% - would fairly place the reconstituted pavement as “lightly bound” and flexural cracking problems should be avoided.

Although a number of papers detailing the results of overseas research into the insitu stabilisation process were identified, most of the research involved investigations into the chemical process rather than on the practical application of insitu stabilisation. Several papers dealt with issues such as the effect of sulphates on cement powder, the use of different percentages of ground blast furnace slag or the effects of ice crystals on the curing process. These topics are not directly relevant to this investigation, and did not provide any useful background for this study, and thus have not been summarised in this report. The lack of papers describing practical experience overseas suggests that Australia is in the forefront in using insitu stabilisation for rehabilitating low density roads, perhaps a result of the relativity large distances travelled and the relatively small population compared to many other overseas countries. It is also possible that much of the research is carried out in-house by private industry, and the techniques developed may be considered to be commercially confidential.

CHAPTER 5 – COMPUTER DESIGN PROGRAMS

Road pavements usually consist of multiple layers with different properties and the material in the pavement may behave differently in different directions. Early pavement design was based on experience where designers and researchers drew up empirical charts to aid design. With the advent of computers, opportunities developed to model pavements (and other soil and rock engineering problems) as layered elastic systems with radial variations in contact stress represented by polynomials. These techniques have been shown to provide a reasonably accurate model of the stresses imposed on road pavements by multiple actions of applied pressures equivalent to the passage of heavy axle traffic passing over the road over many years.

The properties of road pavement material are far from uniform, and the modulus or strength of materials can depend on the amount and nature of containment. To model this variation, granular material layers can be subdivided into thinner layers, the properties of each layer being calculated from the bulk property determined in the laboratory. While tedious for hand calculation, this can be readily achieved using computers.

Typically, the analytical solutions for the stresses, strains and displacements involves integral transformation methods to solve integrals of the form:

$$I = \int_0^\infty A(k) J_n(k) J_\tau(kr) \exp(\pm \delta kz) k^\mu \, dk$$

Source: *Gerrard & Harrison (1971), Wardle (1976) (cited in MINCAD Systems 2004)*

where J denotes the Bessel function of the first kind, and r and z are expressed as multiples of the loaded radius. The coefficients $A(k)$ are found by solving a set of simultaneous equations which represent the loading conditions at the surface, the interface conditions between the layers and the conditions at the base of the lowest layer. Thus the number of equations to be solved for each k value increases with the number of layers considered.

One program to solve these integrals was first written by Dr Leigh Wardle at CSIRO (Harrison, Wardle & Gerrard, 1972). The system was further developed and commercialised as CIRCLY by the Melbourne company, MINCAD Systems. Much of the development has been to develop front ends for the CIRCLY engine to provide easier user input to the program and to refine the solution algorithms. The program has been in regular use in Australia and worldwide for two decades and has been adopted by Austroads and MRD as the recommended mechanistic design program for road pavements. It has been used successfully and shown to provide a reasonable model for this application over thousands of design applications, within the limitations of the input data. With continuing use and experience with more CIRCLY designed roads reaching their design life, the validity of the model will be continually tested for the changing pavement designs currently being used.

The CIRCLY engine is also used as the basis for other related programs such as CIRCDEF, a program developed to calculate layer moduli from the results of falling weight deflectometer tests.

5.1 Material Properties

5.1.1 Cross-Anisotropy and Isotropy in Road Pavement Materials

The elastic material in each layer of the pavement is assumed to be homogeneous but can be cross-anisotropic or isotropic. The elastic properties of isotropic materials are the same in both the vertical and horizontal directions whereas a cross-anisotropic material is one in which the elastic properties are equivalent in all directions perpendicular to an axis of symmetry. The axis of symmetry is assumed to be vertical so that properties in the horizontal and radial directions are uniform.

Austroads 2002 recommends that subgrade materials and unbound granular materials be treated as cross-anisotropic and bound materials such as asphalt and cemented materials are treated as isotropic.

Poissons Ratio

When a sample of material is stretched in one direction, it tends to get thinner in the other two directions. Poisson's ratio (ν, μ), named after Simeon Poisson, is a measure of this tendency. Poisson's ratio is the ratio of the relative contraction strain, or transverse strain (normal to the applied load), divided by the relative extension strain, or axial strain (in the direction of the applied load). For a perfectly incompressible material deformed elastically at small strains, the Poisson's ratio would be exactly 0.5.

The Poisson's ratio is not usually determined for all the pavement material to be used and experience has shown that the following values may be reasonably adopted for design purposes -

Granular material - 0.35;

Cement treated material - 0.20.

Modulus

The Modulus (E) is a measure of the stiffness of the pavement material. It is defined as the ratio of the rate of change of stress with strain.

Pavement layers are subjected to repetitive loading so the Repeated Load Triaxial test is considered the most appropriate laboratory test procedure for measuring elastic modulus. Because of the difficulty of carrying out this test, it is infrequently done. In practice, the modulus is usually determined from the Californian Bearing Ratio (CBR) values obtained on the existing pavement material during the preliminary investigation or from Unconfined Compressive Strength (UCS) test results, the empirical relationships for each adopted by the *Austroads Pavement Design Guide* is used as follows:

$$E = 10 \times CBR$$

where E = Elastic modulus (MPa).

CBR = California Bearing Ratio (%).

and

$$E = k \times UCS$$

where E = Elastic modulus (MPa).

UCS = Unconfined Compressive Strength of laboratory specimen at 28 days (MPa).

k = typically 1000 to 1250, depending on laboratory testing practices.

The value of *k* adopted by the MRD, considering it's standard testing methods, is 1000.

Stress-Strain Relationships

The stress-strain relations for a cross-anisotropic material in a particular layer are:

$$\varepsilon_{xx} = (1/E_h)\,(\sigma_{xx} - \nu_h\,\sigma_{yy} - \nu_{hv}\,\sigma_{zz})$$

$$\varepsilon_{yy} = (1/E_h)\,(-\,\nu_h\,\sigma_{xx} + \sigma_{yy} - \nu_{hv}\,\sigma_{zz})$$

$$\varepsilon_{zz} = (1/E_v)\,(-\,\nu_{vh}\,\sigma_{xx} - \nu_{vh}\,\sigma_{yy} + \sigma_{zz})$$

$$\varepsilon_{xy} = ((1+\nu_h)/E_h)\,\sigma_{xy}$$

$$\varepsilon_{xz} = (1/f)\,\sigma_{xz}$$

$$\varepsilon_{yz} = (1/f)\,\sigma_{yz}$$

The moduli and Poisson's ratios are related by the following equation:

$$\nu_{vh}/E_v = \nu_{hv}/E_h$$

The condition that the strain energy must be positive imposes restrictions on the values of the elastic constants:

$E_h > 0$ $\qquad E_v > 0$ $\qquad f > 0$

$1 > \nu_h > -1$ $\qquad 1-\nu_h-2\nu_{hv}\nu_{vh} > 0$

To be able to model a cross-anisotropic material you need to specify five constants: the vertical Elastic modulus (E_v), the horizontal Elastic modulus (E_h), the Poisson's ratio (ν_{vh}), the Poisson's ratio (ν_h) and the Shear modulus (f). The data values for all five constants are rarely available. The Austroads Pavement Design Guide uses the following simplifications to model subgrade and unbound granular materials:

$$E_h = 0.5\,E_v$$

$$\nu_{vh} = \nu_h = \nu$$

$$f = E_v/(1+\nu)$$

In this case, the material is defined simply by the vertical Elastic modulus, E_v, and a single Poisson's ratio, ν.

Source: *MINCAD Systems (2004)*

Isotrophy

For isotropic materials the restrictions become:

$$E > 0 \qquad\qquad 0.5 > \nu > -1.0$$

For isotropic materials, only the Elastic modulus and Poisson's ratio need to be entered, as they are assumed to be the same in all directions.

Source: *MINCAD Systems (2004)*

5.2 The CIRCLY Pavement Design Program

Input Parameters

The main input parameters required for entry into CIRCLY are outlined below.

Project Reliability

Project Reliability is the probability that the pavement when constructed to the chosen design will outlast its Design Traffic before major rehabilitation is required. This allows for uncertainty in the estimate of traffic growth and loadings, variation in material properties, construction variability and the importance of the road itself. Typical values used for the design of roads based on its AADT are shown in Table 5.1.

Table 5.1 - Recommended Project Reliability Values

Road Class	Project Reliability (%)
Freeway	95 - 97.5
Highway: lane AADT > 2000	90 - 97.5
Highway: lane AADT < 2000	85 - 95
Main Road: lane AADT > 500	85 - 95
Other Roads: lane AADT < 500	80 - 90

Standard Axle Repetitions (SARs)

The design "Standard Axle Repetitions" (SARs) are calculated as outlined in the Austroads Pavement Design Guide for each of the three failure modes - asphalt fatigue, cemented fatigue and subgrade rutting.

CIRCLY calculates a forecast of the failure repetitions for each of the failure modes and compares these values with the expected number of repetitions over the design life of the road pavement. The output can be expressed as a percentage of the design repetitions that will be achieved by the entered design parameters. Values less than 100% indicate that the pavement will not last for the design life, while more than 100% indicate "over-design".

The design period and annual growth rate are optional traffic if the user wishes the comparisons to also be expressed in years.

Pavement Composition and Properties

The material properties for each layer of the trial pavement are entered - thickness, moduli, Poisson ratio and whether the material is cross-anisotropic or isotropic.

For unbound granular material, sub layering is required. The Austroads Pavement Design Guide (2004) uses 5 equally thick sub-layers. The procedure is:

a) Divide the total depth of the unbound granular layer into 5 equally thick sub-layers.

b) The vertical modulus of the top of the sub-layer is the minimum of the value specified in the CIRCLY input and determined using:

$$E_{V\ \text{top sub-layer}} = E_{V\ \text{subgrade}} \times 2^{(\text{total granular thickness}/125)}$$

Example: $E_{V\ \text{top sub-layer}} = 100\ \text{MPa} \times 2^{(250/125)}$

$$E_{V\ \text{top sub-layer}} = 400\ \text{MPa}$$

c) The ratio of modulus of adjacent sub-layers is given by:

$$R = \left(\frac{E_{\text{top granular sub-layer}}}{E_{\text{subgrade}}}\right)^{\frac{1}{5}}$$

Example: $R = \left(\frac{400\ \text{MPa}}{100\ \text{MPa}}\right)^{\frac{1}{5}}$ so $R = 1.32$

d) The modulus of each sub-layer may then be calculated from the modulus of the adjacent underlying sub-layer, beginning with the known subgrade modulus, Table 5.2 is a typical example.

Table 5.2 - Sub-layering Example

Sublayer	Thickness (mm)	Modulus (MPa)
1	50	400
2	50	303
3	50	230
4	50	174
5	50	132

Bound layers within the pavement configuration do not need to be sub-layered.

Axle Loads

The option exists to select the tyre contact stress for a standard axle. Austroads Pavement Design Guide uses a contact pressure of 750 kPa, however if WIM data is available the contact pressure can be altered to reflect actual loading.

5.3 CIRCLY Design Example

As an example, suppose we have a pavement rehabilitation design where the remaining existing pavement over the subgrade comprises two layers of thickness 110 mm and 105 mm respectively, and a top insitu stabilised layer of 200 mm modified with 2% GB cement. A standard thin bituminous seal is to be used.

- The subgrade tested to a CBR of 5%, the existing road sub-base layers tested to a CBR of 8% and 18% respectively, and the stabilised layer gave a UCS of 0.7 MPa.
- The current traffic volume (AADT) is 2000 vpd and a forecast growth of 11%, producing a design SAR of $1.1x10^6$ for rutting failure for the 10 year design life. The other modes of failure are not relevant.
- From Table 5.1, a Project Reliability of 95% is chosen as the road is an important commercial access highway.

To calculate the forecast life using CIRCLY, the following steps are required.

- Enter project identification information and the project reliability.
- Enter the SAR for the relevant failure modes, in this case only the subgrade rutting failure SAR - $1.1x10^6$. Optionally, also enter the desired design life period (10 years) and the growth rate.
- Enter data for each layer, starting at layer 1 as the stabilised layer through to layer 4 as the subgrade. As the stabilised layer has only 2% cement additive, it is classified as "modified" and treated as a granular layer.

Table 5.3 - CIRCLY Data Entry

Layer	1	2	3	4
Description	Top Layer 2% Stabilised	Upper existing	Lower Existing	Subgrade
Granularity	Granular	Granular	Granular	Subgrade
Isotropy	Aniso	Aniso	Aniso	Aniso
Modulus MPa	700	186	88	56
Poissons Ratio	0.35	0.35	0.35	0.45
Interface	Rough	Rough	Rough	Rough

After this data is entered, the Calculate button is clicked for the program to calculate the forecast failure repetitions.

- The failure repetitions are displayed on-screen, in this case only the rutting failure SAR of $1.92x10^6$ is applicable. This is greater than the desired design SAR, hence the design is adequate.
- A report can be printed which also estimates the life to failure based on the growth rate entered.

5.4 Falling Weight Deflectometer

The Falling Weight Deflectometer is becoming the standard tool world-wide used for the non-destructive testing of pavements largely replacing the Beckleman Beam test procedure used previously. The test involves the recording of the deflection response during the dynamic loading of the pavement. The rebound deflection levels give an indication of the structural condition of an existing pavement.

The Falling Weight Deflectometer test rig comprises a load unit, a beam carrying the deflection measuring units and computer hardware to control and measure the loads and deflections. They are typically mounted on a trailer as shown in Figures 5-1 and 5-2.

Figure 5-1 - Falling Weight Deflectometer Trailer

Figure 5-2 - Falling Weight Deflectometer Loading Unit

When plotted, the deflection caused by the applied load results in a deflection bowl shape. The steepness of the bowl near the position of maximum deflection reflects the stiffness of the pavement base with weak bases having steep slopes while stiff bases have flat slopes. An indication of subgrade strength is achieved from deflections recorded at approximately 1 metre away from the position where the load was applied, high deflections indicate weaker subgrades.

It is Main Roads practice to define a bowl by the deflection level at the point of maximum deflection, designated D_0, and at a series of distances from the maximum of the bowl - 200 mm, 300 mm, 450 mm, 600 mm, 900 mm and 1500 mm. All bowl deflections are measured from a zero datum as indicated in Figure 5-3.

The falling weight deflectometer has the ability to vary load levels from 40 kN to 100 kN in 20 kN increments. This enables tests to be carried out with loads representative of the equivalent standard axle load and also with other load levels which may be more representative of in-service conditions. For the pavements investigated in this project, testing load levels of 40, 60 and 80 kN were used for each test location. The 40 and 60 kN deflection bowls provide an indication of the stress dependency of unbound granular layer moduli and enables interpolation to provide a 50 kN deflection bowl which is equivalent to the 750 MPa pressure used for design purposes for an equivalent standard axle.

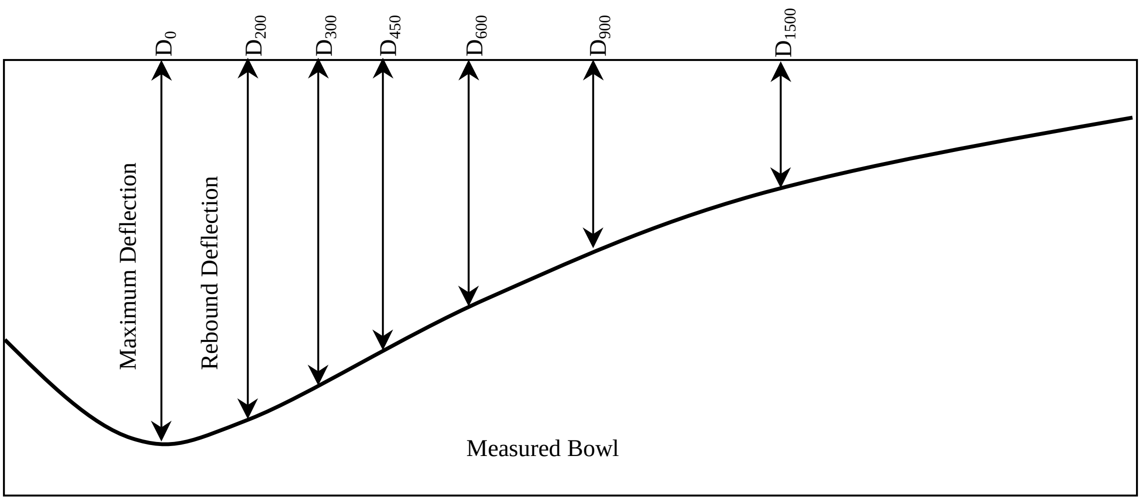


Figure 5.3 - Typical Falling Weight Deflectometer Deflection Bowl

An 80 kN load was also used because when testing stiffer pavements, the deflection levels are generally smaller, and the higher load, although not strictly representative of highway loadings, produces deflection levels of sufficient magnitude for back analysis without affecting estimates of subgrade and bound layer moduli.

A typical test regime would consist of two settling load applications of 40 kN, then a sequence of three applications at 40, 60 and 80 kN respectively. The applied loads and the resulting deflections are recorded in a computer file. The unit is also able to measure the time taken for the deflection wave to reach the sensors, from which an estimate of the subgrade modulus can be obtained.

5.4 The CIRCDEF Falling Weight Deflectometer Program

To analyse the deflection data, CIRCDEF, a CIRCLY based iterative back analysis program, is used to read the required inputs from a data file set up prior to execution. The user sets parameters in the file by specifying a keyword followed by its appropriate value. Parameters which may be set are presented in Table 5.4.

Table 5.4 - CIRCDEF Keywords for Data Entry

Keyword	Description
E	List of the start moduli for each layer. These values are used as the moduli for each layer in the initial iteration by CIRCDEF and for subsequent iterations if the layer is not variable.
EMAX	List of the maximum modulus value for each layer.
EMIN	List of the minimum modulus value for each layer.
HH	List of the thicknesses of each layer (0 may be specified for the last layer to indicate semi-infinite).
ILV	A list of the layer numbers of the variable layers.
KPA	Pressure applied at the load location.
LS	Distance between each of the two circular wheel loads.
LT	List of flags indicating each layers type: T for treated, G for granular and S for subgrade
MAXIT	Maximum number of iterations to be performed.
ND	Number of deflection points to be input ($2 \leq ND \leq 10$).
NL	Number of variable layers ($1 \leq NS \leq 4$). Number of layers for which a

Keyword	Description
	modulus value is to be calculated.
NS	Number of layers in the pavement layers (1≤NS≤8).
RR	List of the distances from the load centre to the measurement position.
RRD	List of the deflections observed at the points specified.
TOL	Tolerance of the fit (Maximum absolute sum of the percentage error in an acceptable solution).
V	List of the value of Poisson's Ratio for each layer.
WGT	The load at each locations (kN)

A typical input file is shown in Table 5.5.

Table 5.5 - Typical CIRCDEF Data Entry File

```
S1L
CORECT 0
ND 7
RRD 1.155 0.918 0.756 0.562 0.403 0.240 0.123
NL 3
TOL 5
MAXIT 20
ILV 1 2 3
EMIN 10 10 10
EMAX 6000 6000 6000
WGT 25.45
PSI 360.00
LS 0.1
NS 3
E 1500 1000 100
V 0.35 0.35 0.45
HH 200 200 0
LT G G S
RR 0 200 300 450 600 900 1500
$END
```

Generally, the depth of each layer (HH) is known from design and construction information, and a "first guess" of the moduli values (E) can also be made from design information. CIRCDEF predicts the deflections which would be obtained with the initial modulus values and determines differences and percentage errors between the "measured" and "predicted" deflections. From these values CIRCDEF calculates a new set of trial moduli values for the layers and repeats the deflection prediction. The iterations are continued until either (i) a suitable solution is found, (ii) the maximum number of iterations is reached or (iii) CIRCDEF detects an inconsistency in the system. When one of these conditions is reached, the “best fit” moduli are output along with the terminating condition.

A typical screen output is shown in Table 5.6. The first few lines of the output echo the input parameters. Details of the deflections predicted by CIRCLY follow, together with differences and percentage errors between the “measured” and “predicted”. Outputs for each iteration follows until complete. Normal practice is to restrict the number of iterations to about seven, so that the operator can manage the process and make manual adjustments if the iterations are not converging. The process is repeated a number of times if necessary until a close match of the full deflection bowl is achieved.

In this example, the deflections were matched within the 5% tolerance after four iterations, and the program exited, giving final values for the moduli as:

Base Layer:	1220	MPa
Sub-base Layer:	31	MPa
Sub-grade:	80	MPa

Table 5.6 - Typical CIRCDEF Data Output

```
S1L

NUMBER OF VARIABLE LAYERS  =  3
NUMBER OF LAYERS IN SYSTEM =  3
NUMBER OF TARGET DEFLECTIONS =  7

    DEFLECTION READINGS IN MM.

POSITION NO:          1       2       3       4       5       6       7
DEFLECTIONS:       1.1550  0.9180  0.7560  0.5620  0.4030  0.2400  0.1230
WEIGHTING FACTOR:  0.866   1.089   1.323   1.779   2.481   4.167   8.130

   DETAILS OF VARIABLE LAYERS

  LAYER NO    SYSTEM    VALUE OF MAXIMUM   VALUE OF MINIMUM
             LAYER NO   VERTICAL MODULUS   VERTICAL MODULUS
     1          1            6000.0             10.0
     2          2            6000.0             10.0
     3          3            6000.0             10.0

   DETAILS OF LAYERED SYSTEM

 LAYER NO   VERTICAL  POISSONS RATIO    THICKNESS        LAYER TYPE
             MODULUS
    1         1500        0.350          200.00      CROSS-ANISOTROPIC
    2         1000        0.350          200.00      CROSS-ANISOTROPIC
    3          100        0.450      SEMI-INFINITE   CROSS-ANISOTROPIC

   DETAILS OF LOADS

  LOAD TYPE     RADIUS    REFERENCE    AVERAGE    LOAD/MOMENT    POWER
                            STRESS      STRESS    PER LOCATION
(1) VERTICAL  150.0091  0.3600E+00  0.3600E+00  0.2545E+05  0.0000E+00
    FORCE

   LOAD LOCATIONS

   LOAD          X               Y
    NO.
    1       0.0000E+00      0.0000E+00
    2       0.1000E+00      0.0000E+00

  POSITION     DEFLECTION       MEASURED     DIFFERENCE       %.DIFF
     1          0.492420        1.155000      0.662580         57.4
     2          0.383019        0.918000      0.534981         58.3
     3          0.337779        0.756000      0.418221         55.3
     4          0.286765        0.562000      0.275235         49.0
     5          0.244381        0.403000      0.158619         39.4
     6          0.179521        0.240000      0.060479         25.2
     7          0.106449        0.123000      0.016551         13.5
             ABSOLUTE SUM:                    2.126666     297.952679
            ARITHMETIC SUM:                                297.952679

PREDICTED MODULI AT ITERATION  1.
 1427.91   62.96   98.55
```

Table 5.6 (cont.) - Typical CIRCDEF Data Output

POSITION	DEFLECTION	MEASURED	DIFFERENCE	%.DIFF
1	0.849426	1.155000	0.305574	26.5
2	0.654477	0.918000	0.263523	28.7
3	0.539762	0.756000	0.216238	28.6
4	0.400768	0.562000	0.161232	28.7
5	0.300201	0.403000	0.102799	25.5
6	0.183914	0.240000	0.056086	23.4
7	0.101591	0.123000	0.021409	17.4
		ABSOLUTE SUM:	1.126861	178.738002
		ARITHMETIC SUM:		178.738002
		AVERAGE:	0.1610	25.5340

PREDICTED MODULI AT ITERATION 2.
1413.07 17.65 77.21

POSITION	DEFLECTION	MEASURED	DIFFERENCE	%.DIFF
1	1.291121	1.155000	-0.136121	-11.8
2	1.050815	0.918000	-0.132815	-14.5
3	0.890191	0.756000	-0.134191	-17.8
4	0.673970	0.562000	-0.111970	-19.9
5	0.500789	0.403000	-0.097789	-24.3
6	0.280155	0.240000	-0.040155	-16.7
7	0.127170	0.123000	-0.004170	-3.4
		ABSOLUTE SUM:	0.657212	108.314219
		ARITHMETIC SUM:		
		AVERAGE:	0.0939	15.4735

PREDICTED MODULI AT ITERATION 3.
1251.32 30.55 79.22

POSITION	DEFLECTION	MEASURED	DIFFERENCE	%.DIFF
1	1.157104	1.155000	-0.002104	-0.2
2	0.912057	0.918000	0.005943	0.6
3	0.758597	0.756000	-0.002597	-0.3
4	0.563523	0.562000	-0.001523	-0.3
5	0.416488	0.403000	-0.013488	-3.3
6	0.241841	0.240000	-0.001841	-0.8
7	0.124799	0.123000	-0.001799	-1.5
		ABSOLUTE SUM:	0.029294	7.020441
		ARITHMETIC SUM:		-5.725770
		AVERAGE:	0.0042	1.0029

PREDICTED MODULI AT ITERATION 4.
1217.55 30.69 80.39

POSITION	DEFLECTION	MEASURED	DIFFERENCE	%.DIFF
1	1.160884	1.155000	-0.005884	-0.5
2	0.910954	0.918000	0.007046	0.8
3	0.755287	0.756000	0.000713	0.1
4	0.558465	0.562000	0.003535	0.6
5	0.411065	0.403000	-0.008065	-2.0
6	0.237595	0.240000	0.002405	1.0
7	0.122898	0.123000	0.000102	0.1
		ABSOLUTE SUM:	0.027749	5.086512
		ARITHMETIC SUM:		0.065377
		AVERAGE:	0.0040	0.7266

CHAPTER 6 – TEST SITES and DATA

6.1 Selection of Test Locations

Compaction density test results for nine MRD Mackay District road rehabilitation projects carried out over the last eighteen months were documented by staff of the MRD Materials Testing Laboratory in Mackay. Each project was divided into a number of "Lots" each of a size that could be reconstructed in a single day. Compaction density tests were carried out at a minimum of three random locations for each lot, making a total of 244 individual tests. These results (collated in Table D.1 of Appendix D) were assessed for suitability for inclusion in this project.

The main factors considered in selecting which test locations to include in the testing regime were as follows:

- Main Roads allocated one week of Falling Weight Deflectometer testing for this project, so fewer than approximately half can be tested;
- where practical, adjacent lots were selected to minimise FWD setup time;
- lots were selected to provide sets of results typical of the range of values experienced for all the projects considered; and
- the rehabilitation carried out was reasonably well defined and layer properties reasonably uniform over the road cross section so that meaningful comparisons can be obtained.

A total of 87 test locations were identified for FWD testing, satisfying the availability of the testing equipment and providing sufficient results for meaningful comparisons

Selection or elimination of each project was made as follows:

- Project 90/33A/806 - rehabilitate a 1.5 km section of the Peak Downs Highway. This project involved correction of the existing pavement with a nominal 50 mm corrector course layer, then insitu stabilisation to a depth of 200 mm with 2.0% general blend cement. The project is made up of two lots with 17 RDD tests. Only one test passed. Because of the range of results, all test locations are included in the FWD test regime.

- Project 120/33B/305 - Sandy Creek to Sawn Creek pavement rehabilitation on a 600 m section of the Peak Downs Highway. This project required shape correction of the existing pavement with a nominal 50 mm corrector course layer, then insitu stabilisation to a depth of 200 mm with 2.5% general blend cement. The resulting sub-base was then overlayed with a slurry mix and a 150 mm base overlay of Type 2.2 cement modified with 1% general blended material. The project is made up of two lots with eight RDD tests being performed on the insitu stabilised layer, approximately half passing the 100% specification requirement. However, most results were close to 100%, and because of the rehabilitation method used, this project was considered to be worth evaluating. All test locations are included for FWD testing.

- Project 107/517/301 - pavement rehabilitation on a 1.6 km section of the Sarina-Homebush Road. This project required a 100 mm overlay and insitu stabilisation to a depth of 300 mm with 3.0% general blend cement. The project is made up of four lots with 16 RDD tests being performed on the insitu stabilised layer, approximately 25% passing, and some tests with very low values. All test locations are included for FWD testing.

- Project 20/519/802 - overlay and rehabilitate a 1.2 km section of the Dysart - Middlemount Road. This project involved the shape correction of the existing pavement with the placement of a corrector course layer (averaging 50 mm), then overlayed with 100 mm of Type 2.2 material followed by insitu stabilisation to a total depth of 200 mm with 2% general blend cement. The project is made up of three lots with 10 Relative Dry Density (RDD) tests. Only 1 test passed the 100% RDD standard specification requirement. This is a typical treatment method for the average rural road and because of the range of results, all test locations are included in the FWD test regime.

- Project 82/533/304 - pavement rehabilitation on two sections of the Marian-Eton Road totalling 5.3 km. A nominal 75 mm corrector course was followed by insitu stabilising to depth of 200 mm with 3.0% cement. This project returned a wide range of test results - from 100% to 95% - and was expected to provide a good example for showing the wide range of results attained within a continuous construction regime. Four typical lots were chosen for inclusion, a total of 24 test locations.

- Project 20/85C/807 - rehabilitate a 710 m section of the Fitzroy Development Road. This project involved correction of the existing pavement with a nominal 50 mm corrector course layer, then insitu stabilisation to a depth of 200 mm with 2.0% general blend cement. The project is made up of two lots with 12 RDD tests and a 50% pass result. The results were highly variable and were expected to provide good comparisons within a localised area. All test locations are included in the FWD test regime.
- Project 90/514/201, involved the stabilisation of existing shoulder material to widen the formation to approximately two lanes of traffic. The material that was treated was not from an established pavement and there is no documented information. This project was rejected for further analysis.
- Project 90/33B/304 - rehabilitate a 5.8 km section of the Peak Downs Highway, subdivided into 14 lots. This project involved correction of the existing pavement with a nominal 75 mm corrector course layer, then insitu stabilisation to a depth of 250 mm with 3.0% general blend cement. A total of 65 tests were carried out on this project. The majority of the RDD tests failed over a range from 95% to 99%. However, the pre-rehabilitation test information and the pavement design report with the CIRCLY design information could not be found so that FWD analysis would not have been possible without excavating several new pits to determine the layer thicknesses and comparisons with the design figures are not possible. As there are other projects with similar RDD test profiles, this project was excluded from the test program.
- Project 82/533/303 is adjacent to Project 82/533/304 and involved similar rehabilitation work. All tests attained or exceeded the required 100 percent figure, and it is thought that the high quality of the topping layer eliminated a lot of material variability resulting in the good test results. Because of the consistency of results for this section, it is expected that the previously described Project 82/533/304 will provide a better variation of test results for the purpose of this investigation so the project and was eliminated from the list for FWD testing.

The FWD testing was carried out in August 2007.

For the purposes of this project and to simplify referencing, each of the adjacent groups of lots is considered to be a single site and identified by a Site ID number. Table D.2 in Appendix D identifies each site in relation to the MRD Job Number identification and MRD Road Number, together with the start and finish chainages of the lots within the project where the selected test locations are found. This is also shown diagrammatically in Figure D-1.

A summary of the tests for each of the selected lots is shown in Table D.3, identifying each lot in each site, the start and end chainages, and the characteristic value (CV) for each lot. A CV value less than 100% is grounds for rejection or for reducing the contract payment as a result of a reduction in service life.

Table D.4 shows a complete listing of the RDD tests carried out on the lots selected for FWD testing as part of this project. This listing and test identification codes will be used as the reference for comparing the FWD test results with the original construction test results.

6.2 Site Design Parameters and Results for Insitu Stabilisation

Table E.1 in Appendix E shows a summary of the pre-design investigation test data for each of the project sites taken from the MRD design reports for each of the MRD projects. The thickness of the sub-base comprises the original road base less the depth of cutting which occurred during reclaiming. The base layer comprises the reclaimed material together with any new material added to improve the base layer properties. The moduli of the subgrade and sub-base were calculated from average CBR values obtained during the field investigations. The moduli for the base layers were determined from 7-day UCS tests (MR Test Method – Q115C) on the design base mix with cement added, the percentage of cement to be added being such as to achieve a targeted moduli value.

The design traffic assumptions for each site are shown in Table E.2, including the number of equivalent standard axles estimated to be applied to the road over the preferred design life of 10 or 20 years.

The initial AADT figures (vehicles per day) and the percentage of heavy vehicles were estimated by the designer from available traffic count information. Where possible, traffic data counters were installed near the areas to be rehabilitated for a short period to provide updated data.

For each project, a traffic growth rate was estimated from available data and a knowledge of the economic activity expected in the region. Historical growth rates can be extracted from the computer program 'TARS' (Traffic Analysis and Reporting System) which contains traffic data from all traffic counts conducted on the road since the program was introduced in 1994.

The factor F_1 is a factor specified by MRD design standards based on computer analysis of data recorded by the three permanent weigh-in-motion stations within the district. The factor makes allowance for the fact that some vehicles are empty, some fully loaded (overloaded) and some partly loaded. The F1 values have increased with time as the size and axle configurations of vehicles have changed and axle loadings have increased. The value of F1 current for Mackay District is 3.2.

The cumulative growth factor (f) is calculated from the standard geometric progression formula to accumulate total counts were the growth rate is constant over the period.

$$f = (1 + 0.01i)\frac{(1 + 0.01i)^y - 1}{0.01i}$$

where i = Growth rate percentage

y = The number of years

Source: *Main Roads Design Manual (1990)*

The total number of equivalent standard axles expected for the appropriate design life is calculated by combining these factors as shown in column 8 of Table E.2.

Table E.3 in Appendix E compares the number of ESAs to failure forecast by CIRCLY to the estimated number of ESAs which will occur during the desired design life of the rehabilitated pavement. As can be seen, the desired design life was only achieved for one of the six sites.

The general philosophy of the MRD, at least in the Mackay District, is to target a minimum of a twenty years for the design life of a new construction road. However when roads are being rehabilitated, the design is impacted by numerous pressures including the availability of funds and the limitations of the machinery used for the rehabilitation.

Consequently, a ten year forecast design life is generally considered to be acceptable for the majority of rehabilitated pavements, and in some situations a shorter life may be adopted as an interim low cost emergency repair to badly failed pavements until a more permanent repair can be carried out. Site 1 illustrates this situation, whereas most of the other sites exceed a ten year life by some margin.

The background leading to the requirement to rehabilitate each of the sites is briefly detailed below.

Site 1 – This section of the Peak Downs Highway had the shoulders widened with poor quality gravel, which was allowing water to infiltrate to the expansive subgrade material below. This was causing serious distress with rutting in the outer wheel path in excess of 50 mm and major cracking appearing. An interim emergency treatment was recommended in an attempt to bridge the poor subgrade and improve the formation for safety reasons.

Site 2 – The existing pavement in this section of the Peak Downs Highway was in poor condition for the full length of the project with extensive patching, large scale pavement repairs and rutting. The average rut depth was approximately 8 mm with an average maximum rut depth of 30 mm. This is a relatively heavily trafficked road and required constant maintenance after rain, so that rehabilitation was considered to be an appropriate medium term solution.

Site 3 – This section of the Sarina-Homebush Road was badly deformed and over the previous 5 years had required excessive maintenance treatments. The pavement depth was insufficient to cater for the heavier traffic now using the road, causing the movement of the subgrade material to be reflected through to the base layer. The preliminary investigation showed that the quality of the existing material was relatively good and the addition of a 100 mm overlay layer and insitu cement stabilisation would be adequate to bridge the subgrade material.

Site 4 – This existing pavement section on the Dysart-Middlemount Road was out of shape with depressions and high spots. Numerous pavement repairs had been carried out on this section of the road and edge drop-off was evident on all shoulders. A corrector course layer (averaging 50 mm) was required for shape correction, followed by a 100 mm overlay of Type 2.2 material to improve the strength and grading of the material was required before insitu stabilisation to a total depth of 200 mm with 2% general blend cement.

Site 5 – The Marion-Eton Road is the designated heavy vehicle bypass road for traffic from Mackay to the mines. The ability to accommodate the heavy and oversized loads for the transportation of mine equipment was an important consideration in the decision to reconstruct this section, and during the design. Not originally built for heavy machinery transport, many sections are developing outer wheel path rutting, and there is a program to upgrade much of this road in the coming years.

Site 6 – Routine pavement and ride quality testing on the Fitzroy Development Road identified that this section had deteriorated and roughness and ride quality was outside the limits for this major access to Middlemount and Dysart. Shape correction was achieved with a nominal 50 mm corrector course layer, which was then insitu stabilised to a depth of 225 mm with 2.0% general blend cement.

CHAPTER 7 – INSPECTIONS and TESTING

7.1 Safety Issues - Risk Assessments

The main risk activities associated with this project were the visual inspection of the sites and the falling weight deflectometer testing. The risk assessments attached in Tables C.1 to C.4 in Appendix C – Working Outdoors, Working in Traffic (visual inspection and FWD testing) and Operating FWD Tester were carried out in a group session with the team involved to identify the potential hazards and detail control measures to reduce the risk of any potential hazards.

The greatest hazard identified was working in traffic, therefore the visual inspections and falling weight deflectometer testing were performed in accordance with the Department of Main Roads, Queensland, Manual of Uniform Traffic Control Devices – 2003 (MUTCD) to ensure the safety of the public and personnel performing the testing.

The visual inspection was performed in accordance with Clause 4.8.2 – Working Between Gaps in Traffic. This clause allows short duration works to be carried out without signs and delineation provided that a lookout person is posted, the work vehicle is parked clear of moving traffic and vehicle mounted flashing lights are operating.

The falling weight deflectometer testing was performed in accordance with Clause 4.9 – Mobile Works. The testing requires the test trailer towed by a vehicle to move along the roadway at a slow speed stopping to perform the test for approximately 35 seconds thereby obstructing a traffic lane. Advance warning signage and speed reduction signage were erected covering a maximum permissible testing distance of 2 km. The testing was performed in a convoy arrangement with a lead vehicle warning approaching traffic, the testing vehicle, a shadow vehicle close behind and a tailing vehicle further back. Personnel within the lead and tail vehicle performed the stop-slow traffic control operations whilst the test was being performed. An example of the traffic arrangement diagram is attached in Appendix C – Figure C-1.

7.2 Visual Inspections

The pavement surfaces were inspected for any signs of distress or deterioration over the complete length of each site, as well as in any adjacent lots which were excluded from testing. The inspections searched for any signs of rutting, cracking, localised depressions, edge failure and aggregate loss on the bitumen surface, the modes of distress which were described in Chapter 2. The presence of any of these distress modes to any significant extent would indicate that the underlying layers have failed or are starting to fail, and consequentially the expected service life of the pavement may not be achieved. The results of the inspection for each site are detailed in Appendix E – Table E.4.

Rutting, depressions and potholes were looked for visually and measured by the deviation of the pavement from a straight edge laid across the lane. Where there were no obvious visual signs of failure, sample spot checks were made using the straight edge to confirm the visual indications. Any cracking, edge failure and aggregate loss were inspected by eye while traversing the complete length of the section. Failures of these types are recorded with the start and finish chainage and an approximation of area covered.

Considering the short time that has elapsed from reconstruction, (eighteen months or less), it was not expected that major failures would be observed, although small localised failures could occur due to the possible variability of the reconstituted material. As can be seen from Table E.4, the pavements are generally showing no signs of distress or failure. No potholing was observed, but some longitudinal cracking on the shoulder and rutting had occurred in small sections of Site 1 and Site 3 and some minor edge cracking in Site 6.

The forecast design life for Site 1 was 2 years, and the pavement had been in service for approximately eighteen months. It was expected that of all the sites studied this site was the most likely to be showing signs of distress. However apart from some defects in isolated sections the pavement seemed to be in good condition with only three small sections exhibiting longitudinal cracking near the edge line.

Only one of these had a prominent longitudinal crack approximately 17 m long - Figure 7-1(a). In the other two sections, a fine crack was only beginning to reflect through - Figure 7-1(b). From observations of the location of the cracks in relation to the wheel tracks and the type of soil in the surrounding country, it was considered that this cracking was more likely to be a result of movement of the underlying expansive subgrade and not so much a failure of the mechanical strength of the pavement itself.

A repaired area where stripping of the seal exposing the base layer was observed over a 30 metre section of this site in the outer wheel track. However, this was caused by an error in calculating the application rate of the seal binder and was not an indication of pavement failure.

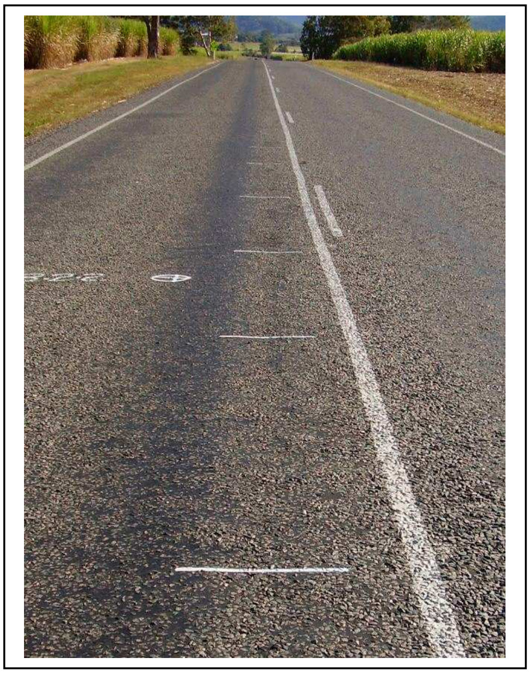
Overall, in spite of the observed cracking, the pavement of Site 1 has performed well, and apart from some minor repairs being required, it is expected that the pavement will exceed the forecast design life by some margin.

(a) Longitudinal Crack at edge line

(b) Longitudinal Crack beginning to reflect through pavement

Figure 7-1 - Longitudinal Cracking – Site 1

One prominent rut approximately 19 m long was observed near the centre line in Site 3. As the rut was near the centre line, outside the vehicle wheel path, this may not indicate a general failure of the pavement, but rather the result of a localised poor quality mix of material within the pavement at this spot – Figure 7-2(a) and 7-2(b).



(a) Rutting near centreline

(b) Rutting measurement

Figure 7-2 - Rutting – Site 3

Site 6 showed single longitudinal cracking at the edge of the shoulder in three different locations. The cracks were each approximately 15 metres long and approximately 300 mm from the pavement shoulder edge. This longitudinal cracking appeared to be a result of ingress of water from the shoulder, weakening the shoulder pavement material rather than traffic induced pavement failure – Figure 7-3.

Figure 7-3 - Cracking – Site 6
At edge of shoulder

All sites except Site 6 exhibited the signs of bitumen bleeding which often occurs during the heat of the summer months. Bleeding shows up as a flushing of the bitumen to the surface of the seal aggregate. Figure 7.4 shows a typical example of the bleeding that occurred at these sites.

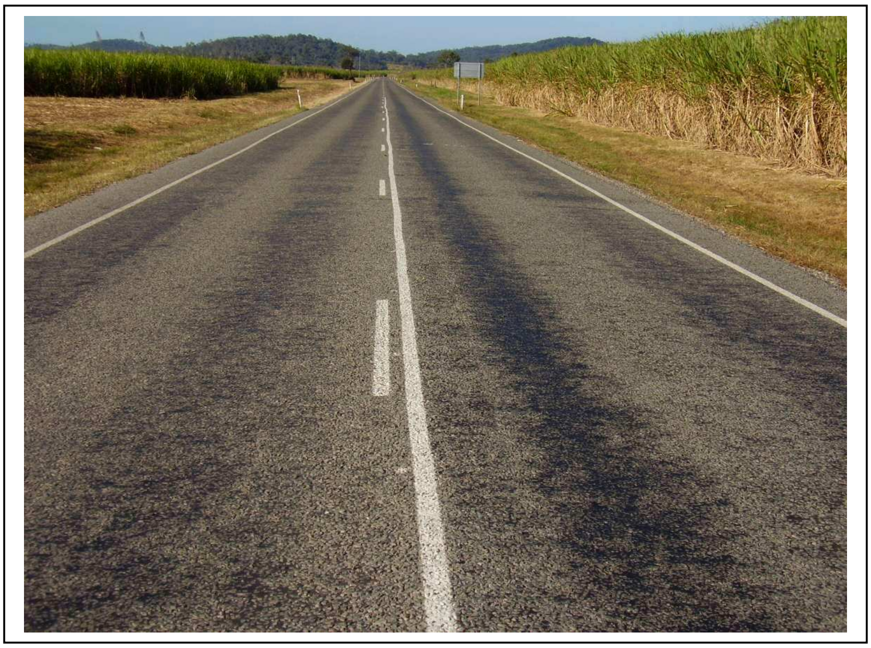


Figure 7-4 - Flushing of Bitumen Inner and Outer Wheel Paths

Although this flushing is pronounced, it has little effect on the service life of the pavement, although maintenance of the surface is sometimes required if the surface "strips" with vehicular traffic. In fact, the flushing occurred in these pavements that have been coarse-sealed (16 mm aggregate during construction) and are due for routine re-sealing with a finer aggregate (10 mm) after approximately two years in service. This re-surfacing will restore the surface appearance and re-establish the wearing surface.

7.3 Falling Weight Deflectometer Testing

Two sets of Falling Weight Deflectometer Testing was conducted on each of the six project sites.

The first set was a set of site specific tests conducted at the chainage where the RDD construction quality control tests were carried out, as listed in Table D.4 - Acceptance Test Data – Selected Sites in Appendix D.

The second set comprised a series of sequential tests at 50 m or 100 m intervals in the outer wheel path on both the out-going and in-going lane. The outer wheel path was chosen because it reflects the worst case scenario of having direct tyre passes and its close proximity to the shoulder edge where water is likely to infiltrate the pavement.

7.4 Site Specific FWD Test Results

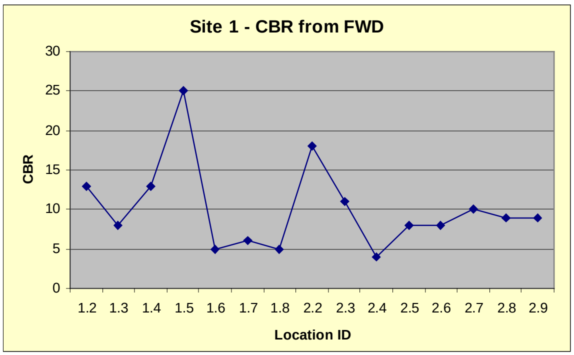
The raw data measured by the FWD for the site specific tests is listed in Table F.1 (a) - (l) of Appendix F. A test sequence at each location involved a single 600 kPa drop to settle the test area, followed by two measured drops at 600 kPa, then one at 850 kPa and one at 1100 kPa. With the drop plate used on this device these pressures are equivalent to nominal drops of 40, 60 and 80 kN.

The two drops of 40 kN are a legacy of the time when the standard equivalent axle was equivalent to 40kN and the software averages the two results. Currently a 50 kN application is equivalent to a standard axle and deflections for this value are interpolated from the 40 and 60 kN readings.

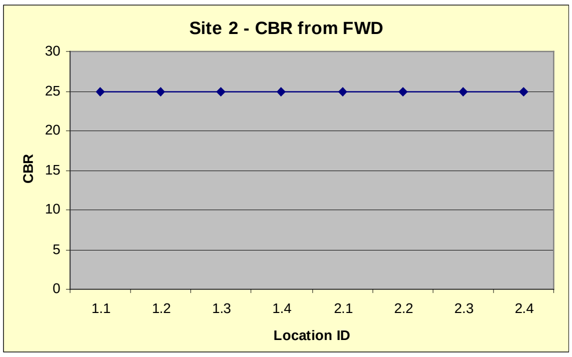
The 80 kN drop is used to provide greater deflection results for stiff pavements. This improves the accuracy of CIRDEF when the deflections are small.

The CBR values for the subgrade for each location obtained directly from the 40 kN FWD time response test are listed in Table G.1 – FWD Comparison Test Data – Selected Sites in Appendix G. These results are displayed in Figures 7-5(a)-(f).

Figure 7-5 - Subgrade CBR from FWD

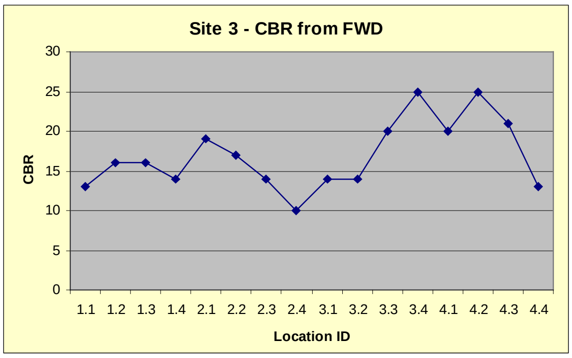


(a) Site 1

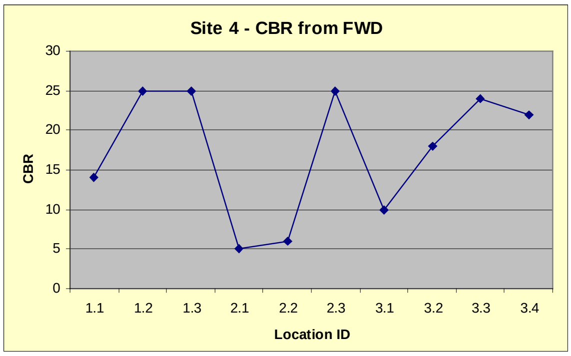


(b) Site 2

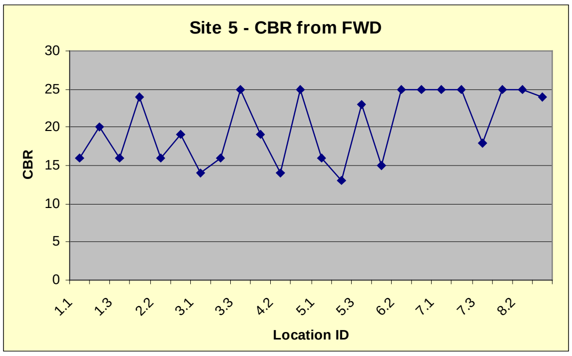
Figure 7-5 (cont.) - Subgrade CBR from FWD



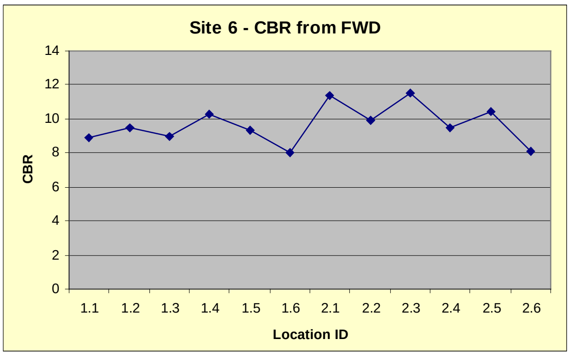
(c) Site 3



(d) Site 4



(e) Site 5



(f) Site 6

It should be noted that the subgrade strength test determined directly from the FWD test is internally restricted to a maximum of CBR 25. Hence a number of the CBR values shown in the graphs are displayed at 25 rather than at it's true value.

As can be seen, the values are quite variable, but generally exceed the values used for the subgrade strength used for the design of the stabilised pavement, validating the use of the subgrade design strength values. These results will be discussed in greater detail in Chapter 8

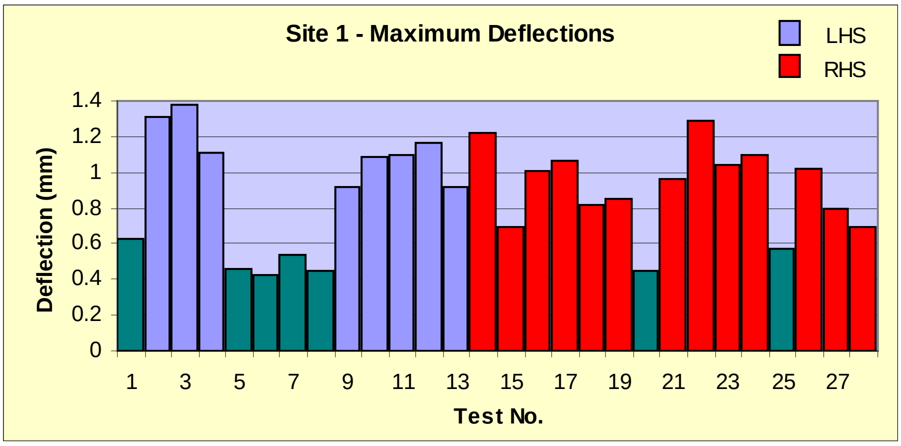
7.5 Sequential FWD Testing Results

The raw data measured by the FWD for the sequential tests are listed in Table F.2 (a) - (l) of Appendix F. The test sequence at each location was the same as for the site specific test sequence.

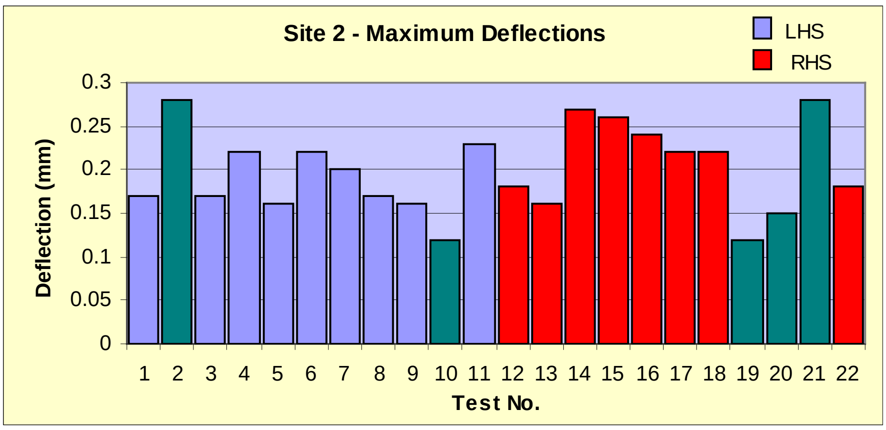
The sequential tests were performed to identify similar strength sections within each site and to enable the determination of the characteristic moduli for each similar strength section. It allows the statistical analysis of the deflection readings so that the mean strength and the spread of strengths can be determined for a more useful picture of the overall performance of the pavement, rather than individual spot values. The maximum deflections for the sequencing testing are displayed in Figures 7-6 (a) - (f).

To obtain realistic comparative moduli values along the length of the section, the deflection readings obtained must be perused to identify whether the results are reasonably consistent. Single very high or very low readings may indicate an inconsistency at that location such as a buried culvert or a previous patch where the pulverised material is not as identified during the pre-design testing regime. Groups of abnormal similar value deflections could indicate that the related section had been rebuilt to a different quality some time in the past, such as a truck stop area within the length of the site. The tests where abnormal readings appeared are shown in green on the charts in Figure 7-6 and will be discussed in more detail in Chapter 8.

Figure 7-6 - Maximum Deflections

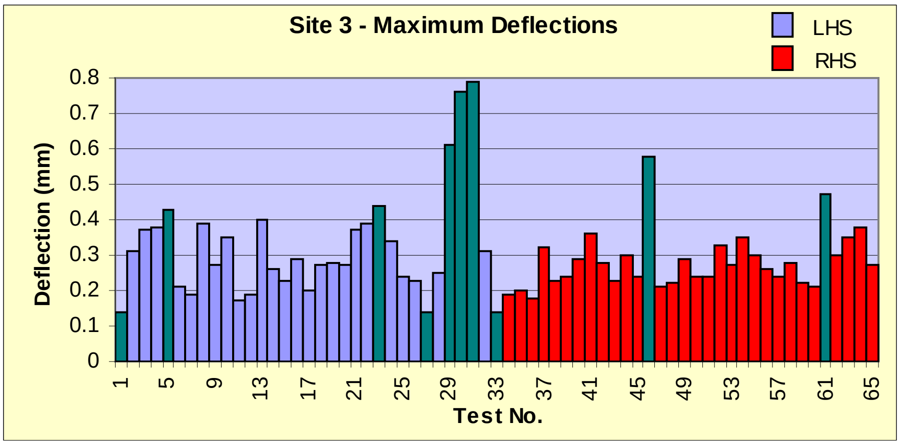


(a) Site 1

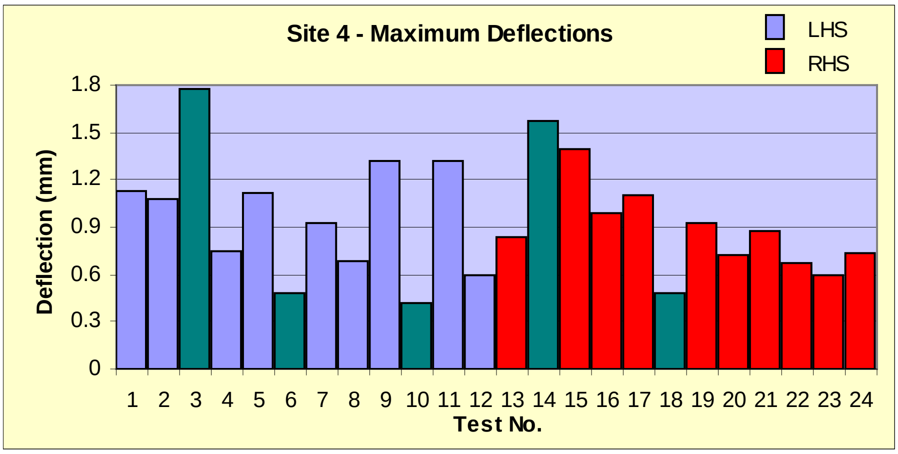


(b) Site 2

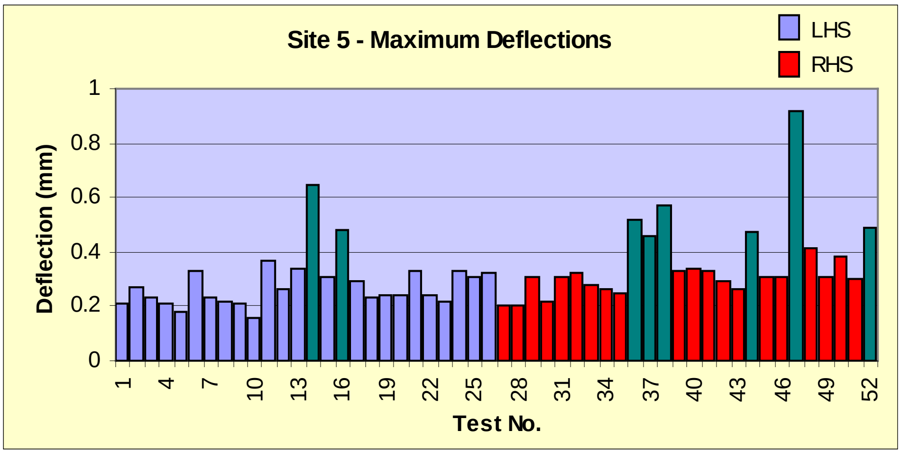
Figure 7-6 (cont.) - Maximum Deflections



(c) Site 3

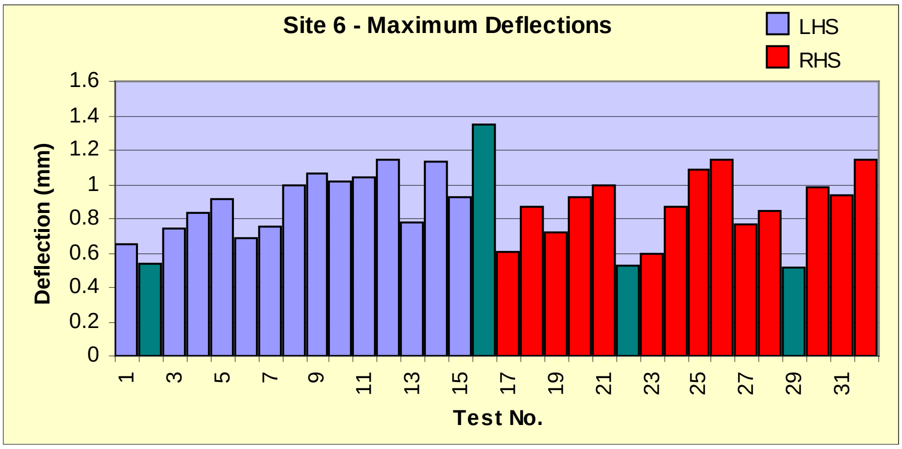


(d) Site 4



(e) Site 5

Figure 7-6 (cont.) - Maximum Deflections



(f) Site 6

CHAPTER 8 – ANALYSIS OF RESULTS

8.1 Reliability and Sensitivity

8.1.1 Falling Weight Deflectometer

Typical deflection bowl shapes obtained from the FWD tests are displayed below in Figure 8-1. The most common shape obtained during the tests is that shown as Series 4, with a constantly decreasing slope.

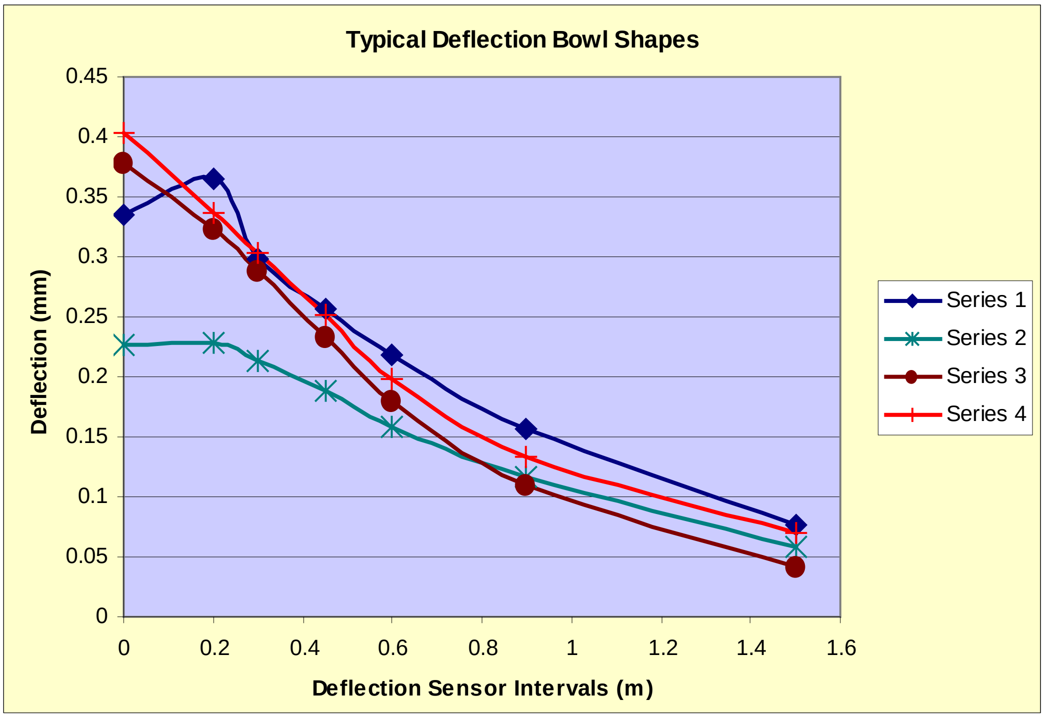


Figure 8-1 - Typical Deflections Results for Site 3

Perusal of the different shapes can provide a rough indication of the structure and strength of the pavement. Deflections measured by the sensors close to the point of application of the force are indicative of the strength of the top (base) layer, while the furthermost sensors tend to show a response related to the strength of the subgrade. Intermediate readings are indicative of the strength of the intervening layers.

Reasonably, higher deflections indicate a weaker material. Steep slopes at the first two or three sensors such as shown for Series 3 and 4 are the most common shapes returned, and indicate relatively strong top (base) and intermediate (subbase) layers on a relatively weak subgrade layer.

The initial flat slope shown by Series 2 indicates that the upper portion of the top (base) layer is strong enough to spread the maximum deflection out to the adjacent sensors, compressing the underlying layers for a distance until the normal decay takes over. This is shown to the extreme in Series 1 where the second deflection is actually greater then at the point of application.

The higher strength pavements produced maximum deflections typically as shown on the graph. Lower strength pavements showed higher maximum deflections, up to 1.6 mm in the worst case and generally followed the shape of the Series 4 graph, albeit generally with a sharper drop-off towards the middle sensors.

To obtain quantitative values for layer strengths, it is necessary to input the deflection data together with layer information including thicknesses into a suitable analysis program such as CIRCDEF – refer Chapter 5. These programs model the pavement structure and by iteration adjust the moduli values until a match with the experimental deflection bowl shape is achieved.

Expert advice regarding the use of FWD and CIRCDEF suggests that the reliability of the results from a single test is relatively low, but confidence in the results increases as a greater number of tests are analysed. The causes of the variability may be some or all of the following:

- although the deflection sensors are calibrated to better than 5%, the location and contact with the road surface may not allow the full accuracy to be achieved;
- local inconsistencies in the pavement at the impact site may produce inconsistencies in application of the force pulse;
- deflections are often very small compared with the maximum measurement range of the device, reducing the absolute accuracy of measurement;
- the layer thicknesses, particularly for a rehabilitated pavement, may vary significantly from the design value.

In order to estimate the sensitivity of the modulus calculation to variability in the assumed layer thicknesses and in the measurement of the deflection bowl, a number of CIRCDEF calculations were carried out on typical deflection bowls, with varying layer thicknesses and deflections.

The modulus for the actual deflection results for the Series 1, 3and 4 deflection lines above were calculated and are shown in Table 8.1.

Table 8.1 - Modulus Sensitivity to Thickness

Series	Thickness mm	%	Base Moduli	%	Subgrade Moduli	%
1	250	-17%	6243	+49%	113	+3%
	300	0	4148	-	110	-
	350	+17%	2950	-29%	108	-2%
3	250	-17%	2871	+42%	150	+3%
	300	0	2016	-	146	-
	350	+17%	1548	-23%	144	-1%
4	250	-17%	3186	+47%	135	+3%
	300	0	2167	-	131	-
	350	+17%	1665	-23%	127	-3%

The assumed variation in top layer thickness of 50 mm (a 17% change) significantly changes the calculated strength by between approximately 20% and 50%, but with little effect on the subgrade strength. It would be expected that variations of up to 25 mm for the pavement layers would possibly occur because the road is not a complete reconstruction and the existing material layer thicknesses are determined from a small number of pit excavations.

Similar calculations with a change in thickness of 25 mm showed a variation in the modulus of ± 14%.

For site 4 which had a base, subbase and subgrade pavement configuration, a number of scenarios were investigated. With the base thickness being altered by 10mm, and the subbase being altered by the same amount in the opposite sense, a change in moduli of about 8% was observed for the base and subbase. Similar tests conducted by altering thickness up to 55 mm provided changes in the base and subbase layers up to 30% – 40 %.

These figures suggest that on the average, an accuracy in the order of 10% – 20% may be expected, however when analysing the results, it needs to be remembered that wider fluctuations may occur.

8.1.2 CIRCLY

To evaluate the approximate sensitivity of calculations using CIRCLY, a typical pavement configuration was selected and the change in forecast ESAs were calculated for changes in layer thickness and modulus. Table 8.2 – Layer Thickness Variations displays the results for pavement failure by excessive compressive strain at the top of the subgrade by changing the top (base) layer and the subbase layer thicknesses.

The nominal 200 mm base layer thickness was varied by 20 mm (columns 2 & 3), a change of 10% which resulted in a change of approximately 26% in the number of ESAs to failure.

Table 8.2 - Layer Thickness Variations

Layer	**Modulus (MPa)**	**Layer Thickness (mm)**				
		1	2	3	4	5
Base	600	**200**	180	220	200	200
Subbase	298	**200**	200	200	180	220
Subgrade	5	**-**	-	-	-	-
ESAs to Failure ($x10^6$)		**2.97**	2.19	4.00	2.29	3.85
% Difference		-	-26	+26	-23	+23

Similarly, the subbase layer thickness was changed by 20 mm (columns 4 & 5) and the effects on pavement performance recorded. The variation of the ESAs to failure in this case was slightly less at 23%.

Table 8.3 – Layer Moduli Variations shows the results of changing the moduli strength of the base and subbase layers. Changing the modulus of the base layer by 50 MPa from the nominal 600 MPa – an 8% change – results in a 15% change in the number of ESAs to failure. Changing the modulus of the subbase layer by 50 MPa from the

nominal 300 MPa – a 16% change – results in a 15% change in the number of ESAs to failure.

Table 8.3 - Layer Moduli Variations

Layer	Layer Thickness (mm)	Modulus (MPa)				
		1	2	3	4	5
Base	200	**600**	550	650	600	600
Subbase	200	**298**	298	298	250	350
Subgrade	0	**5**	5	5	5	5
ESAs to Failure ($x10^6$)		**2.97**	2.52	3.48	2.55	3.50
% Difference		**-**	-15	+15	-15	+15

As may be expected, a variation in the base layer modulus has a larger proportional effect on the ESAs to failure than does a variation in the subbase layer modulus. Note also that at the typical vehicular traffic growth rate of 5% to 10%, a 15% change in the ESAs to failure will also change the forecast service life by typically 1 to 3 years.

It is apparent from the above analysis that unless great care is taken with determining the accurate parameters for the test site, the results for a single location are prone to significant errors, perhaps up to 40% - 50%, although the mean error would be expected to be substantially lower.

Confidence in the results for a pavement section are improved by carrying out sequential testing at 50 m or 100 m intervals, where a deflection profile can be established which aids in identifying rogue tests and areas where the pavement properties change significantly. As a result of experience during the course of this project with the FWD testing process and the application of CIRCDEF and CIRCLY to the results, together with discussions with experts in these fields, the author is confident that in spite of the potential for significant errors, the results obtained for this project are sufficiently accurate to allow meaningful conclusions to be drawn.

8.2 Site Specific Tests

An analysis of the deflections obtained at each site using CIRCDEF produced the modulus values listed in Table G.1 – FWD Comparison Test Data – Selected Sites in Appendix G.

It should be noted that tests at locations 1.1.1 and 1.2.1 were not conducted as site conditions precluded testing at that time. In addition, two test locations in Site 1 - locations 1.1.5 and 1.1.6 - have been removed from considerstion and will not be included in future discussions. The reason for the removal is that the results of the tests showed abnormal deflection readings, and on investigation, it was found that the original pavement in that area was substantially different from the design data, due to the previous construction of a truck parking area on both sides of the road. Hence, layer thicknesses were not as shown in the design data, the actual thicknesses were not able to be obtained, and modulus calculations would therefore be meaningless.

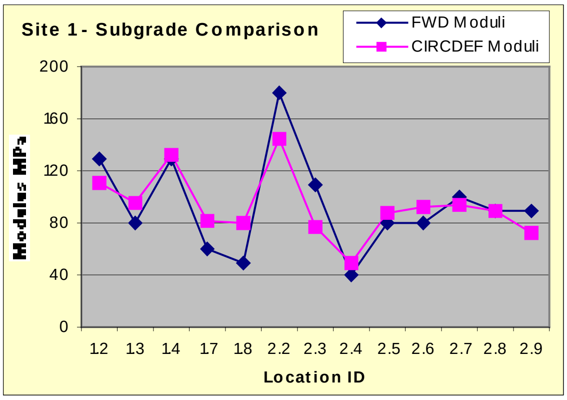
8.2.1 Subgrade Moduli

The calculated results obtained for the subgrade moduli are shown in Figure 8-2 (a) - (f). These values are shown compared with the values obtained directly from the CBR values given by the FWD tests as shown in Figure 7-5. Note that the modulus for the subgrade is obtained by multiplying the CBR values by 10.

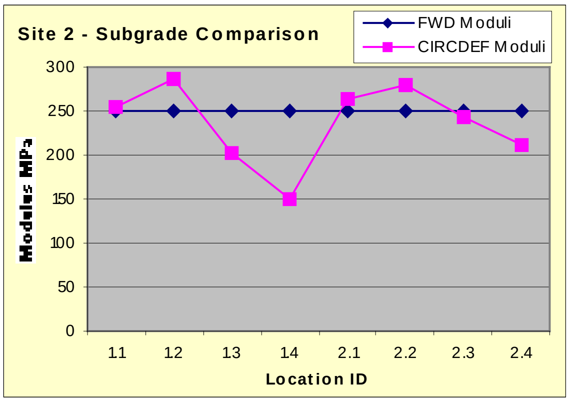
Note also that the results for subgrade strengths determined by the FWD deflectometer wave timing process are capped at a CBR value of 25, hence the subgrade strengths above 250 MPa are shown as 250 MPa. This is particularly noticeable for Site 2, but some readings for the other sites are also capped at this limit.

These results illustrate the difficulty of obtaining accurate moduli values for road pavements from deflection readings, at least where the insitu stabilisation process has been used for rehabilitation. However the results do indicate a general agreement between the two sets of values, albeit some with more agreement than others.

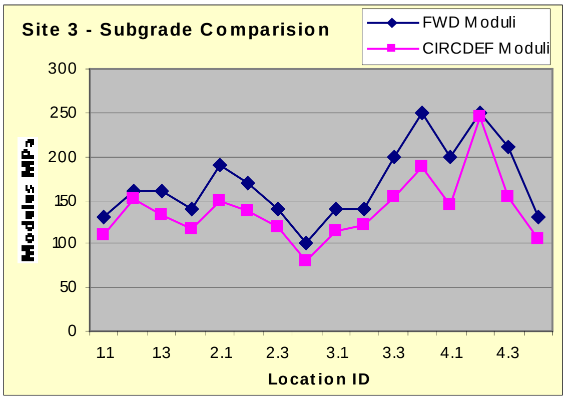
Figure 8-2 - Subgrade Modulus Comparison between FWD & CIRCDEF



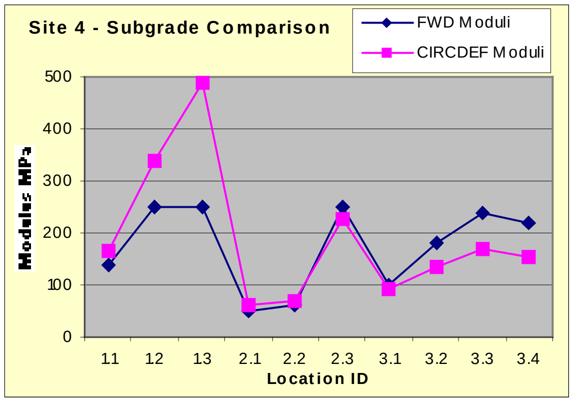
(a) Site 1



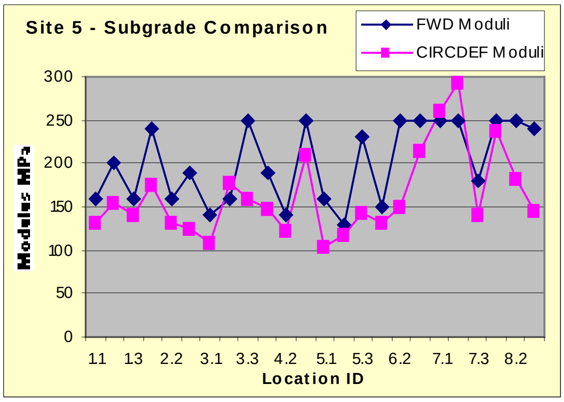
(b) Site 2



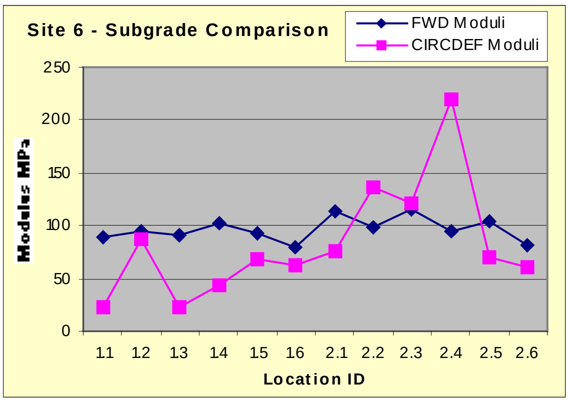
(c) Site 3



(d) Site 4



(e) Site 5



(f) Site 6

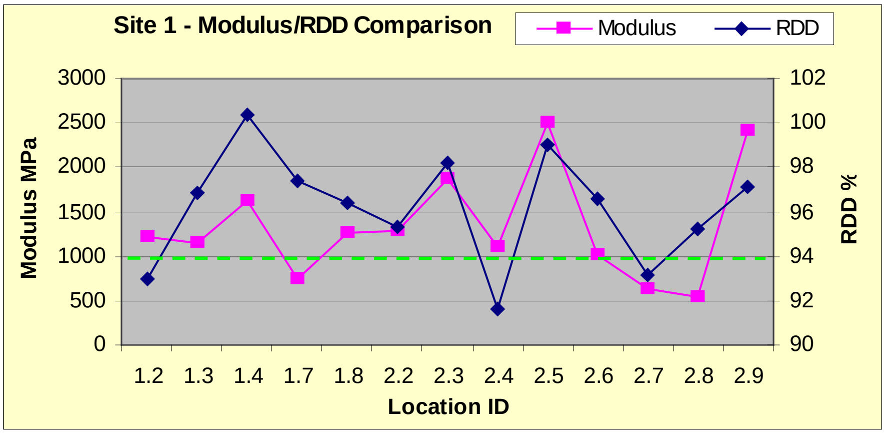
There is a relatively fair agreement for Sites 1, 3, 4 and 5, bearing in mind that the FWD values are capped at 250 MPa. No reason can be offered for the two low readings at locations 1.3 and 1.4 of Site 2 where the FWD values indicate moduli consistently above 250 MPa. Site 6 shows a reasonable correspondence in the centre ranges, but no explanation can be found for the very low readings at the start and the high reading at 6.2.4.

Generally, it may be considered that the comparisons are generally consistent bearing in mind the limited detailed information available for the original layer thicknesses and the variabilities inherent in the final layer thicknesses after reconstruction.

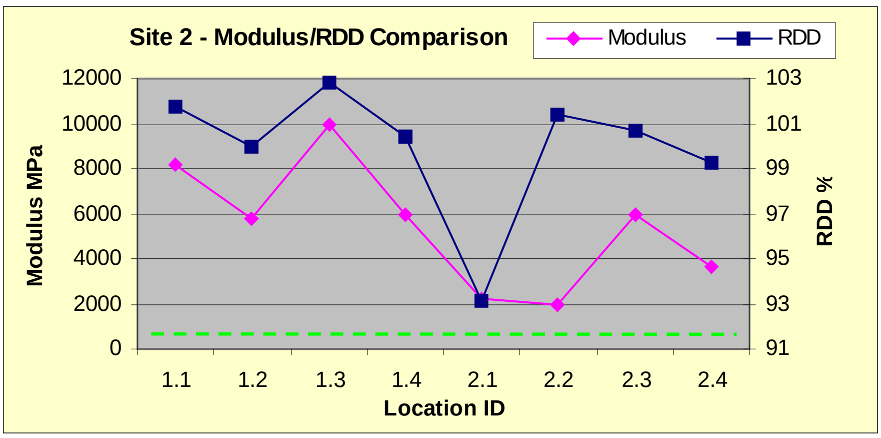
8.2.2 Stabilised Layer Moduli

The modulus values obtained using CIRCDEF for the insitu-stabilised layers for each of the site specific locations are shown in Figure 8-3, together with the field Relative Dry Density results obtained for that location during construction. The target design modulus is shown by the dashed green horizontal line.

Figure 8-3 - Modulus and RDD Comparison

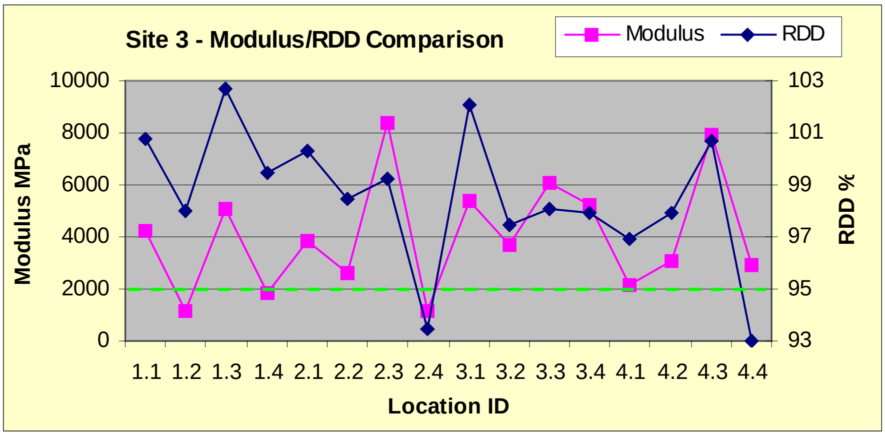


(a) Site 1
Target 1000 MPa

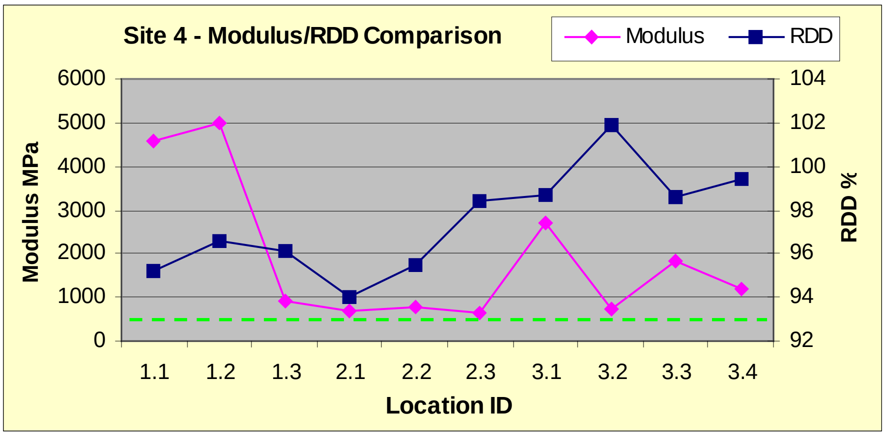


(b) Site 2
Target 650 MPa

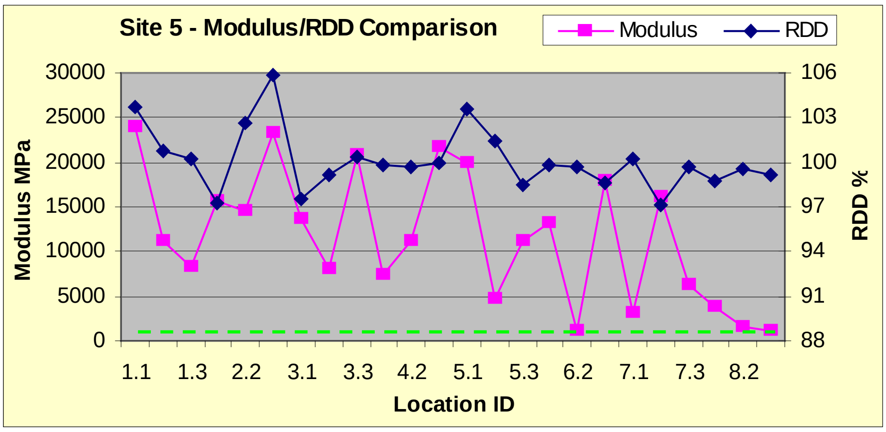
Figure 8-3 (cont) - Modulus and RDD Comparison



(c) Site 3
Target 2000 MPa

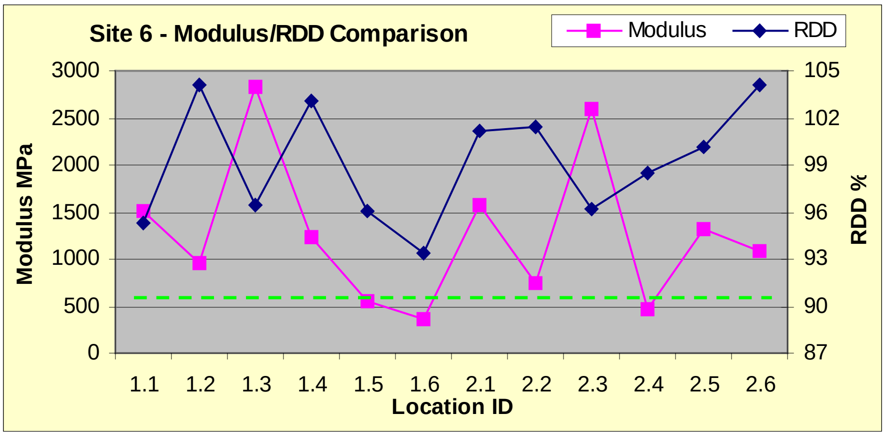


(d) Site 4
Target 600 Mpa



(e) Site 5
Target 1000 MPa

Figure 8-3 (cont) - Modulus and RDD Comparison



(f) Site 6
Target 600 MPa

As can be seen, there is a broad similarity in shape of the RDD and Modulus curves. Peaks in the RDD values generally correspond with peaks in the moduli values, and dips in the RDD values generally correspond with dips in the moduli values. However this is not always the case and it is difficult to conclude that a good RDD result will always indicate a good layer strength or that a poor RDD result will always indicate a poor pavement strength. Points of interest for each site are detailed below.

Site 1 – although all except one RDD reading are below 100%, most of the moduli are above or well above target. The worst RDD reading corresponds with a satisfactory modulus, although only one of the three below-target moduli has an RDD value below 95%.

Site 2 – all moduli are well above target (up to approximately 20 times target). The worst value is more than 3 time the target, even though the RDD reading is 93%.

Site 3 – only two of the sixteen moduli were below target, one of which corresponded to an RDD of 93%, the other to 98%. Another point with a 93% RDD gave a satisfactory modulus. The average modulus is approximately twice target.

Site 4 – all moduli were above target, averaging about four times target, even though the RDD values ranged from 93% to 102%, with seven of the ten below 96%. The highest modulus corresponded with a 94% RDD.

Site 5 – three of the 24 modulus values are only marginally above target, while the remainder are well above target, 18 of them more than five times target. All the RDD values are above 97%.

Site 6 – two of the three below-target moduli show RDD values of 93% and 96% respectively, while the other low result has an RDD of nearly 99%. The highest moduli locations have RDD values around 96%, while one of the modulus values with an above 100% RDD is only marginally above target.

In order to test the premise that test locations on the pavement shoulder may encourage faster failure and lead to low modulus values, the transverse location of the "low modulus" readings were examined. It was thought that proximity to the shoulder edge may contribute to a poor result because of the increased likelihood of moisture ingress or differential settlement between the pavement and natural soil stratus. Ten test locations were in the pavement shoulder region, but only 3 were below the target design modulus. The remaining "low modulus" readings were all well within the traffic lane area. Consequently, there is no evidence to validate this premise and the cause of the low modulus readings remain largely unexplained.

Overall for the entire test sites only 10 of the 83 tests were below the target modulus, whereas 55 of the RDD results were below the 100% requirements ie 68% of the field RDD results failed but only 11% of the modulus values. From these figures, one can draw the inference that rehabilitated pavements will perform considerably better than indicated by the results of the RDD testing taken at the time of construction.

The most obvious fact which arises from viewing the above graphs is the very high proportion of modulus values which are greatly in excess of the target design values – often up to twenty times target and typically averaging from two to five times the target. This raises the question whether the pre-design investigation normally carried out for these projects is appropriate considering the likelihood that the pavements being rehabilitated were built many years ago, and the material properties and thickness of the pavement layers may vary considerably over the length of the rehabilitation project. Although not being considered in detail in this discussion, savings in capital expenditure may ensue from a more detailed consideration of the pavement to be rehabilitated. Also, a more accurate estimate of the life of the rehabilitated road may be obtained, which may affect the economic justification of rehabilitating such a road.

The main controllable variables affecting the cost of reconstruction are the amount of added cement and the amount of new material brought in to improve the material grading. At present day costs, each percentage point of cement powder additive contributes approximately 8% to the cost of the job, typically $14,000 for a 1km reconstruction. Similarly, a 100 mm grading layer contributes approximately 22% to the cost of the job, typically $80,000 for a 1km reconstruction.

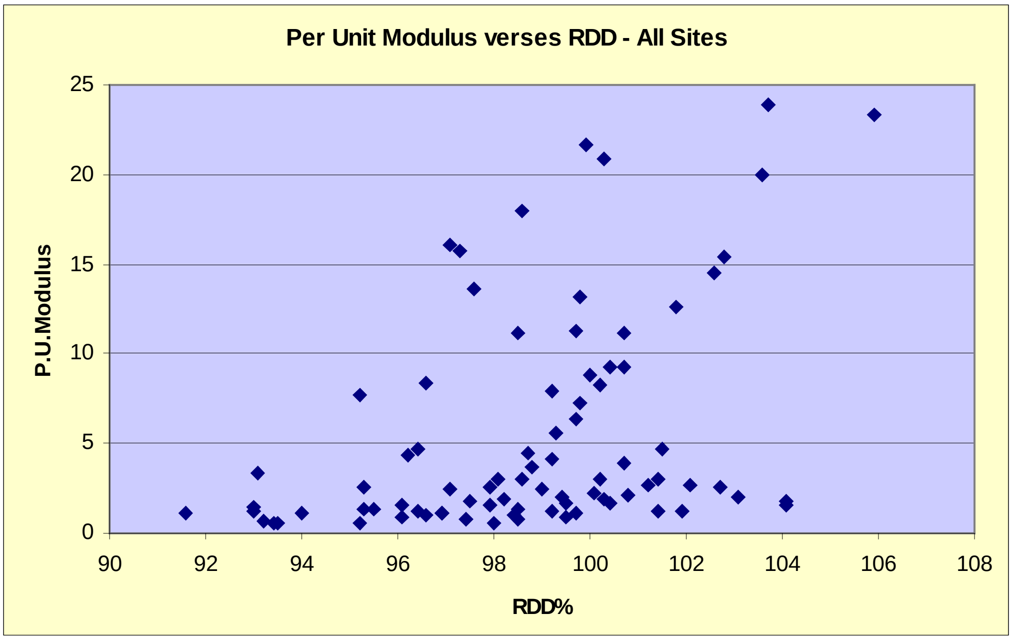
However, in the long term, a stronger pavement should result in a road which requires less maintenance or which will last longer before the next rehabilitation. Using the above moduli figures, it could be inferred that roughly 89% of the road pavement will last for at least the design life, (and much of it for quite a lot longer than the design life), with some 11% of the pavement requiring patching some time before the design life is attained. However, it is often considered that, when offsetting future maintenance against current capital expenditure, the economic justification to spend additional capital in the present to save maintenance expenditure more than fifteen to twenty years in the future becomes moot.

Although it would add to the design cost, the use of the Falling Weight Deflectometer to survey pavements under consideration could provide useful information about the construction of the existing pavement. Raw deflection readings provide the tools to subdivide the road into sections of similar construction, and the variations in the CBR of the subgrade can be estimated directly from the test results. The deflection readings would also provide a useful guide for the selection of test pit locations for spot checks of the layer thicknesses and CBR values. Following from those readings, it would be feasible to estimate the mean and the low-average modulus values of the existing base layers for each similar section of roadway.

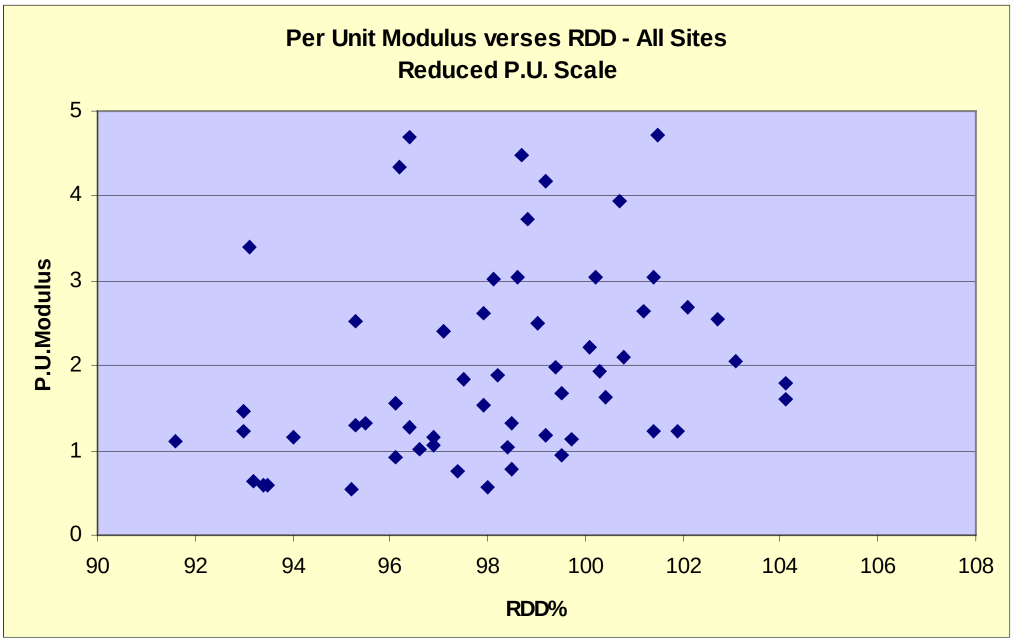
8.2.3 Stabilised Layer Moduli - Correlation with RDD

In order to identify whether there is a correlation between the RDD and modulus values, the modulus values for the stabilised layer for each site were reduced to per-unit values relative to the design modulus, and the values for all test locations sites were plotted against the Relative Dry Density – Figure 8.4. Figure 8.4(a) displays all the moduli, while the maximum for the modulus scale is reduced in Figure 8.4(b) to focus on the modulus values up to five times the design value.

Figure 8-4 - Modulus vs RDD – All Sites
(a) All Values



(b) Reduced Modulus Scale



As can be seen, there is a wide scatter of results for most of the range of RDD values, although, as might be expected, the majority of results are clustered in the 0 – 5 P.U. area. However, there does not appear to be a distinct pattern to any of the results, although it could be noted that there are no modulus values below 1 for RDD values above 100%. However, considering the scatter of results it would be difficult to draw any definitive conclusions from this observation other than that there is no observable correlation between the modulus and the measured RDD test results.

The results do, however seem to confirm that for all cases, the vast majority of results achieve the design modulus regardless of the RDD result, at least down to approximately 91%.

In practice, for each section or lot, the Characteristic Value (CV) of the RDD tests for that lot are calculated to provide a single figure on which to judge the quality of the resulting pavement – calculated as the mean value less the standard deviation multiplied by a factor which depends on the number of tests for that lot.

The modulus values are plotted against these Characteristic Values in Figure 8.5 below. Again, Figure 8.5(a) displays all the results and Figure 8.5(b) limits the modulus to values below 5 P.U. As expected, the scatter in results is similar to the previous plot, with a similar number of points not meeting the target modulus, and no correlation being indicated.

Figure 8-5 - Modulus vs RDD CV – All Sites
(a) All Values

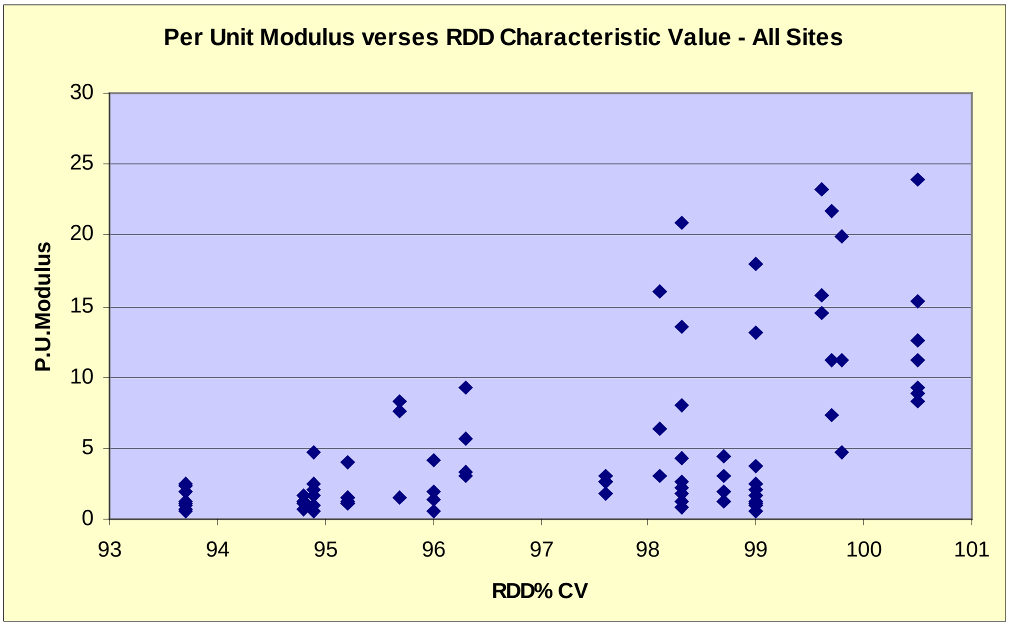
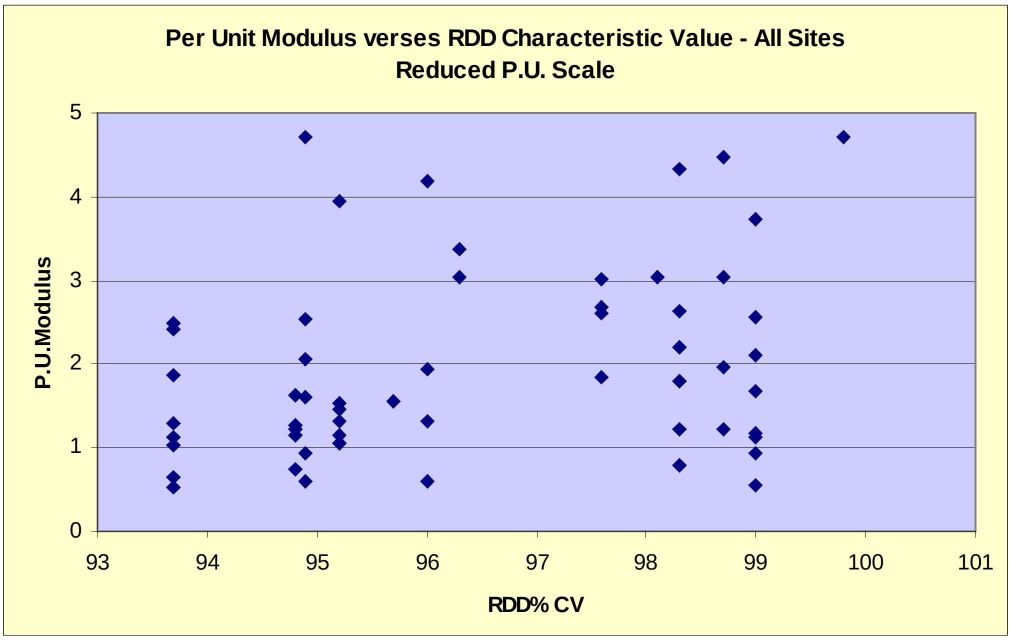


Figure 8-5 (cont) - Modulus vs RDD CV – All Sites
(b) Reduced Modulus Scale



To further extend the concept of using the characteristic value to accept or reject a reconstructed section or lot, the characteristic values for the modulus values were also calculated and compared with the field RDD characteristic values for each of the lots. The results are displayed in Figure 8.6, again showing all points in (a) and points up to 5 PU in (b).

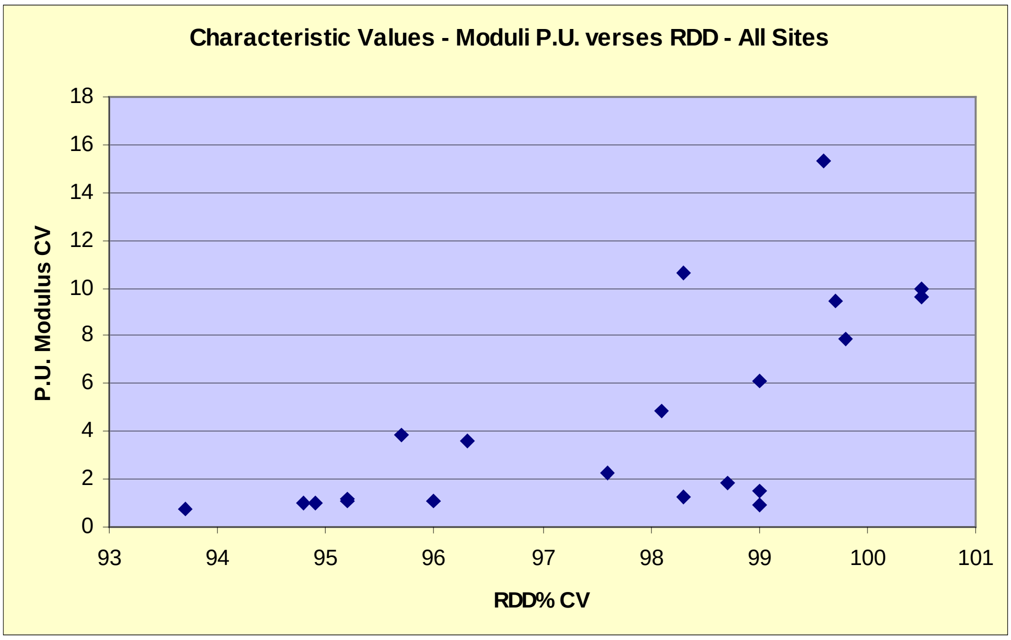
Obviously, the number of points plotted is reduced using this method, and as expected, the plot shows no evidence of a correlation between the RDD and the in-service moduli.

However it should be noted that, by using the Characteristic Value of the RDD for each lot as specified in the Main Roads Standard Specification 11.01, eighteen of the twenty-one lots were rejected as not achieving the required design life. Consequently, the eighteen lots were subjected to reduced level of payments to compensate for the assumed reduced level of performance.

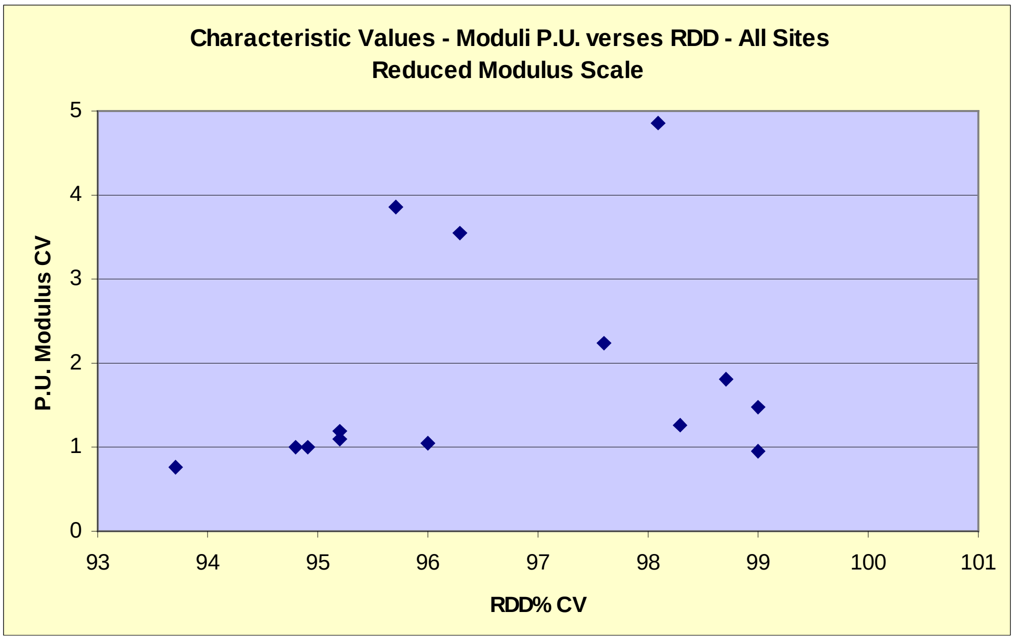
If the same methodology were applied by calculating the characteristic value of the experimentally determined moduli for each lot and rejecting those where the modulus CV is below the design modulus, only two of the twenty-one lots would be rejected. Note also that one of those failed sites produced an RDD value of 99%, which is quite close to being considered satisfactory, while many of the lots with a lower RDD passed easily.

Figure 8-6 - Modulus CV vs RDD CV – All Sites

(a) All Values



(b) Reduced Modulus Scale



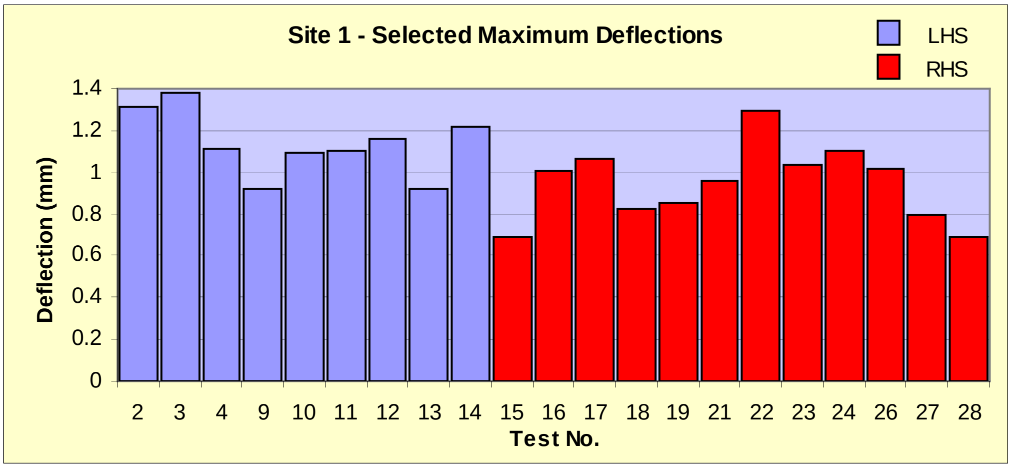
8.3 Sequential Tests

8.3.1 - Deflections

To obtain the representative mean deflections for the sites, the maximum deflection data, obtained during sequential testing as described in Chapter 7, was perused for abnormal deflection results. Abnormal results can be handled in a number of ways. Single abnormalities where there is no obvious reason for the abnormality would usually be discarded as being due to experimental error. In some cases, previous constructions such a truck stop widening or pavement patching or a weak spot in the subgrade may have created a short section which is not typical of the major section of the pavement. In other instances, the results may indicate a grouping of similar strength locations, and in these cases it may be appropriate to split the site into two or more sections and analyse each section separately.

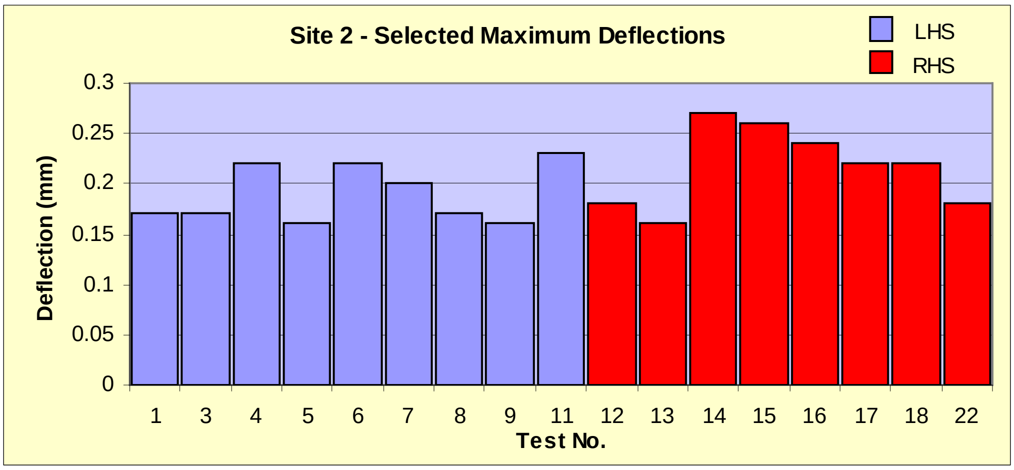
Figures 8-7 (a) – (f) show the maximum deflections for each test location for each site, with the abnormal readings deleted.

Figure 8-7 - Maximum Deflections
(a) Site 1



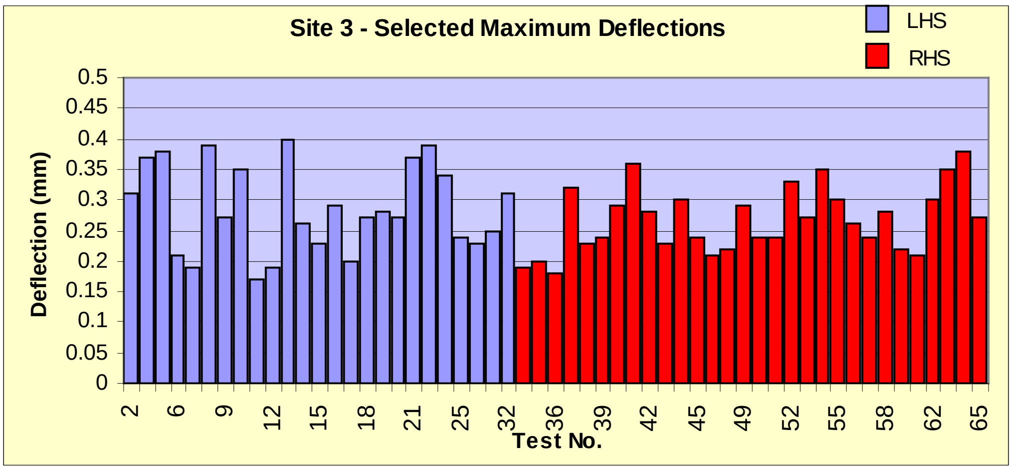
On inspection, tests 5, 6, 7, 8 and 20 were found to be located in a truck stop pad area and it was apparent that this section was different to the normal construction of the rest of the road. Tests 1 and 25 were single abnormalities with no apparent reason. Hence 7 of the 28 test locations were deleted from consideration in the site analysis.

Figure 8-7 (cont) - Maximum Deflections
(b) Site 2



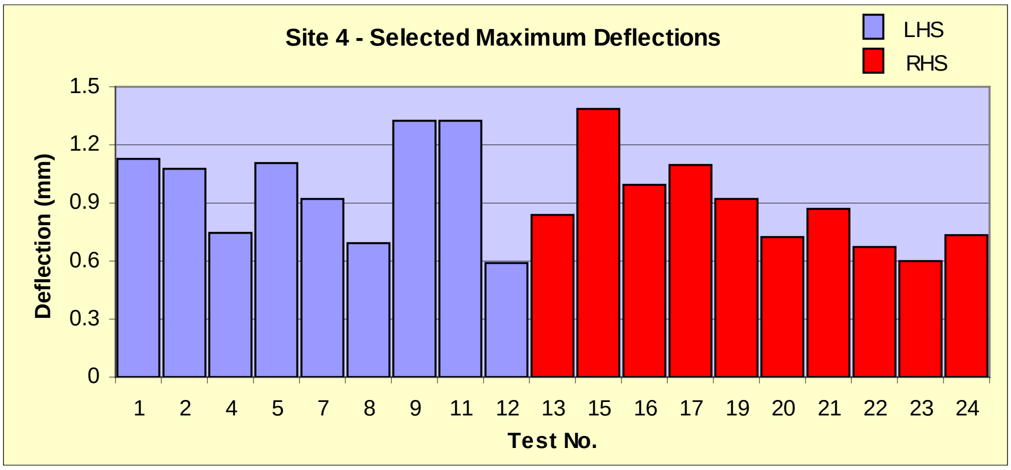
Tests 2 and 10 were single abnormalities with no apparent reason. Tests 19, 20 and 21 formed a group of tests which seemed to be inconsistent with the other results for the right hand side lane, but no specific reason could be identified. Seventeen test locations remain for evaluation.

Figure 8-7 (cont) - Maximum Deflections
(c) Site 3



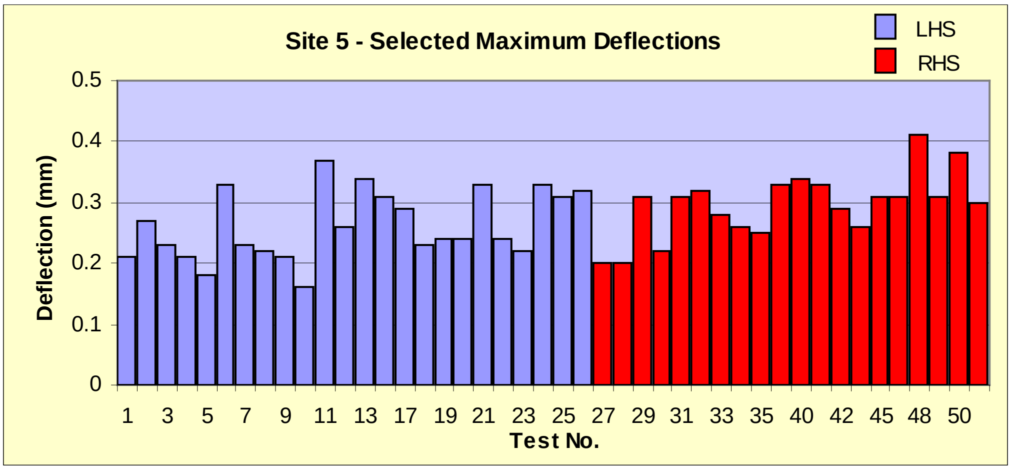
Tests 1, 27 and 33 showed abnormally low deflections, although no specific reason could be identified. Tests 5, 23, 27, 31 and 61 were single anomalies with abnormally high deflections, whilst a group of three tests - 29, 30 and 31 - were high, probably identifying a weak section in the pavement for approximately 150 m. Fifty-five of the 65 tests provide a satisfactory span of results for analysis.

Figure 8-7 (cont) - Maximum Deflections
(d) Site 4



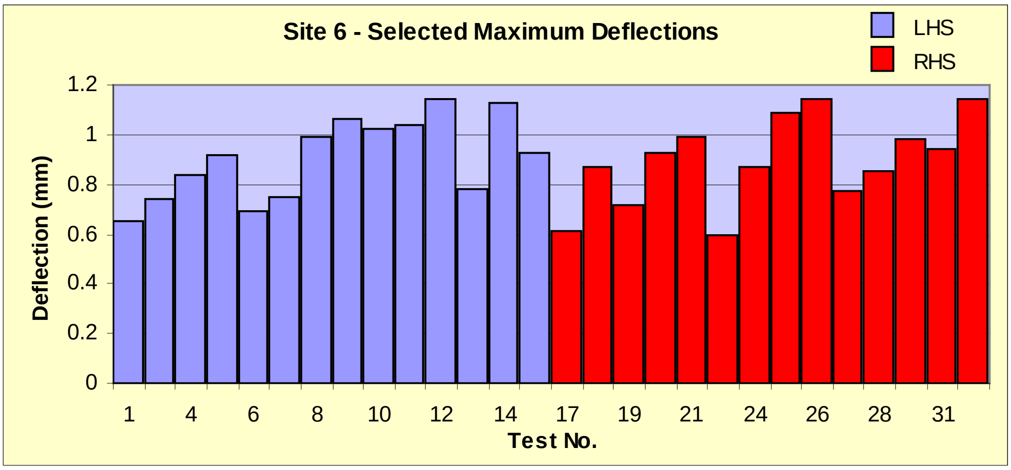
Tests 3, 6, 10, 14 and 18 appeared to be abnormal although no apparent reason could be found. Nineteen of the 24 results remain.

Figure 8-7 (cont) - Maximum Deflections
(e) Site 5



Tests 14, 16, 36, 37, 38, 44, 47 and 52 appeared to show deflections a little higher than the general trends, possibly identifying some occasional weak sections in the pavement. No specific reasons were able to be identified. Fifty-four test results remain for evaluation.

Figure 8-7 (cont) - Maximum Deflections
(f) Site 6



Tests 2, 16, 22 and 29 appeared to be singular abnormal deflections, although no apparent reason could be identified. Twenty-eight of the 32 tests are retained.

Once the "similar sections" have been determined, the mean and standard deviation (S.D.) of the deflections at each sensor radius was calculated for each section. It is common practice to review the validity of the section choices by checking the deflection characteristic value (C.V.) - the standard deviation divided by the mean and converted to a percentage. Characteristic values of less than twenty percent indicate that the chosen locations are reasonably consistent and may be interpreted as a single section.

Table 8.4 – Mean Deflections lists the results of the analysis of the adjusted deflections for each site together with the standard deviation and characteristic value.

Table 8.4 - Mean Deflections - Sequential Tests

Site	Item	Deflection (mm)						
		0	200	300	450	600	900	1500
Site 1	Mean	1.038	0.810	0.672	0.498	0.365	0.228	0.119
	S.D.	0.199	0.148	0.112	0.087	0.067	0.042	0.024
	C.V.	19.2	18.3	16.6	17.4	18.3	18.3	19.9
Site 2	Mean	0.212	0.180	0.162	0.135	0.113	0.081	0.044
	S.D.	0.036	0.029	0.031	0.028	0.027	0.018	0.011
	C.V.	16.77	16.26	18.89	21.07	24.08	22.21	25.31

Table 8.4 (cont) - Mean Deflections - Sequential Tests

Site	Item	Deflection (mm)						
		0	200	300	450	600	900	1500
Site 3	Mean	0.280	0.257	0.238	0.208	0.176	0.130	0.070
	S.D.	0.065	0.053	0.045	0.035	0.027	0.019	0.013
	C.V.	23.07	20.75	18.84	16.90	15.08	14.45	18.05
Site 4	Mean	0.991	0.724	0.572	0.398	0.275	0.151	0.068
	S.D.	0.261	0.180	0.139	0.098	0.072	0.049	0.025
	C.V.	26.30	24.89	24.40	24.58	26.34	32.51	36.88
Site 5	Mean	0.286	0.255	0.235	0.198	0.163	0.114	0.056
	S.D.	0.059	0.046	0.041	0.036	0.033	0.029	0.020
	C.V.	20.50	18.13	17.69	18.20	20.14	25.57	36.50
Site 6	Mean	0.950	0.696	0.558	0.408	0.305	0.200	0.110
	S.D.	0.172	0.104	0.070	0.049	0.037	0.022	0.008
	C.V.	18.10	14.95	12.63	11.99	12.06	11.08	7.32

Table 8.5 – Sequential Tests Deflection Ranges consolidates the mean deflection results for each site in Table 8.4, to provide deflection sets for the mean, mean plus one standard deviation, and mean plus two standard deviations. The moduli calculated from these values are indicative of the mean modulus for each site, and the moduli where approximately 84% and 97.5% of the tests respectively show moduli above these values.

8.3.2 – Mean Moduli

The moduli for the mean deflections and for the deflections at one and two standard deviations removed from the mean were calculated using CIRCDEF. The results are shown in Table 8.6 – Site Moduli Comparison, together with the design moduli for each sites for comparison.

Table 8.5 - Sequential Tests Deflection Ranges

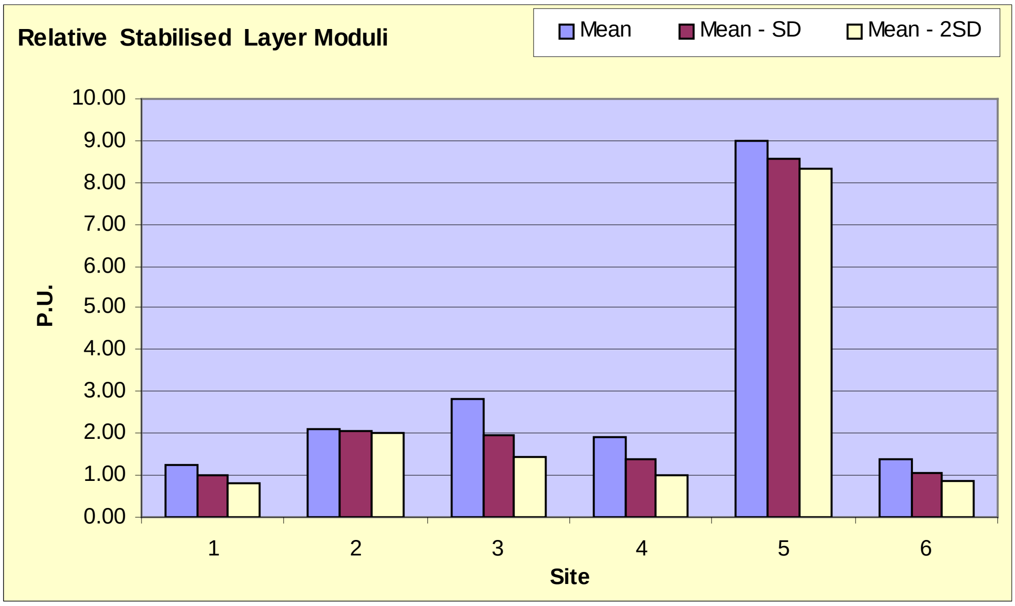
Site	Item	Deflection (mm)						
		0	200	300	450	600	900	1500
Site 1	Mean	1.038	0.810	0.672	0.498	0.365	0.228	0.119
	+S.D.	1.237	0.958	0.784	0.585	0.432	0.270	0.143
	+2 S.D.	1.437	1.105	0.896	0.672	0.499	0.311	0.167
Site 2	Mean	0.212	0.180	0.162	0.135	0.113	0.081	0.044
	+S.D.	0.248	0.210	0.193	0.163	0.140	0.099	0.056
	+2 S.D.	0.283	0.239	0.223	0.192	0.167	0.117	0.067
Site 3	Mean	0.280	0.257	0.238	0.208	0.176	0.130	0.070
	+S.D.	0.344	0.310	0.283	0.243	0.203	0.149	0.082
	+2 S.D.	0.409	0.363	0.328	0.278	0.230	0.168	0.095
Site 4	Mean	0.991	0.724	0.572	0.398	0.275	0.151	0.068
	+S.D.	1.252	0.905	0.711	0.496	0.347	0.199	0.093
	+2 S.D.	1.513	1.085	0.851	0.594	0.420	0.248	0.118
Site 5	Mean	0.286	0.255	0.235	0.198	0.163	0.114	0.056
	+S.D.	0.345	0.302	0.276	0.234	0.196	0.143	0.076
	+2 S.D.	0.404	0.348	0.317	0.270	0.229	0.172	0.097
Site 6	Mean	0.950	0.696	0.558	0.408	0.305	0.200	0.110
	+S.D.	1.122	0.800	0.628	0.457	0.342	0.223	0.119
	+2 S.D.	1.294	0.904	0.699	0.506	0.378	0.245	0.127

Table 8.6 - Site Moduli Comparison

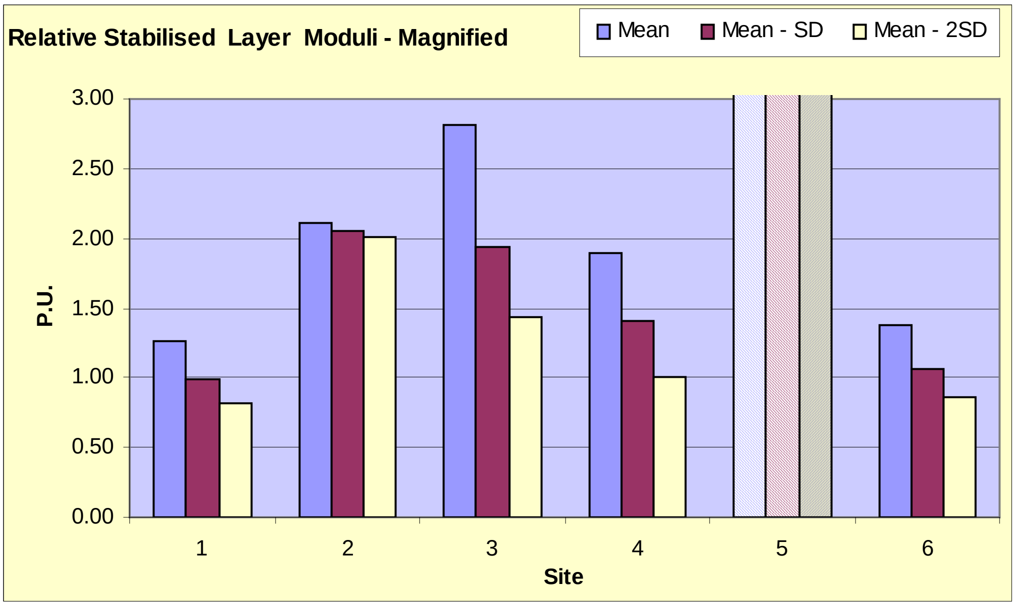
Site ID No.	Design Modulus MPa				Test Modulus MPa		
	Subgrade	Sub-base	Base	Item	Subgrade	Sub-base	Base
1	40	69	1000	Mean	83	44	1258
				+SD	69	42	996
				+2SD	60	39	818
2	50	650	1000	Mean	221	1367	14072
				+SD	180	1330	11954
				+2SD	150	1309	10032
3	30	-	2000	Mean	136	-	5635
				+SD	121	-	3878
				+2SD	109	-	2870
4	71.4	71.0	600	Mean	132	29	1135
				+SD	93	25	846
				+2SD	73	28	600
5	50	-	1000	Mean	141	-	8983
				+SD	115	-	8549
				+2SD	95	-	8327
6	50	186	600	Mean	96	76	825
				+SD	87	70	636
				+2SD	79	60	820

The modulus values in Table 8.6 for the stabilised layer are plotted in Figure 8-8 for comparison. The moduli are displayed as per-unit values of the moduli relative to the design modulus for the relevant site. Note that for Site 5, the calculated per unit values were substantially higher than for the other sites, and the values are re-plotted at a magnified scale in Figure 8.8(b) to make the values for the other sites more readable.

Figure 8-8 - Relative Stabilised Layer Moduli Comparisons
(a) All Values



(b) Magnified Scale



In all cases, the mean modulus of the stabilised layer exceeds the design modulus, ranging from relatively small margin to the very significant nine times target. The modulus values obtained from the deflections at one standard deviation from the mean also achieve or exceed the design moduli, from which may be inferred that 84% or more of the pavement has a modulus greater than the design modulus.

The moduli calculated from the deflections at two standard deviations from the mean exceed the design target for sites 2, 3, 4, and 5, which indicates that at least 97.5% of the pavement has achieved the design modulus. However, the 2SD moduli for sites 1 and 6 were below the design target so that a larger proportion of the pavement, up to 16%, is weaker than designed and can be expected to fail before the end of the design period.

8.3.3 – Site Modulus Discussion

Sites 1, 4 and 6

Sites 1,4 and 6 were pure pavement rehabilitation projects on rural roads. The MRD targeted these sections because of poor rideability results achieved during a routine survey regularly carried out on all main roads. These sections were all low grade roads with relatively light traffic, and were constructed with relatively shallow pavements. Because of the lower ranking of these roads, rehabilitation was mainly focussed on improving the surface shape rather than providing a strong road for heavy traffic with a large proportion of commercial activity. Minimal material was added for shape correction and material grading improvement.

Because of this, a substantial variability in the properties of the original base and subgrade material could be expected. Additionally, the stabilised layer had little material added to improve the strength and grading. A weaker pavement and variable deflection measurements would be anticipated, with relatively high standard deviation over the modulus values being shown. This assumption is compatible with the results shown above.

Site 2

Site 2 is a combination of rehabilitation and reconstruction on a small section of road. The existing base and subbase materials were thoroughly mixed and insitu stabilised to form the subbase layer for the cement treated base layer constructed with new imported material. The road carries a high proportion of heavy vehicle traffic and the original pavement used to form the subbase was a good quality material. As expected, the consistency of the existing material and the new base layer material resulted in a pavement where the range of deviation from the mean modulus is relatively small.

Site 3

For Site 3, the existing pavement material was improved by mixing a proportion of new material to the pulverised layer. This site's terrain is quite undulating with some hard rocky outcrops which means that the properties of the original pavement and underlying support is quite variable. This reflects through to the modulus strength of the rehabilitated upper layers. This variability results in the relatively large range of moduli about the mean shown by the graph.

Site 5

Site 5 is the designated heavy vehicle bypass route to the Bowen Basin coal mines. The rehabilitation included the placement of a 75 mm corrector layer of new material before stabilisation. The added cement content was relatively high at 3%. The original pavement structure was of a consistent high quality material, but required upgrading to cater for the heavier duty as the heavy vehicle bypass route. Thus, the high mean modulus and relatively tight deviations from the mean values are to be expected.

Overall, it would appear that the tests have provided results which are consistent with the construction methods and the materials used in the reconstruction. Consequently, interpretation of the resulting moduli can be made with a reasonable degree of confidence that the figures reflect the ranges of moduli with acceptable accuracy.

8.3.4 – Sequential Tests Life Forecasts

Using the calculated moduli listed above, the remaining service life for each site was calculated using CIRCLY, with the results shown in Table 8.8

The number of SARs remaining until failure were calculated for the mean modulus and the moduli at 1 and 2 standard deviations from the mean. Table 8.5 – Remaining SARs to Failure, displays the results of this CIRCLY analysis.

The normal failure mode is by an excessive compressive strain at the top of the subgrade. However, in four cases, CIRCLY determined that failure would occur by tensile fatigue cracking at the bottom of the cement treated layer. To determine the remaining forecast life, the worst cause scenario must be used.

Sites 1, 2, 4 and 6 followed the expected pattern with the forecast SARs decreasing in line with the modulus values. In these cases the "second standard deviation" modulus is the critical value, accounting for 97.5% of the pavement, giving a good representation of the life of the pavement as a whole.

With Site 3, the SARs for the mean moduli is lower than for the first and second standard deviations. The pavement failed not by the excessive compressive strain at the top of the subgrade rather by the tensile strain at the bottom of the cement treated layer. This means that the stiffness of the stabilised layer over the weaker supporting layers would induce fatigue cracking and the layer would begin to crumble from the bottom and propagate towards the top of the layer, significantly reducing the life of the pavement. At the other moduli values, the failure mode returned to the compressive strain at the top of the subgrade.

Site 5 showed a very high mean and a small deviation range, so that all failure modes were by fatigue at the bottom of the stabilised layer. As can be seen the expected SARs to failure is significantly less then the design SARs. This means that the pavement will probably fail quite sooner than expected because of the excessive stiffness of the stabilised layer.

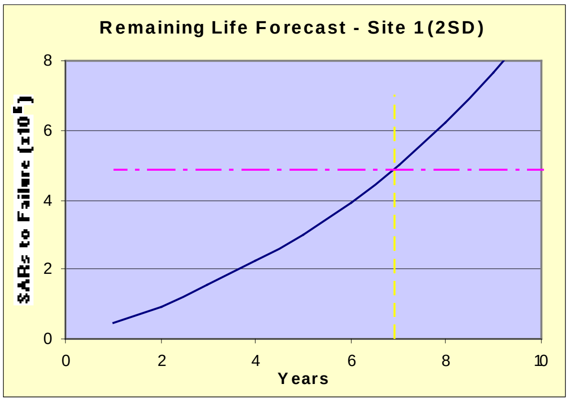
Table 8.7 - Remaining ESAs to Failure

Site ID No.	**SARs to Failure (x 10^6)**			
	Design	**Mean**	**1SD**	**2SD**
1	1.0	12.5	7.0	4.9
2	4.5	10.3	8.9	6.9
3	0.93	4.6*	7.5	7.3
4	1.05	16.0	5.7	2.0
5	3.7	0.6*	0.6*	0.6*
6	1.9	10.7	7.8	5.1

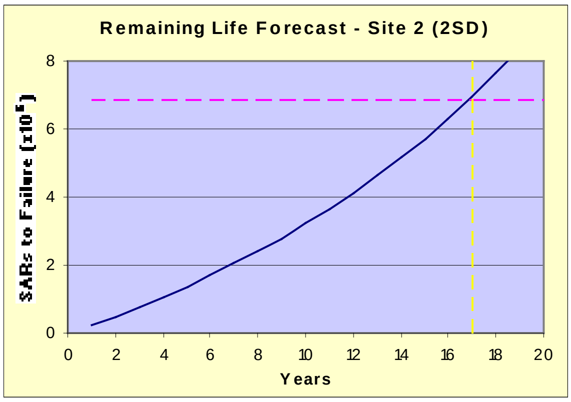
* Failure mode by the fatigue cracking at the top of the cement treated layer.

The lowest SARs values in Table 8.7 were then used as the target line to determine the remaining life of the pavement in years. The cumulative growth forecast ESAs were plotted and thus the remaining life determined. The remain life forecast from each of the sites is depicted in Figure 8.9 below.

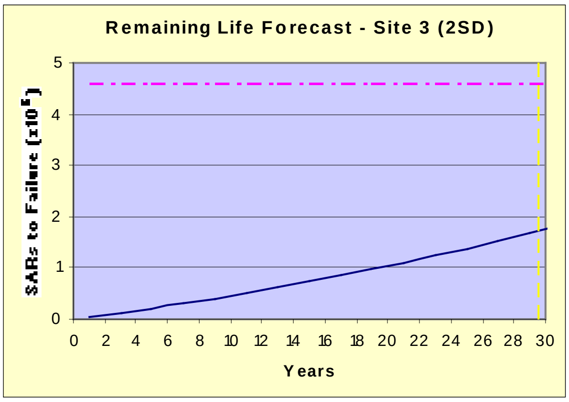
Figure 8-9 - Relative Stabilised Layer Moduli Comparisons



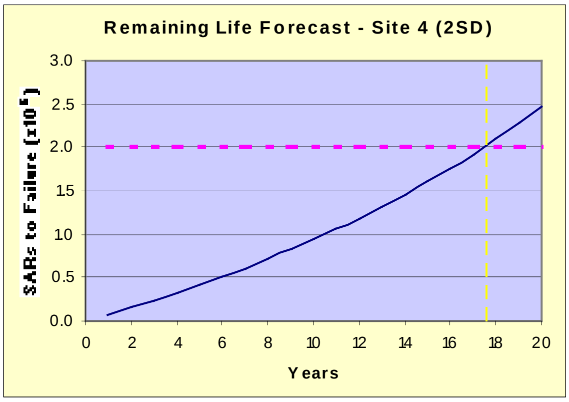
(a) Site 1



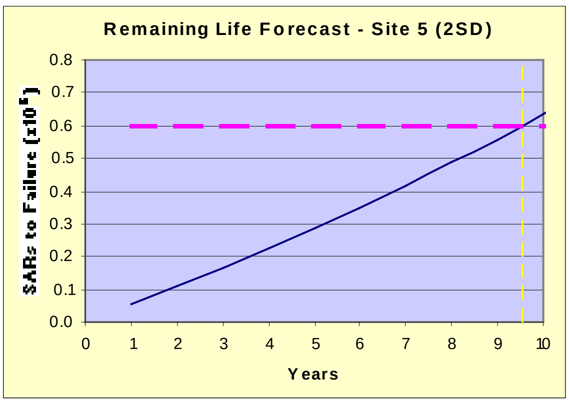
(b) Site 2



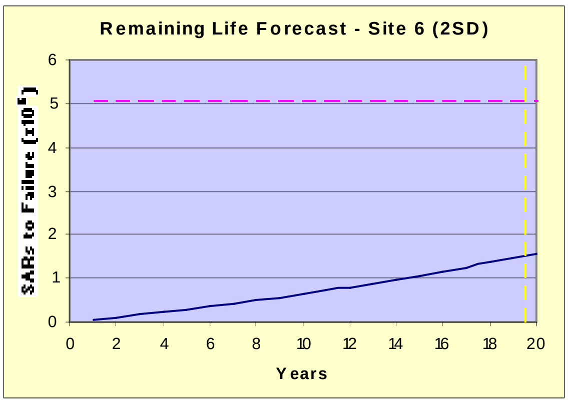
(c) Site 3



(d) Site 4



(e) Site 5



(f) Site 6

The remaining life results for each of the sites is tabulated with the design life in years for comparison in Table 8.8.

Table 8.8 - Forecast Life

Site	Design Life	Current In-Service Life	Calculated Remaining Life
1	2	1.7	6.9
2	12	1.5	17
3	16	1.5	> 20
4	12	1	17.5
5	29	2.5	9.5
6	9	1	> 20

Site 1

This site was a temporary fix to correct the heavy rutting that had occurred over time because of the expansive subgrade underneath. The temporary fix was only designed for a life of 2 years and as can be seen from Table 8.8 it should be nearing the end of its design life.

The calculated remaining life of this pavement, from CIRCLY, is another 6.9 years. The target modulus for the stabilised layer was 1000 MPa and the 2nd deviation modulus, used for the life forecast, achieved 818 MPa. The reason for the increased design life is the difference in the strength of the natural subgrade material, which was at the time of testing 60 MPa. The calculation of the strength of the subgrade used for design purposes is usually tested in a worst case scenario by a soaked CBR. This subgrade material on this section of road would be expected to be relatively dry with no heavy rainfall occuring in the region for some time.

The visual inspection confirmed this result as very little fatigue was evident.

Site 2

This test site was a small proportion of a large project leading onto a bridge. This road is a high priority road carrying a high proportion of heavy vehicle traffic. The target modulus of the stabilised layer was 650 MPa and the modulus achieved for this layer was 1309 MPa, significantly stronger. The stabilised section was overlayed with a high quality cement treated material, which easily achieved category one status. The design life of the project was 12 years and it has been in-service for one and a half years to date. The calculated remaining life of this section is 17 years.

Site 3

This section of the Sarina-Homebush Road was badly deformed, requiring an additional 100 mm of material to bridge the poor subgrade material. This road is not heavily trafficked and was designed to achieve a design life of 16 years. The targeted modulus for the stabilised layer was 2000MPa and the second standard deviation strength was 2870 MPa. A small percentage growth rate of only 2.4% and current light traffic volumes has increased the life of this pavement well over the 16 years and could possible achieve a life well over 25 years. The road is also eventually expected to fail because of fatigue failure at the bottom of the stabililsed layer and not by failure of the subgrade.

Site 4

This existing pavement section on the Dysart-Middlemount Road was out of shape with depressions and high spots. The low cost stabilisation treatment was designed for a life of 12 years. The remaining life of the pavement is expected to be another 17.5 years. The subgrade material is on average slightly stronger than the existing material.

Site 5

The Marion-Eton Road is the designated heavy vehicle bypass road for traffic from Mackay to the mines. A high design life of 20 years was targeted but the design figures indicated a calculated SARs to failure equivalent to 29 years. The results from the deflection tests suggest that the stiffness of this pavement is excessive and the base layer will fail by fatigue after only 9.5 years. It would be interesting to follow the performance of this pavement to verify the forecast of this analysis.

Site 6

This original pavement section on the Fitzroy Development Road was similar to Site 4 and required a low cost stabilisation treatment which would provide a life of at least 9 years. The deflection testing indicates that the pavement would be expected to last in excess of twenty years, this largely due to the current low traffic levels and the greater strength of the layers. The stabilised layer at 800 MPa was slightly stronger than the design assumption, as was the subgrade material, achieving a CBR of 8 in a majority of the FWD tests compared with the design CBR of 5.

CHAPTER 9 – CONCLUSIONS

This project involved reviewing the processes for the design and rehabilitation by insitu stabilisation of several sections of road pavements and assessing the in-service performance of those pavements after being in service for up to approximately eighteen months.

The primary broad objective of the investigation was to evaluate whether the Relative Dry Density testing done during construction provided a measure of the value of the modulus achieved for the stabilised layer and whether the likely service life would be equal to or greater than the design life when the Relative Dry Density test results were less than the specified 100%.

Visual inspections of the sites verified that after only six to eighteen months of service, the major proportion of the pavements were showing no signs of distress. The area covering the failures equated to less than 0.1% of the total pavement, and did not appear to have been caused by the traffic loading. There were no reasons to suggest any conflict between the design and construction processes and the expected use of the pavement.

Analysis of site specific deflection data, where deflection measurements were made adjacent to the spots where the Relative Dry Density test were taken, indicated no correlation between the Relative Dry Density values and the moduli of the stabilised layer. While there appears to be a trend for high RDD values to correspond with high moduli values, this is not consistently true. Additionally, a low RDD value can not be used to infer that the pavement layer did not achieve the design modulus.

Furthermore, by the application of the characteristic value technique to the moduli recorded for each lot in a similar way to the application of the RDD, all but two of the twenty-one lots attained a modulus CV equal to or above the design modulus, whereas the RDD CV test failed all but two. Many lots with a low RDD exceeded the design modulus by a considerable amount.

Hence, it can be inferred that RDD results below 100% do not necessarily result in a below-standard pavement, at least for RDD values down to approximately 93%. Conversely, an RDD above 100% does not necessarily indicate a satisfactory pavement.

Because of the absence of a correlation between RDD and modulus, it would appear that the RDD test results cannot be used to forecast a possible loss of useful service life. The estimate of service life requires some alternate form of measurement such as deflection testing at regular intervals using a Falling Weight Deflectometer, such as the sequential testing carried out for this investigation.

However, the results of this testing suggests that in general, the actual moduli and the forecast life of the pavement is not very consistent with the values anticipated by the design. It would appear that many more investigations similar to this project will need to be carried out to investigate the relationship between design assumptions and the life of the final product. Unfortunately, definitive proof of the assumptions and performance could take up to 20 or more years.

Of particular concern are the results for the sites where the measured stabilised layer moduli are well in excess of the design moduli, creating a situation where the failure mode changes, with a substantially reduced forecast life. The implications of this situation needs to be further investigated by a more comprehensive study at that site and if possible at other sites which exhibit similar phenomena.

It is recommended that:

- RDD testing of insitu stabilised pavement construction projects be retained as a quality control measure;
- the application of the reduced level of service payment required in the Main Roads Specification 11.07 where the characteristic value is less than 100% be applied only to characteristic values less than 93%;
- the requirements of the present specification relating to the construction process be retained so that compactions in the range 96% to 100% RDD will be achieved as at present;
- further studies be undertaken on similar reconstruction projects in other districts to compare with the results of these investigations;
- a set of controlled studies be instituted on a small number of new insitu stabilisation projects where more detailed pre-design investigations can be made and the variations in the existing layer properties can be more rigorously determined.

REFERENCES & BIBLIOGRAPHY

Amey. D.J 1985, *Performance of Cement Stabilised Pavements in Payneham*, South

ARRB 1999, *Insitu Stabilisation of Marginal Pavement Material*, APRG Technical Note 9, Austroads Pavement Research Group

Australia, Cement and Concrete Association of Australia, Technical Report.

Austroads 1998, *Guide to Stabilisation in Roadworks*, Austroads Inc. Sydney.

Austroads 2003, *Guide to Best Practice for the Construction of Insitu Stabilised Pavements*, Austroads Inc. Sydney.

Austroads 2004, *Pavement Design: A Guide to the Structural Design of Road Pavements*, Austroads Inc, Sydney.

AustStab 1999, *Australian Binders used for the Stabilisation and Road Recycling Industry*, National AustStab Guidelines.

AustStab 1999, *Site investigation for the rehabilitation of low trafficked roads using insitu recycling*, National AustStab Guidelines.

Hall. B.S 2005, *Insitu Cement Stabilisation*, Department of Main Roads, Mackay

Higgins, D.D 2005, *Soil Stabilisation with Ground Granulated Blastfurnace Slag*, UK Cementitious Slag Maker Association (CSMA)

Gerrard, C.M. and Harrison, W.J. 1971, The Analysis of a Loaded Half Space Comprised of Anisotrophic Layers, CSIRO, Australia

Gordon, C 2005, *Determination of Insitu Flexible Moduli of Modified Pavement Materials*, University of Southern Queensland, Toowoomba

Hodgkinson. G.F. 1991, *Long Term Performance of local government road pavements recycled by cement stabilsation,* In proceedings of the 6th National Local Government Engineering Conference. Hobart.

Jameson, G.W. 1996, *Origins of Austroads Design Procedure for Granular Pavements*, ARRB TR Research Report no.292, ARRB Transport Research Ltd, Melbourne.

Main Roads Department of Queensland 1990, *Pavement Design Manual*, Second Edition, Director of Transport Technology, Brisbane.

Main Roads Department of Queensland 1992, *Pavement Rehabilitation Manual*, Second Edition, Pavements and Asset Startegy, Brisbane

Main Roads Department of Queensland 1999, *Workshop on Low Volume Roads*; Technical Training Solutions, Brisbane

Main Roads Department of Queensland 1990, *Material Testing Manual*, Director of Transport Technology, Brisbane

Main Roads Department of Queensland 1999, *Department of Main Roads Standard Specifications*, Director of Transport Technology, Brisbane

Meijer, H 1995, *The Performance of Cement Stabilised Road Pavements Thesis*, School of Civil Engineering, University of Technology, Sydney.

MINCAD Systems Pty Ltd 2004, *CIRCLY 5 User Manual*, Richmond, Victoria

Morton P.C. 1993, *Recycling and Stabilisation of Pavement Materials*, Adelaide.

MRD Test Method No. Q102A - 1983, *Standard Moisture Content – Over Drying*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q103A - 1996, *Particle Size Distribution (Wet Sieving)*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q104D - 1988, *One Point Liquid Limit (Cone Penetrometer)*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q105 - 1987, *Plastic Limit and Plasticity Index*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q106 - 1989, *Linear Shrinkage*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q110A - 1996, *Dry Density / Moisture Relationship – Standard Compaction*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q110C - 1982, *Dry Density / Moisture Relationship of Cement Treated Material – Standard Compaction*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q111A – 1996, *Insitu Dry Density – Sand Replacement*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q113A – 1993, *Californian Bearing Ratio - Sand Compactive Effort*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q113C – 1998, *Californian Bearing Ratio at Nominated Levels of Dry Density and Moisture Content*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q114B – 1978, *Insitu Californian Bearing Ratio – Dynamic Cone Penetrometer*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q115 – 1978, *Unconfined Compressive Strength*, Main Roads Department of Queensland, Brisbane

MRD Test Method No. Q719 – 1998, *Field Spread Rate of Solid Stabilisation Agents (Fabric Mat)*, Main Roads Department of Queensland, Brisbane

NAASRA 1987, *A Guide to the Visual Assessment of Pavement Condition*, Austroads, Sydney.

Pike, ML 1997, *Assessment of Thin Layered Insitu Stabilised Road Pavements Thesis*, School of Civil Engineering, University of Technology, Sydney.

Rubin D.S and Levin R.I 1994, *Statistics for Management*, Prentice Hall International Incorporated, New Jersey

Smith. W. and Vorobieff. G. 2007, *Recognition of sustainability by using stabilisation in road rehabilitation*, ASA Sustainability & Slag Conference.

Sullivan, B.W 2000, *Municipal Services Study Book 2000*, Distance Education Centre, Toowoomba, Queensland

Symons, M.G. and Poli, D.C. 1998, *Properties of Australian soils stabilised with cementitious binders*, University of South Australia

University of Southern Queensland 2000, *Municipal Services Study Book*, Distance Education Centre, Toowoomba.

Vorobieff, G. 1998a, *Performance and Design of Insitu Stabilised Local Government Roads*, Insitu stabilisation of Local Government Roads Seminar, Brisbane: www.auststab.com.au

Vorobieff, G. 1998b, *The Australian Experience*, In Proceedings from National Pavement Stabilisation Strategies for the future forum, Rotorua, New Zealand: www.auststab.com.au.

Vorobieff, G. 2004, *Modification versus bound pavements*, NZIHT Stabilisation of Road Pavements Seminar June 2004, AustStab.

Wardle, L.J. 1976, *Integral Transform Methods for Multilayered Anisotrophic Elastic Systems*, CSIRO, Australia

Williams R.I.T. 1986, *Cement Treated Pavements*, Design and Construction, Elsevier Applied Science Publishers, England.

Wilmont T. 1993. '*Selection and Operation of Equipment for Deep Layer in-situ stabilisation*'. Cement and Concrete Association of Australia. Brisbane.

APPENDIX A – PROJECT SPECIFICATION

Faculty of Engineering and Surveying

ENG4111 Research Project

Project Specification

March 2007

Student:	Mark Weatherley
Student Number:	Q9723871X
Project Topic:	The effect of compaction on the design life of rehabilitated in-situ stabilised (cement powder) pavements.
Supervisors:	Mr Trevor Drysdale, USQ Mr William Lansbury, RoadTek Mackay District, QLD
Aim:	To investigate whether there is an unacceptable reduction in the level of service of rehabilitated pavements which have been in-situ stabilised with cement powder but where the specified compaction level was not achieved using "standard" work procedures for in-situ stabilisation.
Background:	The construction arm of the Main Roads Department in the Mackay District, RoadTek undertakes approximately 8 road rehabilitation projects each year involving the in-situ stabilisation of pavement material with general blend cement. Each project is subdivided into half-road width lots of approximately 700 metres, and a soil test regime is carried out for each lot. The annual budget for these projects is approximately $6.5 million and accounts for approximately 35% of the infrastructure construction/reconstruction carried out by RoadTek in the District. In a significant number of projects the standard stabilisation process does not produce compaction results which meet specification. The standard contract provides for a reduced level of payment to compensate for a reduced level of service inferred because of failure to meet compaction specifications or alternatively the compacted pavement has to be reworked to achieve the specified compaction. On average, the typical reduction in payment for the reduced level of service is approximately $14,000 per project, or alternatively $18,000 per project for reworking, ie an estimated $150,000 annually. There is anecdotal evidence that despite not meeting specification there is no appreciable degradation of service for compactions above about 93% compaction, hence the expense of meeting specification is unnecessary, and the specification could be relaxed with a consequent cost saving.

This project will investigate whether there is any factual basis for these anecdotal inferences. If so, the project will recommend changes to the contract requirements, or alternatively, recommend changes to the standard procedure for in-situ stabilisation to ensure the compaction standard is met.

On a local level the Standard Specification allows for district specific addenda or supplementary specifications. It has been suggested that this problem is not unique to Mackay and it is possible that the results of this study may be applicable on a state wide basis.

Program:

1. Review literature relating to the design and compaction of road pavements with particular reference to plant-stabilised and in-situ stabilised materials, correlating the effect of compaction on service life and it's relevance to this project.
2. Review the design standards currently in use and determining the rationale behind the requirements of the specification, including but not limited to the design life, in-situ material strength and the compaction required in the specification.
3. Review the soil test documentation available in the Mackay District, identify the lots with suitable test data and collate the relevant parameters that may impact on the pavement performance;
4. Perform and record visual assessments of the pavement condition for each lot, in accordance with the Austroads Standard, together with estimations or readings of traffic density and length of time in service since reconstruction;
5. Arrange deflection testing using the Falling Weight Deflectometer (FWD) at locations corresponding to the original test data to estimate the underlying strengths, and compare the results. If possible, compare these results with other newly constructed stabilised pavements so that comparisons of expected service life with "normal" construction methods can be made;
6. Analyse the data to identify whether there is a correlation with the parameters collected for each pavement section;
7. Evaluate whether there is sufficient evidence to indicate whether an expected service life equivalent to normal design life can be attained with a less stringent compaction requirement, or alternatively reinforce the need to attain the specified compaction and review the in-situ stabilisation process to more regularly achieve these results without re-work; and
8. Recommend changes (if any) which could be incorporated into the Standard Specification MRS 11.07.
9. Presentation of project work in required oral and written formats.

Approved: 26/03/07

USQ Supervisor: (signed) Trevor Drysdale

RoadTek Supervisor: (signed) Bill Lansbury

Student Name: (signed) Mark Weatherley

APPENDIX B – ROAD PAVEMENT DESIGN

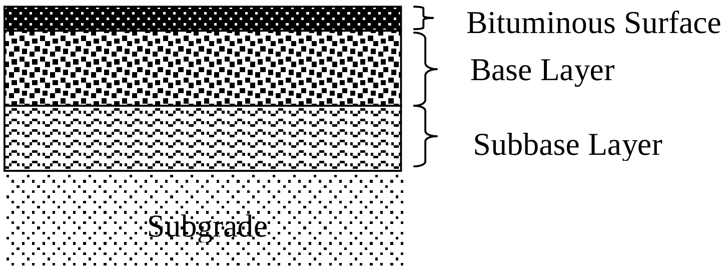
When a traffic route is chosen to carry vehicles from one location to another, it is generally found that the natural soil is not strong enough to support repeated applications of even relatively light wheel loads without significant permanent deformation.

It is therefore necessary to cushion the natural soil by creating a structure capable of bearing the applied loads and distributing them over the natural soil. This structure is called a pavement (Municipal Services Study Book 2000, p. 4.1).

B.1 Pavement Structure

A pavement is generally constructed by preparing the natural soil to the required profile, then constructing a number of layers of material, of increasing strengths, over the natural soil, and capping the structure with a water-proofing and wearing layer. Figure B.1 shows a typical construction of a pavement.

Figure B-1 - Typical Pavement



The subgrade is the base of the construction and is typically the existing soil, although in some instances it may be necessary to excavate or place embankment material to reach the subgrade formation level. If the natural ground is structurally too weak it may be necessary to excavate and replace with selected fill material or to treat the soil to improve its properties. The main purpose of the overlying layers is to distribute the traffic load so the subgrade can support it without damage.

The base and subbase are the main load-bearing layers of a pavement. The materials used to construct the bases are chosen for their inherent load-spreading capabilities when correctly laid. They are typical made up of crushed rock of various sizes up to 19 mm interspersed with finer rock and fine clay material. The material usually comes from specialised quarries and is often transported for considerable distances to the construction site. When properly compacted the air voids within the layer are minimal.

Generally, the higher the strength, the more expensive the material. The greatest strength is required in the top-most layer, so for thick pavements, costs can be reduced by building the pavement in layers with less expensive material under the top layer.

The bituminous surfacing is a bitumen and aggregate mix applied typically in two applications, a fine aggregate primerseal followed by a more viscous seal with larger aggregate. The bituminous layer provides a seal to minimise water infiltrating the pavement, and the bound aggregate provides the wearing surface to resist the wear of the traffic and prevent the bitumen being worn away. On heavily traffic roads, an additional asphalt "wearing layer" may be added to provide a longer lasting wearing surface. All the pavements that are the subject of this report have been surfaced by the conventional bituminous aggregate mix.

B.2 Pavement Structure Classification

The Austroads *Guide to the Structural Design of Road Pavements* (Austroads 2004) contains procedures for the design of:

- flexible pavements consisting solely of unbound pavement materials;
- flexible pavements that contains one or more bound layers; and
- rigid pavements.

A flexible pavement consisting solely of unbound materials (natural crushed rock with no additional binding additive) transmits loads imposed at its surface to the subgrade level by a combination of contact pressure, mechanical interlock and cohesion between the particles. It achieves this through the use of materials which have some flexibility so that they deflect under load without cracking, and hence, without losing strength. The area over which the load is supported increases with depth, so that the stresses in the pavement decrease with the distance below the pavement surface. To achieve this base material is usually specially manufactured by mixing crushed quarry rock and fines.

In more recent years road constructors have found that the standard flexible pavement mix can be improved in strength and performance by the addition of small quantities of binders such as cement, bitumen, polymers and other similar additives. These pavements have come to the forefront in response to the increasing demands placed on the performance of the pavement with increasing traffic intensity and loading. They are constructed from natural manufactured material with a small percentage of the binding material added, typically 1% to 4% of the additive. Although still classified as flexible pavements, their failure mechanism has been found to be more complex and the design of these pavements requires detailed analysis rather than the empirical approach which can be used for unbound pavements. The advent of powerful computers has made the design of this type of pavement more widespread.

Rigid pavements consist of layers of plain or reinforced concrete constructed on top of the subgrade. They are referred to as rigid pavements due the stiffness of the pavement in relation to the subgrade.

Insitu stabilisation can be carried out to either of the flexible pavement types described above. Rigid pavements are not able to be rehabilitated in this way and will not be considered further for this report.

B.3 Design Methods

Traditionally, the design of pavement thickness has been carried out for flexible pavements without binders using the so-called Empirical Method. This method is based on the accumulation of experience of road authorities around the world and has provided a good guide for road engineers to predict the performance of a pavement structure. Austroads provides recommendations for the application of this method in Australia and the details of this method are described below.

With the advent of computers, mechanistic design methods have been developed to provide a more theoretical design based on an analysis of stresses developed at each boundary layer. These methods are expected to be of more general application, for example they can be used for analysing bound layers, although they must be used with care if the design technique has not been fully proven by actual results. The mechanistic method recommended by Austroads uses the CIRCLY program to analyse stresses through the pavement structure.

The mechanistic method is used for the design of the insitu stabilised pavements which are the subject of this report, and was also used for the analysis of the post-construction strengths of the pavements being studied. The method is described in more detail below.

B.4 California Bearing Ratio (CBR)

For traditional pavement design, the required thickness of a layer is determined by the strength of the underlying layer. The strength of unbound layers is traditionally measured by a quantity called the California Bearing Ratio (CBR).

The California Bearing Ratio (CBR) test measures the force needed to cause a 50 mm diameter plunger to penetrate 2.5 mm into a sample. It was developed by the US Corps of Engineers in the early 1940s and introduced into Australia after the Second World War. The original test was performed on a Californian crushed rock, to which a CBR value of 100 was assigned. The strength of other materials is proportionally related to that bearing capacity of the Californian crushed rock and is expressed as a percentage.

Typical subgrade material ranges from CBR 2 to 10, where the CBR 2 value would indicate a very poor quality material which would normally require some sort of additional treatment or the addition of select fill.

Typical subbase materials average around CBR 25 and base materials range from CBR 60 to 80. The greater the CBR value the thinner the pavement thickness required but at a higher cost for the supply.

The CBR value is used directly for the traditional or empirical design method, which is based on practical experience on the performance of pavements. However, the use of the CBR is so widespread that other properties required for different design methods are often deduced from the CBR value of the material.

In particular, the mechanistic analysis method described later requires the properties of the materials to be characterised by their elastic stiffness, or modulus. The elastic modulus, however is difficult to determine. As pavement layers are subjected to repetitive loading, the Repeated Load Triaxial test is considered the most appropriate laboratory test procedure for measuring elastic modulus but, because of the difficulty, is rarely done. To determine the modulus from the CBR value, the empirical relationship adopted for subgrade materials is:

$$E = 10 \times CBR$$

where E = elastic modulus (MPa)

CBR = California Bearing Ratio (%)

When determining the modulus from the CBR for subbase and base layers, different authorities use different relationships, however the MRD uses the conversion chart shown in Figure B-2. As can be seen, there is a wide spread in the results of the research carried out to determine this relationship.

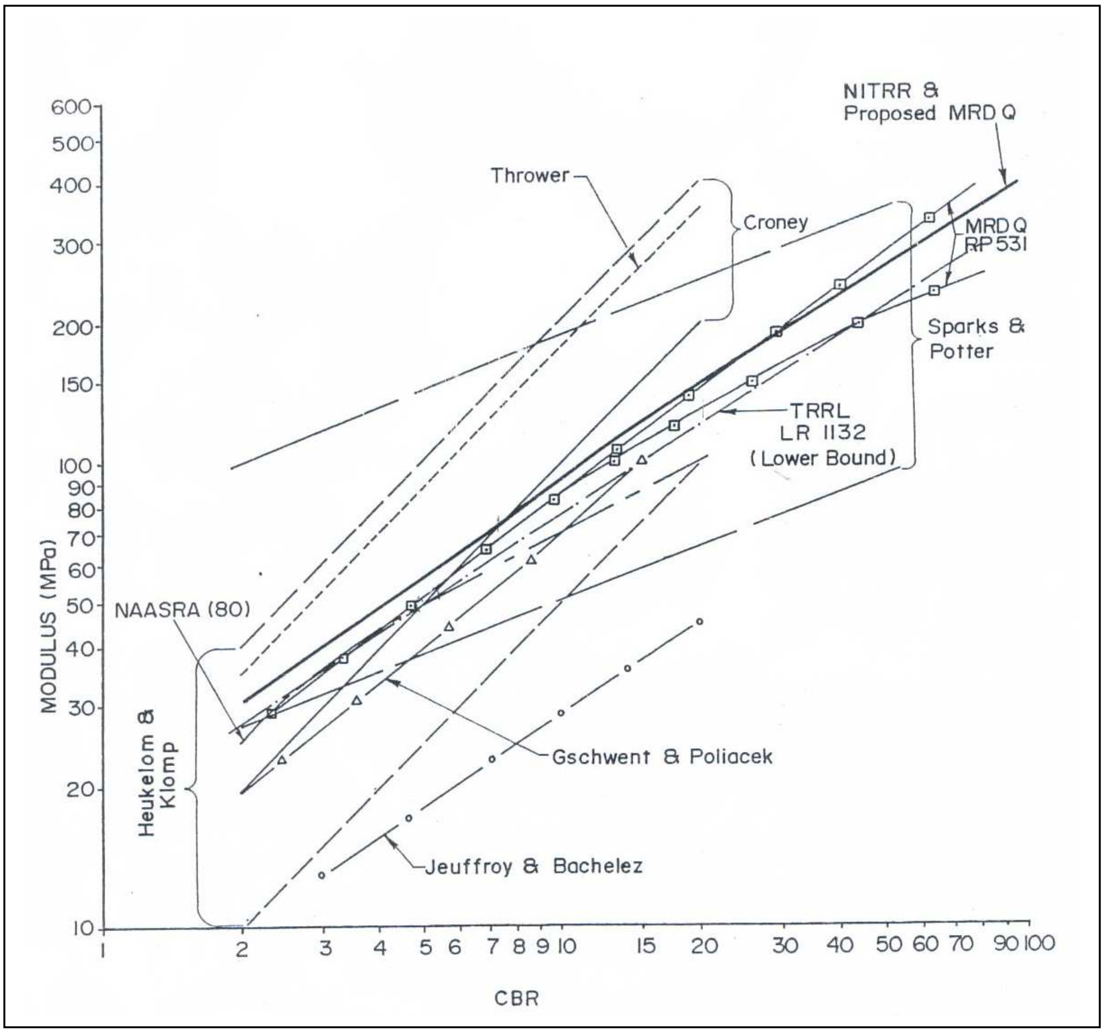


Figure B-2 - Summary of CBR vs Modulus Relationship
Source: *Main Roads Pavement Design Manual (1991)*

B.5 Unbound Material Properties

For heavily trafficked roads higher quality granular materials are required than for lightly trafficked roads. The high quality material is rarely found in their natural state and must be processed by crushing and sieving.

Crushers and screens are used to distribute the material according to their particle size. The different particle sizes are than mixed together in accurately determined proportions to give the desired grading for the strength required for the pavement layer.

The parameters that must be considered when selecting suitable unbound pavement materials are:

- grading (particle size distribution);
- particle shape;
- plasticity of the fine fractions;
- hardness of the source rock; and
- permeability and the ability to dissipate pore pressure developed under repetitive load.

Grading

The performance of the pavement is influenced by the proportions of fine and coarse fractions present. The coarse fractions are those retained on a 4.75 mm Australian Standard (AS) sieve, whilst those passing are termed fine fractions. Material passing the 75 µm AS sieve are referred to simply as fines or binder.

The unbound material which will form either a base or sub-base layer must be able to withstand the stresses imposed upon it, the graduation of coarse and fine fractions requires mixtures that achieve a high dry density. The particle size distribution generally is based on successively smaller particles filling the voids between adjacent coarser ones and touching them . Fuller showed that a granular mass has a relatively high dry density when the particle size distribution follows a certain rule, which is written:

$$\frac{p}{P} = \left[\frac{d}{D} \right]^n$$

where P = percentage of mass passing sieve d

P = percentage of mass passing sieve D

N = a value between 0.5 and 0.3.

The maximum densities are achieved when values for n are between 0.45 and 0.5. When the n is greater than 0.5, there are insufficient fines to fill the voids which can have the following effects:

- high stability in confined, low if unconfined;
- variable density;
- increased permeability;
- difficult to work and compact;
- not affected by adverse moisture conditions.

When n is less than 0.3 the reverse effect applies, the material contains too many fines resulting in:

- decreased strength and stiffness;
- reduced density;
- decreased permeability;
- increased tendency towards segregation and excess surface fines;
- strength affected by moisture;
- easy to work and compact.

Particle Shape

Particle shape is described by the ratio of length to thickness, flakiness, and length to width, elongation. The lower the proportion of flaky or elongated particles, the better the mechanical interlock. The optimum particle shape is angular and prismoidal.

Plasticity

In the early 1900's, Swedish chemist Albert Atterberg developed an empirical method of describing the changes in state of cohesive soils from liquid through plastic to solid. These change points are called the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) and together are called the Atterberg limits.

There is a close relationship between the limits and the properties of a soil such as compressibility, permeability, and strength. Atterberg also defined the plasticity index (PI) as a measure of the plasticity of a soil. The plasticity index is the range of water contents where the soil exhibits plastic properties, defined as the difference between the liquid limit and the plastic limit (PI = LL - PL). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 tend to have little or no silt or clay.

Plasticity is associated with the fines fraction (i.e finer than 0.425mm sieve). If the fines component is in excess and plastic it can cause an undesirable potential for volumetric expansion and contraction. The Plastic Index (PI) test, being sensitive to the amount of clay present, can be an indicator of the potential loss of stability that can occur due to the softening of the clay component when wetted. The linear shrinkage (LS) test is used to determine the type of plastic material present and will increase with the amount of organic and fibrous content.

The Plasticity Index is useful to give an indication of the bindability and workability of gravel mixes and their suitability as pavement material. Typical PI values will depend on the position of the layer in the pavement. Top base layers are normally constructed of high strength material, low in fines, relying mainly on internal friction between particles for its load bearing capacity and stability. The PI for this layer will generally have a maximum of 4%. Subbase material, which is lower in strength and higher in fines, relies on both internal friction and cohesion properties to achieve the required strength and stability. The PI will increase to 12% because of the increased percentage in fines.

Hardness of Source Rock

This property is measured by the Los Angeles Abrasion test in which the coarse stone hardness, toughness and soundness are factors inherent in the nature of the parent rock.

The softer the material, the higher the potential for breakdown and generation of fines – with a consequential loss of strength. The hardness is a factor determining the life expectancy, since breakdown over time results in a volume change, which displays itself as rutting.

Pore Pressure and Permeability

Pore pressure develops when the material is placed under repetitive load at a frequency that it can not dissipate between load cycles and only occur when sufficient moisture is present. Its effects include the exuding of fine material through cracked seal (pumping), allowing further moisture ingress and resulting in progressive pavement failure. Permeability is principally governed by the amount of material passing the 0.075 sieve. It is also particularly affected if the 0.002mm fraction is increased.

B.6 Flexible Pavement Design - Empirical Method

The Empirical Method is a traditional method based on the original research carried out by the Californian State Highway Department which resulted in the CBR design method based on the strength of the subgrade. (Jameson, G.W. 1996). The method was further developed in the United Kingdom and the United States and adopted with modifications by the Victorian Country Road Board which formed the basis for the current methodology adopted by Austroads.

This design method uses a design chart to enable determination of pavement layer thicknesses based on the strength of the underlying layer represented by its CBR value. The chart currently used in Australia is contained in the Austroads Pavement Design Manual - Figure 8.4. The caveat to this chart is that no provision is made for a limitation to the allowable design traffic caused by fatigue cracking of an asphalt surface - the chart is based on allowable design traffic in terms of rutting and shape loss. It may be used solely for pavements comprised of unbound layers of granular material which are surfaced with either a bituminous seal or a thin asphalt layer (less than 40mm). The design chart is reproduced as Figure B-3.

Design using this method requires a knowledge of the CBR of the subgrade and of the material to be used for the base layers, and the total number of equivalent standard axles expected over its design life – the design traffic. The only failure method considered is the failure of the subgrade causing rutting and the chart provides the information to determine the required thickness of the subbase and base to prevent the high stresses reaching the subgrade and causing failure.

The design traffic is the number of heavy vehicle axle groups (H_{VAG}) which have been converted to the number of equivalent standard axles (ESA) that will occur throughout the design life of the road. The calculation of this value is described in more detail in the Design Traffic section below.

The reason this method is not applicable to the design of bound flexible pavements is that these pavements have different failure modes, which are taken into account for the mechanistic method. However, for unbound granular pavements, the strength of this method is that it is actually based on the observed performance of pavements in service.

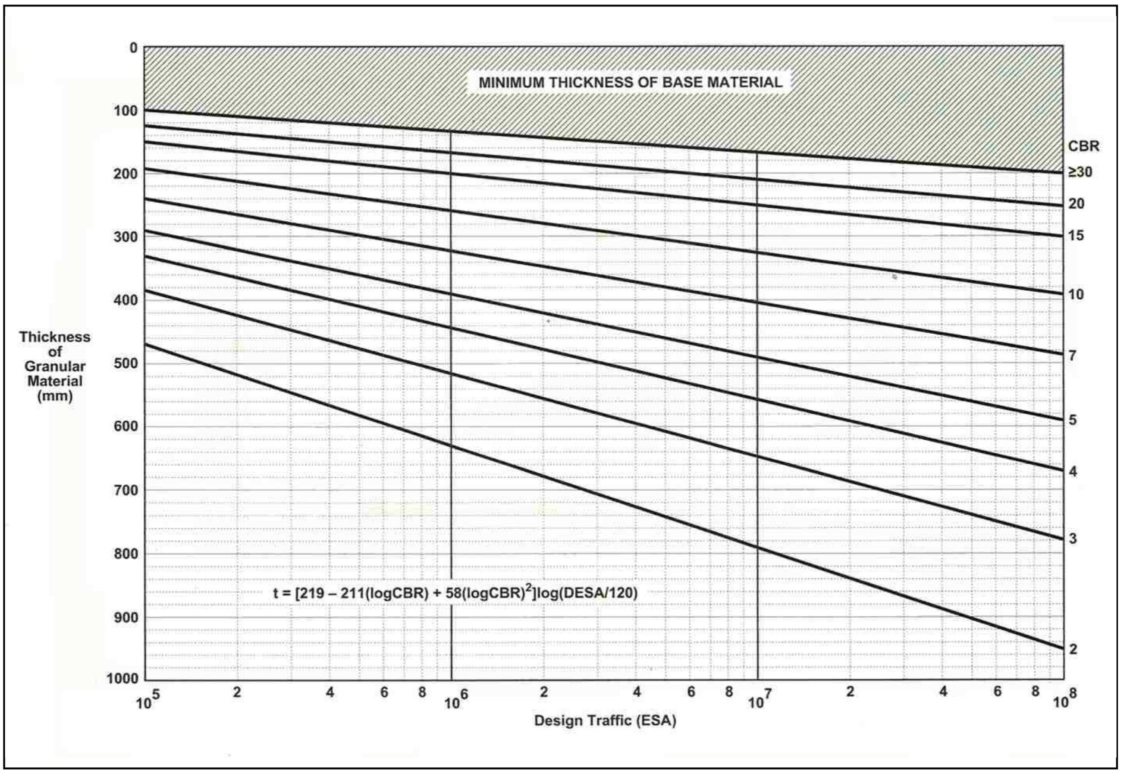


Figure B-3 - Empirical Road Pavement Design Chart
Source: *Austroads Pavement Design Guide (2004)*

B.7 Flexible Pavement Design - Mechanistic Method

The mechanistic method of design uses computer programs to analysis the performance of pavement layers based on a structural model of the pavement. The model is represented as shown in Figure B-4.

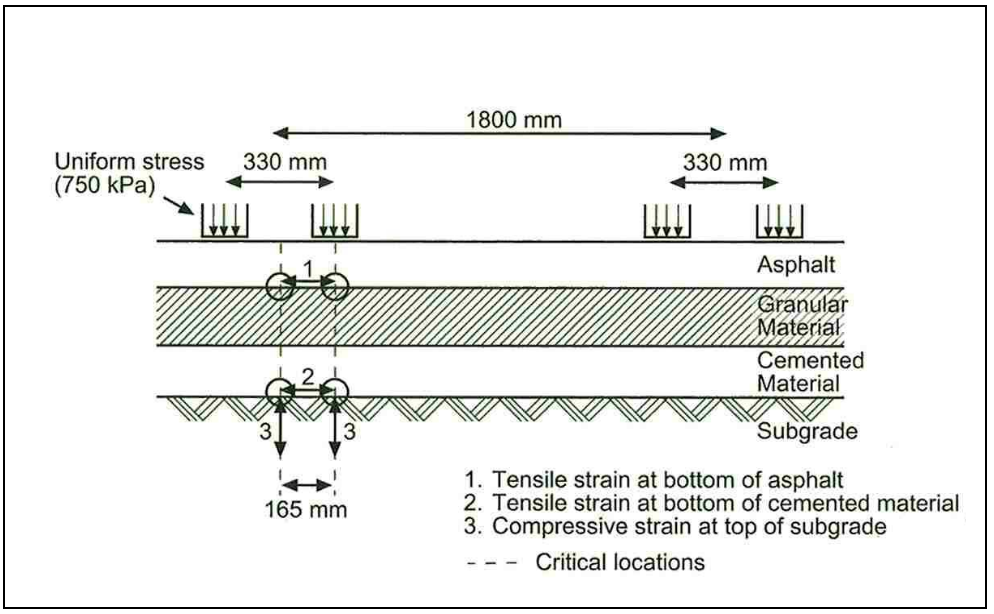


Figure B-4 - Failure Modes in Pavement Design
Source: *Austroads Pavement Design Guide (2004)*

Each layer is considered to be comprised of a homogeneous linearly elastic material which has found to provide a reasonable simulation of pavement behaviour. Each layer is characterised by its elastic stiffness properties ie modulus and Poisson's ratio.

There are a number of software programs available for linear elastic models but the program most commonly used in Australia is the CIRCLY program written in 1977 by Dr Leigh Wardle at CSIRO.

However, the mechanistic design model has not been validated for granular pavements having asphalt layers less than 40mm thick. The design model may suggest that pavements with thin asphalt surfacings can perform comparably to thick asphalt pavements at high traffic loadings.

To use the CIRCLY program a trial pavement design is entered and the program calculates the allowable repetitions for the three failure modes:

- tensile strain at bottom of asphalt;
- tensile strain at bottom of cemented material; and
- compressive strain at top of subgrade.

The program then compares these values with the estimated number of repetitions and provides a reading of the percentage of life consumed. If the design is unsatisfactory the design is then modified and the process repeated.

The design traffic is the number of heavy vehicle axle groups (H_{VAG}) which have been converted to a number of standard axle repetitions (SAR) that will occur throughout the design life of the road. The details are described in the Design Traffic section below.

To allow a greater confidence that the road will perform adequately over its design period, a reliability factor can also be introduced into the calculations. This allows for uncertainty in the estimate of traffic growth and loadings, variation in material properties, construction variability and the importance of the road itself. Typically, values as shown in Table B.1 are applied to the design.

Table B.1 - Project Reliability

Desired Project Reliability	80%	85%	90%	95%	97.5%
Reliability Factor	2.5	2.0	1.5	1.0	0.67

Source: *Austroads Pavement Design Guide (2004)*

The Mechanistic Method attempts to ascertain the point of failure by calculating the critical stresses and strains that occur throughout the multi-layered structure based on the linear elastic multi-layer theory (although methodologies exist based on visco-elastic and elatic-plastic theories).

The CIRCLY program uses the linear elastic multi-layer theory and has been adopted by Austroads as the standard mechanistic technique.

B.8 Design Traffic

Both the empirical and mechanistic design methods require an estimate of the total number of compressive actions caused by the wheels of vehicles to successfully design the pavement.

The empirical method uses the estimated value of “Equivalent Standard Axles” (ESA) while the mechanistic method uses “Standard Axle Repetitions” (SAR) for each failure mode and would normally be different for each of these modes. These values are taken over the design life of the pavement.

The calculation of these figures requires an estimate of volume of traffic traversing the pavement. Because the damage caused is a power relationship to the applied load, the damage caused by light passenger and similar vehicles is negligible, so an estimate of heavy vehicle traffic only is required. Commonly and historically, were only simplistic traffic counters were available, the loadings as based on a count of heavy vehicles, their assumed loadings and distribution percentage of different heavy vehicle type. Hence, it is common to estimate a value designated heavy vehicle axle groups, N_{DT}, as the first step in calculating the required traffic parameters.

The basic method for calculating N_{DT} as proposed by Austroads is the following formula:

$$N_{DT} = 365 \times (AADT \times DF) \times \frac{\% HV}{100} \times N_{HVAG} \times LDF \times CGF$$

where:

where:	AADT	=	Average annual daily traffic (vehicles per day)
	DF	=	Direction Factor is the proportion of the two-way AADT travelling in the direction of the design lane.
	%HV	=	Average percentage of all traffic comprising heavy vehicles.
	N_{HVAG}	=	Average number of axle groups per heavy vehicles
	LDF	=	Lane Distribution Factor
	CGF	=	Cumulative Growth Factor

Annual Average Daily Traffic (AADT)

The total traffic passing a point throughout the year divided by 365 is the annual average daily traffic volume (AADT). It is usually estimated by using a traffic counter for a two week period, three times a year, but on rare occasions a permanent counter may have been installed.

Direction Factor (DF)

The direction factor allows the adjustment of the count depending on whether it is counting single traffic or traffic in both directions.

Percentage of Heavy Vehicles (%HV)

This is the average percentage of heavy vehicles from the annual average daily traffic. The percentage of heavy vehicles is taken because light vehicles contribute very little to the structural deterioration of the pavement (Austroads 2004).

Heavy Vehicle Axle Groups (N_{HVAG})

This is the average number of axle groups per heavy vehicles. In the absence of specific counter data this value would be estimated from a knowledge of the type of traffic using the road or by carrying out short term spot observations.

Lane Distribution Factor (LDF)

This is the proportion of the traffic volume assigned to the heaviest trafficked lane which becomes the design lane. This factor only applies to multi-lane carriageways where traffic volumes can vary significantly. Where the roads are two lane, one lane for traffic travelling in each direction, the Lane Distribution Factor (LDF) will be 1.

Cumulative Growth Factor (CGF)

The design is based on the total amount of heavy vehicle axle groups that will travel over the pavement for its life. Examination of historical data will give an idea of the trends that have occurred and can be used as a starting point for forecasting. However, there is a need to research the economic development that is occurring in the area that will influence the traffic volumes on the road.

Simplistically, if traffic is forecast to grow at a certain percent each year, a factor can be determined by using the exponential growth equation, with which to multiply the starting traffic volume to determine the total traffic over the required period. Obviously, the designer may have to modify this factor to account for step growth in traffic volumes due to specific developments that are likely to occur in the region or alternately postulate a higher growth factor.

The formula for calculating the cumulative growth factor assuming constant growth as recommended by Austroad is:

$$\text{Cumulative Growth Factor (CGF)} = \frac{(1 + 0.01R)^{P} - 1}{0.01R}$$

where:

R = Growth Rate (%)

P = Design Period (years)

A typical annual growth rate for rural roads and highways within the Mackay District is 5%.

Design Period (P)

The design period used in the cumulative growth factor is the time span that the pavement is expected to function without any need for major rehabilitation or reconstruction works. In determining a design period consideration must be given to:

- available funds for the project;
- importance of the road;
- likely future upgrading to improve the capacity of the road;
- reactive subgrades, consolidation of fill material or compressibility of the soil strata that will cause distress resulting in the requirement for rehabilitation or reconstruction work; and
- existing fixed levels such as, kerb or overhead structures, constraining the selection of rehabilitation treatments to more costly options.

A typical design period for flexible pavements is 20 - 40 years. It is import to realise that the pavement is designed to provide satisfactory service over this design period, and this can only be expected if the actual traffic volumes and loadings do not exceed the estimated traffic volumes and loading.

Once having calculate the N_{DT} this figure has to be convert to the design figures required for the relevant pavement design method, viz Equivalent Standard Axles (ESA) for the Empirical Method and the three values of the Standard Axle Repetitions for the Mechanistic Method. The determination of these figures is described in more detail below, however an estimate of the distribution of heavy vehicles is required to determine these parameters.

B.9 Traffic Data Collection

The methods for collecting traffic data range from the simplistic to the advance.

Manual Traffic Counting

A manual traffic count involves people counting the traffic that passes a particular point over a certain period of time. It can also be used for assessing the percentage of heavy vehicles. This method is very labour intensive and consequently not used very often.

Single Tube Axle Counters

Single tube counters use a air filled tube connected to a control box that uses the air pressure pulses and a computer program to estimate the traffic numbers and provides a rough percentage of heavy vehicles.

Vehicle Classification Counters

These counters consist of two air filled tubes connected to a black control box on the side of the road recording the air pressure when something runs over the tubes. A computer program is used to assess the information downloaded from the black control box. They can provide information such as speed of vehicle, traffic flow at different times in the day and vehicle classification types (using wheel speed x time to work out axle spacings). The vehicle classes of which the output is based are shown in Figure B-5. Consequently, the number of each axle group types can be calculated from this data, however no data about the actual loads on each axle group type is available.

Weigh-In-Motion Systems

These weigh-in-motion systems are used to determine axle group configurations and loadings. They collect the axle load and configuration data while the vehicle travels over sensors installed into the pavement. They can be used to provide excellent estimates of traffic in terms of equivalent standard axles. Unfortunately units are expensive to install and maintain, so there are relatively few installed. However, the information gathered on the few permanent sites on axle loadings can be used as indicative loadings for axles throughout the region.

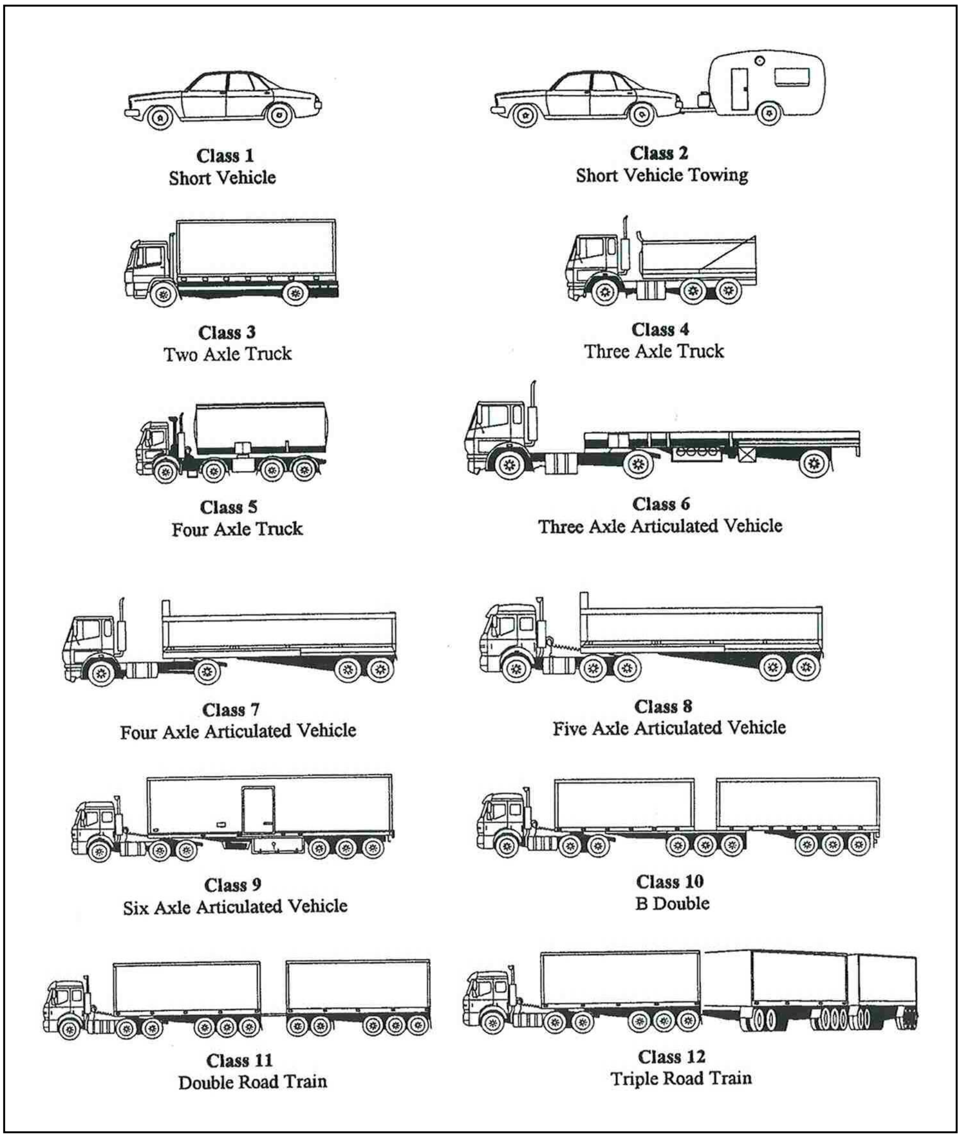


Figure B-5 - Classification of Vehicles
Source: *Austroads Pavement Design Guide (2004)*

B.10 Imposed Axle Loadings

The empirical and mechanistic design methods require the heavy vehicle axle groups to be convert to Equivalent Standard Axles (ESA) or Standard Axle Repetitions (SAR).

Equivalent Standard Axles (ESA) / Standard Axle Repetitions (SAR)

Both design methods are based on the concept of a standard axle which is:

> *The standard axle consists of a single axle with dual wheels carrying a load of 80 kN. The circular contact stress being applied to the pavement at 330mm centres over each dual wheel is 750 kPa for highway traffic.*

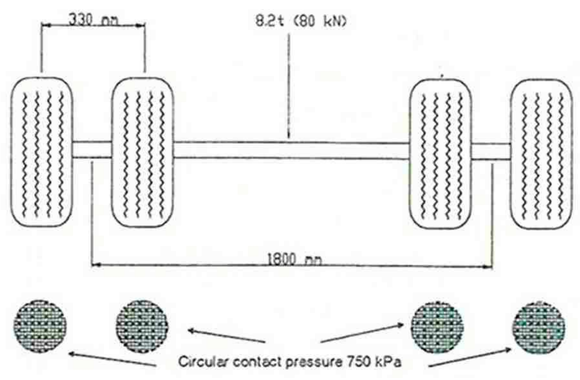


Figure B-6 - Standard Axle
Source: *Pavement Design Training Manual (MRD)*

Experimental work has determined that different axle profiles can take different loads to cause the same amount of damage as a standard axle (Table B.2).

Table B.2 -
Axle Load Values Equivalent to a Standard Axle

Axle Group Type	Load (kN)
Single Axle with Single Tyres (SAST)	53
Single Axle with Dual Tyres (SADT)	80
Tandem Axle with Single Tyres (TAST)	90
Tandem Axle with Dual Tyres (TADT)	135
Triaxle with Dual Tyres (TRDT)	181
Quad-axle with Dual Tyres (QADT)	221

Source: *Austroads Pavement Design Guide (2004)*

If an axle group is loaded to a different loading from that shown in Table B.2 it is necessary to calculate the equivalence in terms of the standard axle.

Experiments have determined that the equivalence obeys the following formula:

$$EA = \left(\frac{L}{SL} \right)^{m}$$

where:

EA	=	Equivalent number of standard axles
L	=	Actual load of axle group
SL	=	Standard load for that axle group
m	=	An exponent depending on the method of failure

For the empirical method, design is based on the strength of the subgrade and the exponent is 4. The mechanistic method, uses three failure modes, fatigue of the asphalt layer (exponent of 5), rutting/shape loss (exponent of 7) and fatigue of cement material layer (exponent of 12).

Obviously this is a very vigorous calculation which requires an extensive knowledge of the traffic volumes for the forecast period as well as accurate details of the heavy vehicle loadings and axle types. Consequently, the standard axle loadings used in practice can only be a best estimate, based on current data and a forecast of the economic activity of the surrounding area in the future. Hence, most organisations tend to use predetermined average factors for each of the calculations.

B.11 Application to Insitu Stabilisation

This project deals with roads that have been designed and built as flexible pavements with unbound material more than twenty years ago using the empirical design method. Portions of these roads have reached the end of their service life and are showing signs of fatigue and pavement wear. The increase in heavy vehicular traffic as a result of the coal mining industry has accelerated the deterioration.

The roads can be rehabilitated by completely rebuilding the road, or, if the existing pavement material is suitable, by insitu stabilisation of the top layer. Where insitu stabilisation is to be considered, the existing road pavement material is tested to determine whether the less expensive cement insitu stabilisation process may be applicable.

The following tests provide the information required.

- The subgrade material is evaluated using a Dynamic Cone Penetrometer. It allows for a CBR value to be postulated for the natural material by plotting the penetration of the cone against the number of drops of the weight. The level of saturation of the subgrade is also determined as this point so that an assessment can be made on whether the DCP result is the worse case scenario, as CBR will be higher in drier conditions.
- The Particle Size Distribution and Atterberg Limits of the subgrade and each of the pavement layers aids in the determination of the most suitable type of stabilisation method and in the classification of material types for CBR testing. The particle size distribution and plasticity index (PI) from the Atterberg Limits most appropriate for cement insitu stabilisation is that the quantity of material passing the 75μm sieve should be less than 25% and a large PI range. The laboratory CBR test is expensive and time consuming therefore if the mateials are classified into groups of similar properties and gradings it reduces the number of CBR tests required for the determination of moduli.
- Soaked CBR tests are performed in wet coastal regions because the subgrade and pavement layers are likely to be saturated for a substantial period and saturated CBR value is more relevant.

APPENDIX C – RISK MANAGEMENT

Risk Management Charts appropriate to this project are required for the visual inspection of the pavements for the selected sites, and for the operation of the Falling Weight Deflectometer testing rig. Tables C.1 and C.2 relate to the visual inspection, Tables C.2 and C.3 to the Falling Weight Deflectometer testing.

Figure C-1 shows the temporary signage and its location on either side of the work site.

Risk Management Chart for Visual Assessment of Project Pavements

Description of Hazards	People at Risk	Number at Risk	Parts of Body	Risk Level
Working Outdoors	2	2	Face, arms and legs	Minor

<table>
<tr><th>Categories</th><th>Short Term Control</th><th>Long Term Controls</th><th>Completion Details</th></tr>
<tr><td>P.P.E</td><td>- Wear broad brimmed hat, long sleeved shirt and long trousers.
- Wear Safety glasses at all times
- Apply 30+ sunscreen liberally on exposed areas
- Wear steel capped boots</td><td>- Limit exposure as much as possible</td><td rowspan="2">Employer: RoadTek
Prepared by: Mark Weatherley
Date: 14/05/07
Assented to by: Rodney Smith
Position: WH&S Officer

Signature:

Date:</td></tr>
<tr><td>Thermal – hot cold ambient temperatures</td><td>- Wear PPE described above at all times and drink plenty of water.
- Rotate shifts of work to cooler part of the day, if necessary.</td><td>- Limit exposure as much as possible</td></tr>
</table>

Table C.1 - Risk Management Chart - Visual Assessment of Project Pavements
Working Outdoors

Risk Management Chart for Visual Assessment of Project Pavements

Description of Hazards	People at Risk	Number at Risk	Parts of Body	Risk Level
Struck by Vehicle whilst Working within Gaps in Traffic from M.U.T.C.D	2	2	Whole Body	Major

Categories	Short Term Control	Long Term Controls	Completion Details
P.P.E	‣ Wear high visibility clothing	‣ Wear high visibility clothing	Employer: RoadTek Prepared by: Mark Weatherley Date: 14/05/07 Assented to by: Rodney Smith Position: WH&S Officer Signature: Date:
Separation	‣ Park work vehicle clear of travelling lane. ‣ Ensure flashing light on work vehicle is operating to warn approaching traffic. ‣ Place a look out person so that he can see traffic approaching in both directions from a distance of 200m. ‣ If traffic density too high either vary working times to avoid the high density traffic or use traffic controllers to stop traffic whilst performing inspections.	‣ Limit exposure as much as possible	

Table C.2 - Risk Management Chart - Visual Assessment of Project Pavements
Traffic

Risk Management Chart for Falling Weight Deflectometer Testing

Description of Hazards	People at Risk	Number at Risk	Parts of Body	Risk Level
Struck by Vehicle whilst performing FWD testing	4	4	Whole Body	Major

Categories	Short Term Control	Long Term Controls	Completion Details
P.P.E	‣ Wear high visibility clothing	‣ Wear high visibility clothing	Employer: RoadTek Prepared by: Mark Weatherley Date: 14/05/07 Assented to by: Rodney Smith Position: WH&S Officer Signature: Date:
Separation	‣ Place appropriate advanced warning signage as per the requirements of the M.U.T.C.D. before commencing the testing. ‣ Traffic Controllers will be position to isolate the FWD trailer and the personnel from the travelling public. ‣ Delineate the travelling corridor around the FWD trailer with traffic cones.	‣ Limit exposure as much as possible	

Table C.3 - Risk Management Chart - Falling Weight Deflectometer Testing
Hazard from Traffic

Risk Management Chart for Falling Weight Deflectometer Testing

Description of Hazards	People at Risk	Number at Risk	Parts of Body	Risk Level
Operating FWD testing Machine	4	4	Whole Body	Major

Categories	Short Term Control	Long Term Controls	Completion Details
P.P.E	▸ Wear protective gloves when setting up the trailer for testing to prevent cut hazards. ▸ Correct manual handling technique to be used. ▸ Correct footwear to be worn. ▸ Appropriate rated hearing protection to be worn (Sound – 85dBa over 8 hrs) ▸ Workers to be aware of heat stress and ensure that fluid intake is adequate when working in a hot environment.	▸ Limit exposure as much as possible	Employer: RoadTek Prepared by: Mark Weatherley Date: 14/05/07 Assented to by: Rodney Smith Position: WH&S Officer Signature: Date:
Separation	▸ Machine to be controlled by a ticketed operator. ▸ Personnel to stand clear of the underside hammer like sections when the machine is operating.	▸ Limit exposure as much as possible	

Table C.4 - Risk Management Chart - Falling Weight Deflectometer Testing

Hazard from Operating Machinery

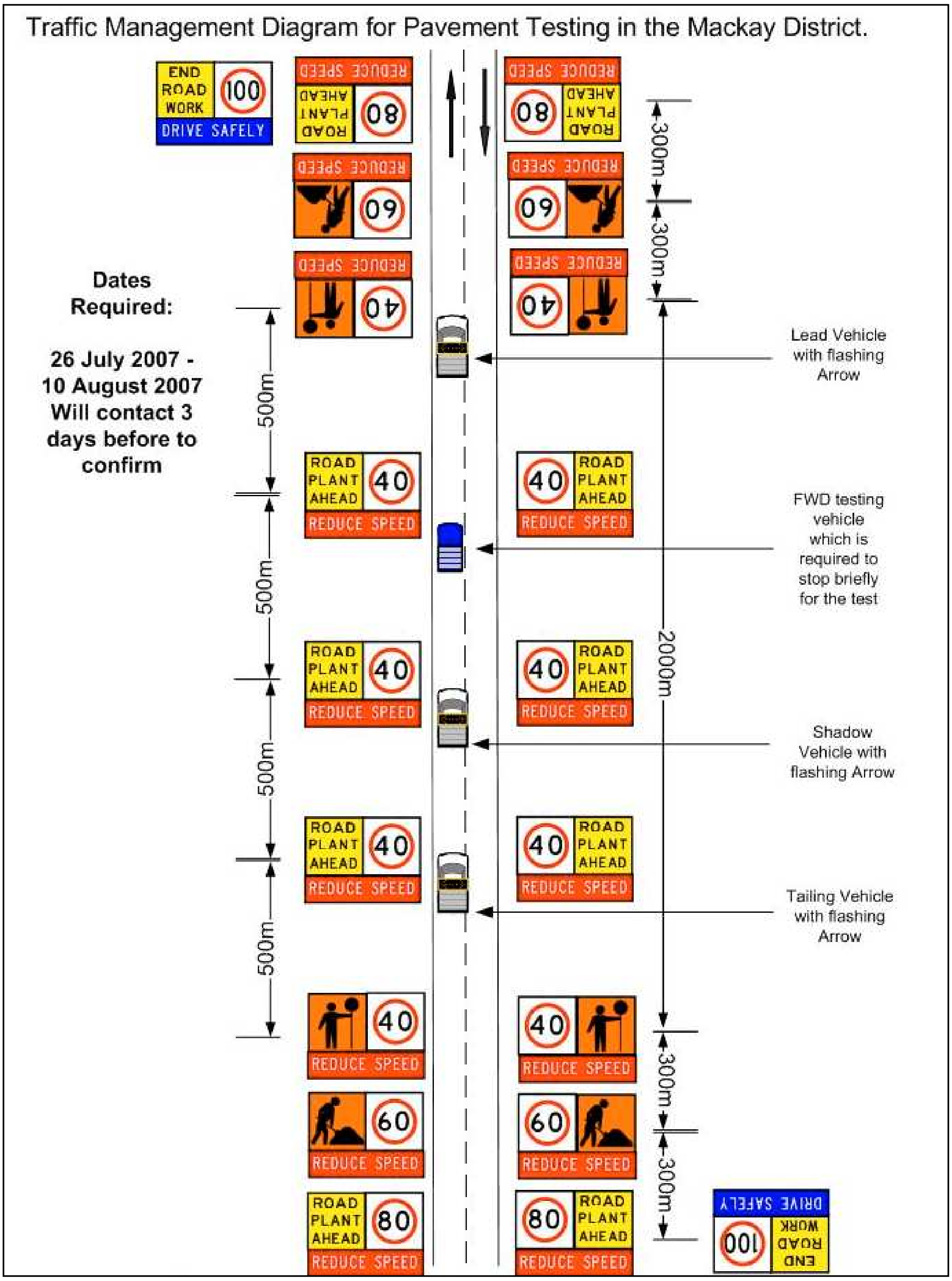


Figure C-1 - Pavement Testing Traffic Signage Arrangement Diagram

APPENDIX D – ACCEPTANCE TEST RESULTS

Table D.1 lists the Relative Dry Density (RDD) acceptance test results for all insitu stabilisation projects carried out in the Mackay District over approximately the previous eighteen months, for which detailed test data was retained. As the budget for Falling Weight Deflectometer (FWD) testing was limited, each job was reviewed and approximately 50 percent of the test sites were selected for FWD testing. Selection was based on the rehabilitation being generally for normal traffic lanes rather than for lane widening, and where a significant depth of insitu material was included in the rehabilitation. In addition, where large projects showed generally similar results, typical sample lots were selected rather than including all lots.

Test locations which have been rejected for FWD testing and analysis as part of this project are shaded in the table.

Table D.2 allocates Site ID numbers to the separate road sections selected for FWD testing as part of this project, and cross-references the Site ID numbers to the original MRD Job Numbers.

Table D.3 summarises the RDD quality control acceptance test results for each of the selected lots, allocates lot identification numbers to each lot, and defines the start and end chainages. Each site is subdivided into Lots, each lot covering a section of road rehabilitation which was completed in a single day. Figure D-1 shows a diagrammatic representation of this location information.

Table D.4 lists each RDD test site location in the lots selected for FWD testing as part of this investigation. RDD tests were carried out for each lot in each project site at the locations shown in the table, and these locations were used to locate comparison FWD tests.

Table D.1 - Raw Acceptance Test Data - All Sites

MRD Job No	Test No.	Side L/R	Lot	Chain m	Offset m	RDD * %	CV ** %	Pass Y/N
90/33A/806	383	R	PS01	153158	3.0	94.4		
90/33A/806	382	R	PS01	153313	0.2	95.3		
90/33A/806	381	R	PS01	153462	1.7	98.2		
90/33A/806	380	R	PS01	153588	3.6	91.6		
90/33A/806	326	R	PS01	153755	0.8	99.0		
90/33A/806	325	R	PS01	153871	3.0	96.6		
90/33A/806	324	R	PS01	154115	4.4	93.2		
90/33A/806	323	R	PS01	154204	1.7	95.2		
90/33A/806	322	R	PS01	154365	1.6	97.1	93.7	N
90/33A/806	384	L	PS02	153103	2.2	94.9		
90/33A/806	385	L	PS02	153221	1.4	93.0		
90/33A/806	386	L	PS02	153537	2.4	96.9		
90/33A/806	387	L	PS02	153602	2.2	100.4		
90/33A/806	330	L	PS02	153821	0.5	95.1		
90/33A/806	329	L	PS02	153946	2.1	98.9		
90/33A/806	328	L	PS02	154186	3.5	97.4		
90/33A/806	327	L	PS02	154441	2.0	96.4	94.8	N
90/33B/304	080.1	L	PS05	38915	3.3	102.9		
90/33B/304	080.5	L	PS05	39157	3.2	97.7		
90/33B/304	080.3	L	PS05	39417	2.4	98.7		
90/33B/304	080.4	L	PS05	39674	0.7	100.7	98.6	N
90/33B/304	082.1	R	PS06	39042	2.2	100.0		
90/33B/304	082.2	R	PS06	39233	1.0	103.3		
90/33B/304	082.3	R	PS06	39399	1.8	97.4		
90/33B/304	082.4	R	PS06	39553	3.2	102.7	99.2	N
90/33B/304	084.1	L	PS07	39911	1.9	99.7		
90/33B/304	084.2	L	PS07	40206	0.6	95.9		
90/33B/304	084.3	L	PS07	40295	3.9	97.1		
90/33B/304	084.5	L	PS07	40562	0.9	95.0	95.7	N
90/33B/304	96.1	R	PS08	39868	1.6	98.5		
90/33B/304	96.2	R	PS08	40012	3.5	97.9		
90/33B/304	96.3	R	PS08	40439	2.6	98.7		
90/33B/304	96.4	R	PS08	40552	1.9	98.6	98.2	N
90/33B/304	120.7	L	PS09	40809	2.6	98.5		
90/33B/304	120.2	L	PS09	41006	3.8	98.7		
90/33B/304	120.6	L	PS09	41300	1.7	99.1		
90/33B/304	120.4	L	PS09	41420	3.2	98.0	98.3	N
90/33B/304	122.5	R	PS10	40815	2.5	98.4		
90/33B/304	122.6	R	PS10	40906	1.9	99.5		
90/33B/304	122.3	R	PS10	41216	1.4	95.6		
90/33B/304	122.7	R	PS10	41517	2.2	100.0	97.2	N
90/33B/304	139.5	L	PS11	41602	2.3	98.0		
90/33B/304	139.2	L	PS11	41983	3.3	98.0		

MRD Job No	**Test No.**	**Side L/R**	**Lot**	**Chain m**	**Offset m**	**RDD * %**	**CV ** %**	**Pass Y/N**
90/33B/304	139.3	L	PS11	42051	1.1	99.1		
90/33B/304	139.4	L	PS11	42340	1.1	96.0	97.0	N
90/33B/304	141.1	R	PS12	41754	2.8	98.4		
90/33B/304	141.2	R	PS12	41828	2.1	99.5		
90/33B/304	141.3	R	PS12	42188	1.9	98.6		
90/33B/304	141.4	R	PS12	42425	0.9	100.3	98.7	N
90/33B/304	143.1	L	PS13	42483	2.8	98.5		
90/33B/304	143.2	L	PS13	42860	2.5	100.4		
90/33B/304	143.3	L	PS13	43100	3.4	96.0		
90/33B/304	143.4	L	PS13	43228	2.9	99.4	97.4	N
90/33B/304	145.5	R	PS14	42588	0.5	99.2		
90/33B/304	145.2	R	PS14	42696	2.5	97.7		
90/33B/304	145.3	R	PS14	42978	0.3	98.6		
90/33B/304	145.6	R	PS14	43157	1.5	99.2	98.2	N
90/33B/304	961/7	R	PS01	43458	0.7	98.3		
90/33B/304	961/2	R	PS01	43604	3.5	99.0		
90/33B/304	961/8	R	PS01	43927	2.6	97.2		
90/33B/304	961/4	R	PS01	44146	4.1	97.5		
90/33B/304	961/5	R	PS01	44366	3.4	98.5		
90/33B/304	961/6	R	PS01	44517	3.1	101.6	97.6	N
90/33B/304	967/1	L	PS02	43514	1.4	97.6		
90/33B/304	967/7	L	PS02	43652	2.6	101.0		
90/33B/304	967/3	L	PS02	43802	0.2	101.7		
90/33B/304	967/4	L	PS02	44014	3.4	100.1		
90/33B/304	967/5	L	PS02	44309	0.7	96.9		
90/33B/304	967/6	L	PS02	44587	3.7	100.7	98.3	N
90/33B/304	1048.1	R	PS03	44685	4.0	98.5		
90/33B/304	1048.2	R	PS03	44951	1.6	99.1		
90/33B/304	1048.3	R	PS03	45209	3.8	98.2		
90/33B/304	1048.4	R	PS03	45318	0.6	97.5		
90/33B/304	1048.5	R	PS03	45497	3.2	99.4		
90/33B/304	1048.6	R	PS03	45877	0.3	100.6		
90/33B/304	1048.7	R	PS03	46067	1.1	102.1	98.2	N
90/33B/304	1061.1	L	PS04	44826	3.9	101.5		
90/33B/304	1061.2	L	PS04	44922	0.8	99.9		
90/33B/304	1061.3	L	PS04	45163	4.5	95.6		
90/33B/304	1061.4	L	PS04	45429	0.7	102.6		
90/33B/304	1061.5	L	PS04	45628	2.8	99.4		
90/33B/304	1061.6	L	PS04	45693	1.4	100.7		
90/33B/304	1061.7	L	PS04	45934	0.8	98.3	98.0	N
120/33B/305	0882	L	PS01	65919	3.9	101.9		
120/33B/305	0866	L	PS01	65998	1.2	98.4		
120/33B/305	0867	L	PS01	66151	1.9	98.3		
120/33B/305	0868	L	PS01	66258	0.5	97.6		
120/33B/305	0869	L	PS01	66290	2.9	105.1		

MRD Job No	Test No.	Side L/R	Lot	Chain m	Offset m	RDD * %	CV ** %	Pass Y/N
120/33B/305	0870	L	PS01	66412	0.9	102.9	98.5	N
120/33B/305	0878	R	PS02	65863	3.8	99.5		
120/33B/305	0879	R	PS02	65960	3.6	102.1		
120/33B/305	0880	R	PS02	66071	1.6	98.4		
120/33B/305	0881	R	PS02	66181	0.9	98.0		
120/33B/305	0875	R	PS02	66361	6.7	96.1		
120/33B/305	0876	R	PS02	66454	3.0	100.4	97.6	N
120/33B/305	0871	L	PS03	66837	3.3	101.8		
120/33B/305	0872	L	PS03	66913	1.3	100.0		
120/33B/305	0873	L	PS03	66980	4.2	102.8		
120/33B/305	0874	L	PS03	67171	1.3	100.4	100.5	Y
120/33B/305	0926	R	PS04	66826	2.5	93.1		
120/33B/305	0863	R	PS04	67028	4.0	101.4		
120/33B/305	0864	R	PS04	67045	3.7	100.7		
120/33B/305	0927	R	PS04	67254	4.4	99.3	96.3	N
107/517/301	865	L	PS01	2334	1.8	100.8		
107/517/301	866	L	PS01	2596	0.6	98.0		
107/517/301	867	L	PS01	2660	3.6	102.7		
107/517/301	887	L	PS01	2913	0.9	99.5	99.0	N
107/517/301	872	R	PS02	2296	1.5	102.1		
107/517/301	873	R	PS02	2424	3.3	97.5		
107/517/301	874	R	PS02	2723	2.5	98.1		
107/517/301	875	R	PS02	2904	1.8	97.9	97.6	N
107/517/301	888	L	PS03	3115	2.9	100.3		
107/517/301	889	L	PS03	3314	1.0	98.5		
107/517/301	890	L	PS03	3495	3.2	99.2		
107/517/301	897	L	PS03	3665	1.1	93.5	96.0	N
107/517/301	892	R	PS04	3122	0.5	96.9		
107/517/301	893	R	PS04	3351	3.5	97.9		
107/517/301	894	R	PS04	3544	0.9	100.7		
107/517/301	895	R	PS04	3718	1.5	93.0	95.2	N
20/519/802	15332	L	PS01	28579	1.9	95.2		
20/519/802	15323	L	PS01	28929	1.5	96.6		
20/519/802	15324	L	PS01	29035	0.5	96.1	95.7	N
20/519/802	15325	R	PS02	28643	0.4	94.0		
20/519/802	15326	R	PS02	28726	2.1	95.5		
20/519/802	15327	R	PS02	29062	1.8	98.4	95.2	N
20/519/802	15329	L	PS03	29255	1.3	98.7		
20/519/802	15331	R	PS03	29313	0.5	101.9		
20/519/802	15328	L	PS03	29486	1.1	98.6		
20/519/802	15330	R	PS03	29521	2.3	99.4	98.7	N
82/533/303	15521	L	PS01	492	1.7	102.9		
82/533/303	15522	L	PS01	546	2.8	100.8		
82/533/303	15523	L	PS01	730	0.3	105.9		

MRD Job No	Test No.	Side L/R	Lot	Chain m	Offset m	RDD * %	CV ** %	Pass Y/N
82/533/303	15540	L	PS01	908	4.3	100.3		
82/533/303	15525	L	PS01	1011	0.9	102.0		
82/533/303	15541	L	PS01	1125	4.7	99.7	100.3	Y
82/533/303	15515	R	PS02	485	4.7	100.4		
82/533/303	15516	R	PS02	573	5.0	103.3		
82/533/303	15517	R	PS02	668	1.0	102.9		
82/533/303	15518	R	PS02	882	3.1	100.6		
82/533/303	15542	R	PS02	990	3.0	100.3		
82/533/303	15520	R	PS02	1166	4.4	101.8	100.6	Y
82/533/303	15473	L	PS03	1883	0.9	100.7		
82/533/303	15474	L	PS03	2029	4.0	100.2		
82/533/303	15475	L	PS03	2186	3.3	101.0		
82/533/303	15503	L	PS03	2341	2.0	99.4		
82/533/303	15501	L	PS03	2497	2.6	102.8		
82/533/303	15504	L	PS03	2635	2.6	99.6		
82/533/303	15502	L	PS03	2741	1.1	100.1	99.7	N
82/533/303	15490	R	PS04	1884	3.1	100.6		
82/533/303	15491	R	PS04	2098	0.2	100.0		
82/533/303	15492	R	PS04	2154	2.6	101.4		
82/533/303	15493	R	PS04	2313	0.6	102.2		
82/533/303	15494	R	PS04	2506	2.0	100.4		
82/533/303	15495	R	PS04	2563	4.9	101.8		
82/533/303	15496	R	PS04	2700	5.0	101.8	100.5	Y
82/533/303	15453	L	PS05	2813	3.6	99.4		
82/533/303	15454-1	L	PS05	3030	1.8	101.5		
82/533/303	15455	L	PS05	3171	1.6	103.0		
82/533/303	15456	L	PS05	3253	3.6	100.1		
82/533/303	15464	L	PS05	3434	3.0	102.8		
82/533/303	15465	L	PS05	3559	3.0	102.7		
82/533/303	15466	L	PS05	3722	1.6	99.6	100.1	Y
82/533/303	15457	R	PS06	2906	1.5	102.7		
82/533/303	15458	R	PS06	2942	0.3	100.6		
82/533/303	15459	R	PS06	3144	3.0	106.9		
82/533/303	15460	R	PS06	3280	2.5	102.1		
82/533/303	15461	R	PS06	3412	0.5	99.9		
82/533/303	15462	R	PS06	3611	3.7	101.8		
82/533/303	15463	R	PS06	3678	0.9	99.8	100.1	Y
82/533/304	452	R	PS01	3812	2.1	101.9		
82/533/304	506	R	PS01	4027	4.0	99.6		
82/533/304	454	R	PS01	4087	1.2	102.5	100.5	Y
82/533/304	446	L	PS02	3820	2.9	97.5		
82/533/304	447	L	PS02	3904	0.3	103.3		
82/533/304	448	L	PS02	4088	1.7	102.2	99.4	N
82/533/304	475	R	PS03	4356	2.8	104.2		
82/533/304	476	R	PS03	4641	2.3	103.7		

MRD Job No	Test No.	Side L/R	Lot	Chain m	Offset m	RDD * %	CV ** %	Pass Y/N
82/533/304	477	R	PS03	4754	0.8	104.3	103.9	Y
82/533/304	469	L	PS04	4356	3.8	100.1		
82/533/304	470	L	PS04	4654	1.3	99.7		
82/533/304	471	L	PS04	4756	2.0	103.9	100.0	Y
82/533/304	485	R	PS05	5001	3.2	103.6		
82/533/304	486	R	PS05	5061	2.3	101.5		
82/533/304	487	R	PS05	5243	2.8	98.5	99.8	N
82/533/304	479	L	PS06	4981	1.1	103.7		
82/533/304	480	L	PS06	5060	8.0	100.7		
82/533/304	481	L	PS06	5321	11.4	100.2	100.5	Y
82/533/304	511	R	PS07	5454	2.6	99.8		
82/533/304	512	R	PS07	5630	1.5	99.7		
82/533/304	513	R	PS07	5813	3.2	98.6	99.0	N
82/533/304	491	L	PS08	5359	2.9	97.3		
82/533/304	492	L	PS08	5645	1.7	102.6		
82/533/304	493	L	PS08	5921	1.3	105.9	99.6	N
82/533/304	731	L	PS09	6004	2.0	97.6		
82/533/304	732	L	PS09	6424	2.4	99.2		
82/533/304	733	L	PS09	6498	3.3	100.3	98.3	N
82/533/304	751	R	PS10	6014	1.2	100.2		
82/533/304	752	R	PS10	6332	0.7	97.1		
82/533/304	762	R	PS10	6641	1.6	99.7	98.1	N
82/533/304	775	L	PS11	6765	3.7	99.8		
82/533/304	790	L	PS11	6965	0.8	99.7		
82/533/304	791	L	PS11	7255	3.4	99.9	99.7	N
82/533/304	763	R	PS12	6770	0.7	98.8		
82/533/304	764	R	PS12	7031	3.0	99.5		
82/533/304	765	R	PS12	7242	2.6	99.2	99.0	N
82/533/304	793	L	PS13	7464	1.4	101.6		
82/533/304	794	L	PS13	7846	2.8	97.2		
82/533/304	795	L	PS13	8097	2.5	101.1	98.7	N
82/533/304	767	R	PS14	7600	3.7	104.1		
82/533/304	768	R	PS14	7919	3.3	99.0		
82/533/304	769	R	PS14	7995	1.5	99.3	99.3	N
20/85C/807	1227	L	PS01	153632	3.2	95.3		
20/85C/807	1226	L	PS01	153828	2.1	104.1		
20/85C/807	1201	L	PS01	154170	1.7	96.4		
20/85C/807	1200	L	PS01	154296	1.7	103.1		
20/85C/807	1199	L	PS01	154646	2.1	96.1		
20/85C/807	1198	L	PS01	154910	3.6	93.4	94.9	N
20/85C/807	1229	R	PS02	153649	0.8	101.2		
20/85C/807	1228	R	PS02	153784	2.4	101.4		
20/85C/807	1205	R	PS02	154227	1.9	96.2		
20/85C/807	1204	R	PS02	154291	1.1	98.5		
20/85C/807	1203	R	PS02	154607	2.6	100.1		

MRD Job No	Test No.	Side L/R	Lot	Chain m	Offset m	RDD * %	CV ** %	Pass Y/N
20/85C/807	1202	R	PS02	154918	2.0	104.1	98.3	N
90/514/201	1453	L	PS301	16338	3.0	94.9		
90/514/201	1455	R	PS301	16518	3.2	99.0		
90/514/201	1454	L	PS301	16660	3.2	95.6		
90/514/201	1456	R	PS301	16728	3.0	98.3	95.7	N
90/514/201	1435	R	PR201	23033	2.7	99.6		
90/514/201	1424	R	PR201	23961	2.1	102.6	100.2	Y
90/514/201	1434	L	PL201	23041	2.6	104.0		
90/514/201	1433	L	PL201	23287	2.5	92.6		
90/514/201	1423	L	PL201	23982	2.3	97.1	94.8	N
90/514/201	1428	L	PL202	24043	3.0	93.3		
90/514/201	1427	L	PL202	24498	3.2	98.5		
90/514/201	1414	L	PL202	25145	2.2	98.4	95.1	N
90/514/201	1426	R	PR202	24136	2.3	98.3		
90/514/201	1425	R	PR202	24442	2.4	97.6		
90/514/201	1429	R	PR202	24971	3.0	98.2	97.8	N
90/514/201	1399	R	PR101	26178	3.0	97.9		
90/514/201	1398	R	PR101	26534	2.4	100.6		
90/514/201	1392	R	PR101	27227	2.5	97.9	98.0	N
90/514/201	1415	L	PL101	26194	2.5	99.6		
90/514/201	1397	L	PL101	26821	3.0	100.0		
90/514/201	1393	L	PL101	27324	2.5	101.0	99.8	N

* RDD – Relative Dry Density % at the test location after compaction.

** CV – Characteristic Value – a statistical combination of the RDD values for a group of RDD tests for a single lot. The lot passes if the CV is 100% or higher.

Table D.2 - Project Site Identification and Description

Identification of the separate sites selected for FWD testing for comparison with construction quality control RDD test results. Project Site ID numbers are allocated to each selected site, cross-referenced to the MRD Job Number.

Site ID No.	MRD Job No.	Road No.	Chainage km		Description
			Start	Finish	
1	90/33A/806	33A	153.000	154.500	Peak Downs Highway (Rehabilitation Project)
2	120/33B/305	33B	66.700	67.300	Peak Downs Highway (Sandy Creek to Sawn Creek)
3	107/517/301	517	2.150	3.850	Sarina-Homebush Road (Mt Convenient to West Plane Creek Road)
4	20/519/802	519	28.400	29.600	Dysart-Middlemount Road (Rehabilitation between Norwich Park Mine and Shire Boundry)
5	82/533/304	533	4.800	7.400	Marian-Eton Road (Mullers Road to Crebers Corners)
6	20/85C/807	85C	153.500	155.100	Fitzroy Development Road (Rehabilitation Project)

Figure D-1 - Lot Identification for Selected Test Sites

Showing the arrangement of lots for each selected site. Test locations within each lot are identified by the lot ID number ie *s.l.t* where *s* is the site number, *l* is the lot number for that site and *t* is the test location number within the lot, generally numbered in increasing chainage order.

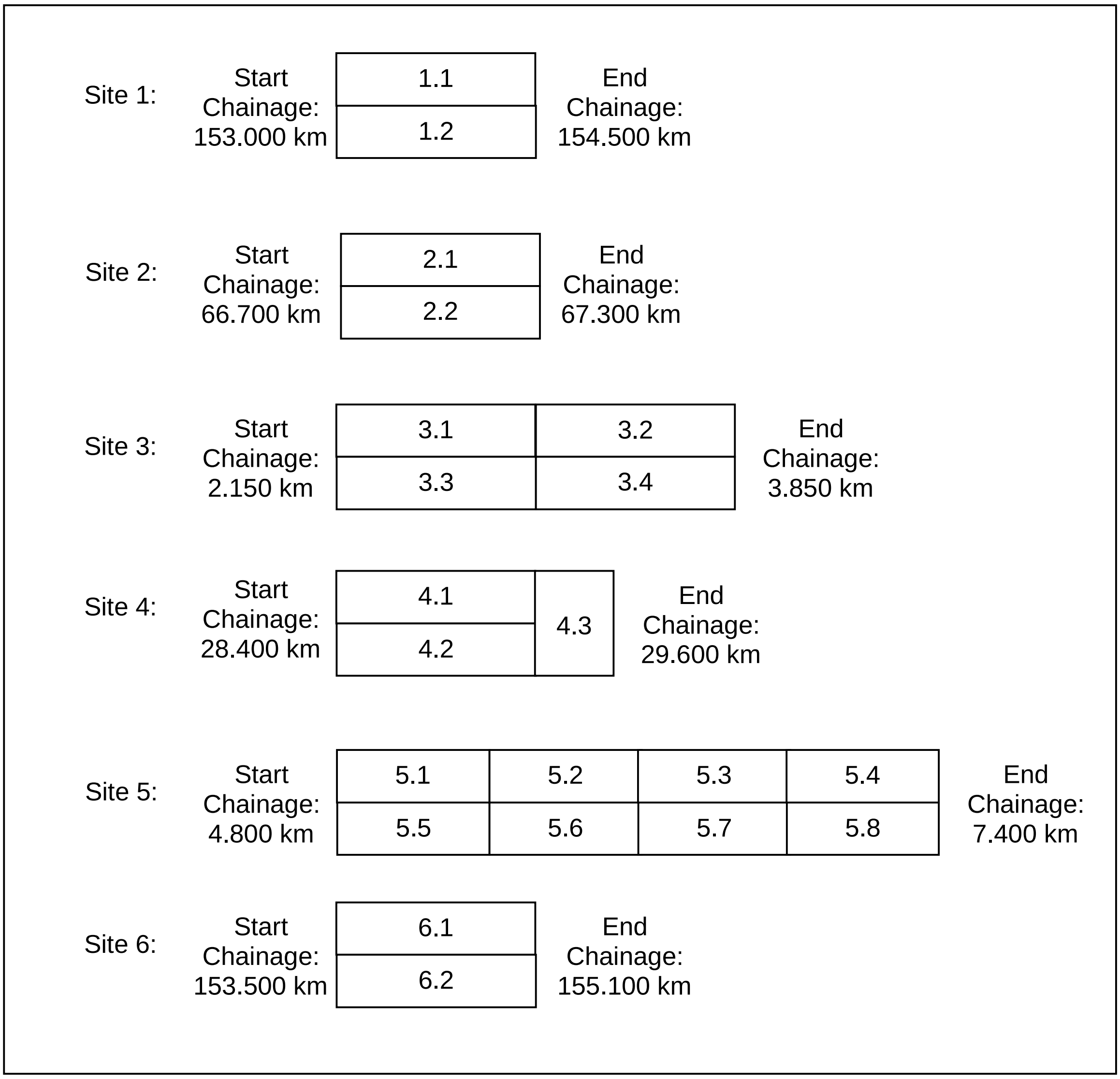


Table D.3 - Acceptance Test Results Summary

Summarising the Relative Dry Density construction quality control test results for each of the lots selected for FWD testing.

Site ID No.	Lot ID	Chainage km		Side	No of Test Sites	RDD Range %	CV * %
		Start	End				
1	1.1	153.000	154.500	L	8	100.4-93.0	94.8
1	1.2	153.000	154.500	R	9	98.2-91.6	93.7
2	2.1	66.600	67.300	L	4	102.8-100	100.5
2	2.2	66.600	67.300	R	4	101.4-93.1	96.3
3	3.1	2.150	3.000	L	4	102.7-98.0	99.0
3	3.2	3.000	3.850	L	4	100.3-93.5	96.0
3	3.3	2.150	3.000	R	4	102.1-97.5	97.6
3	3.4	3.000	3.850	R	4	100.7-93.0	95.2
4	4.1	28.400	29.150	L	3	96.6-95.2	95.7
4	4.2	28.400	29.150	R	3	98.4-94.0	95.2
4	4.3	29.150	29.600	LR	4	101.9-98.6	98.7
5	5.1	4.800	5.350	L	3	103.7-100.2	100.5
5	5.2	5.350	5.990	L	3	105.9-97.3	99.6
5	5.3	5.345	6.700	L	3	100.3-97.6	98.3
5	5.4	6.700	7.400	L	3	99.9-99.7	99.7
5	5.5	4.800	5.350	R	3	103.6-98.5	99.8
5	5.6	5.350	5.990	R	3	99.8-98.6	99.0
5	5.7	5.345	6.700	R	3	100.2-97.1	98.1
5	5.8	6.700	7.400	R	3	99.5-98.8	99.0
6	6.1	153.5	155.1	L	6	104.1-93.4	94.9
6	6.2	153.5	155.1	R	6	104.1-96.2	98.3

* CV - the Characteristic Value of the RDD test results for each lot – a statistical combination of the RDD values for a group of RDD tests for a single lot. The lot passes if the CV is 100% or higher.

Table D.4 - Acceptance Test Data – Selected Sites

Complete listing of RDD construction quality control acceptance tests for the lots selected for FWD testing as part of this investigation. The locations of these tests were used as the basis for locating the comparison FWD tests.

Test No.	Site ID No	Lot ID	Test ID	Chain m	Offset m	Side L/R	RDD %	CV %	Pass Y/N
1	1	1.1	1.1.1	153103	2.2	L	94.9		
2	1	1.1	1.1.2	153221	1.4	L	93.0		
3	1	1.1	1.1.3	153537	2.4	L	96.9		
4	1	1.1	1.1.4	153602	2.2	L	100.4		
5	1	1.1	1.1.5	153821	0.5	L	95.1		
6	1	1.1	1.1.6	153946	2.1	L	98.9		
7	1	1.1	1.1.7	154186	3.5	L	97.4		
8	1	1.1	1.1.8	154441	2.0	L	96.4	94.8	N
9	1	1.2	1.2.1	153158	3.0	R	94.4		
10	1	1.2	1.2.2	153313	0.2	R	95.3		
11	1	1.2	1.2.3	153462	1.7	R	98.2		
12	1	1.2	1.2.4	153588	3.6	R	91.6		
13	1	1.2	1.2.5	153755	0.8	R	99.0		
14	1	1.2	1.2.6	153871	3.0	R	96.6		
15	1	1.2	1.2.7	154115	4.4	R	93.2		
16	1	1.2	1.2.8	154204	1.7	R	95.2		
17	1	1.2	1.2.9	154365	1.6	R	97.1	93.7	N
18	2	2.1	2.1.1	66837	3.3	L	101.8		
19	2	2.1	2.1.2	66913	1.3	L	100.0		
20	2	2.1	2.1.3	66980	4.2	L	102.8		
21	2	2.1	2.1.4	67171	1.3	L	100.4	100.5	Y
22	2	2.2	2.2.1	66826	2.5	R	93.1		
23	2	2.2	2.2.2	67028	4.0	R	101.4		
24	2	2.2	2.2.3	67045	3.7	R	100.7		
25	2	2.2	2.2.4	67254	4.4	R	99.3	96.3	N
26	3	3.1	3.1.1	2334	1.8	L	100.8		
27	3	3.1	3.1.2	2596	0.6	L	98.0		
28	3	3.1	3.1.3	2660	3.6	L	102.7		
29	3	3.1	3.1.4	2913	0.9	L	99.5	99.0	N
30	3	3.2	3.2.1	3115	2.9	L	100.3		
31	3	3.2	3.2.2	3314	1.0	L	98.5		
32	3	3.2	3.2.3	3495	3.2	L	99.2		
33	3	3.2	3.2.4	3665	1.1	L	93.5	96.0	N
34	3	3.3	3.3.1	2296	1.5	R	102.1		
35	3	3.3	3.3.2	2424	3.3	R	97.5		
36	3	3.3	3.3.3	2723	2.5	R	98.1		
37	3	3.3	3.3.4	2904	1.8	R	97.9	97.6	N
38	3	3.4	3.4.1	3122	0.5	R	96.9		
39	3	3.4	3.4.2	3351	3.5	R	97.9		

Test No.	Site ID No	Lot ID	Test ID	Chain m	Offset m	Side L/R	RDD %	CV %	Pass Y/N
40	3	3.4	3.4.3	3544	0.9	R	100.7		
41	3	3.4	3.4.4	3718	1.5	R	93.0	95.2	N
42	4	4.1	4.1.1	28579	1.9	L	95.2		
43	4	4.1	4.1.2	28929	1.5	L	96.6		
44	4	4.1	4.1.3	29035	0.5	L	96.1	95.7	N
45	4	4.2	4.2.1	28643	0.4	R	94.0		
46	4	4.2	4.2.2	28726	2.1	R	95.5		
47	4	4.2	4.2.3	29062	1.8	R	98.4	95.2	N
48	4	4.3	4.3.1	29255	1.3	L	98.7		
49	4	4.3	4.3.2	29313	0.5	R	101.9		
50	4	4.3	4.3.3	29486	1.1	L	98.6		
51	4	4.3	4.3.4	29521	2.3	R	99.4	98.7	N
52	5	5.1	5.1.1	4981	1.1	L	103.7		
53	5	5.1	5.1.2	5060	8.0	L	100.7		
54	5	5.1	5.1.3	5321	11.4	L	100.2	100.5	Y
55	5	5.2	5.2.1	5359	2.9	L	97.3		
56	5	5.2	5.2.2	5645	1.7	L	102.6		
57	5	5.2	5.2.3	5921	1.3	L	105.9	99.6	N
58	5	5.3	5.3.1	6004	2.0	L	97.6		
59	5	5.3	5.3.2	6424	2.4	L	99.2		
60	5	5.3	5.3.3	6498	3.3	L	100.3	98.3	N
61	5	5.4	5.4.1	6765	3.7	L	99.8		
62	5	5.4	5.4.2	6965	0.8	L	99.7		
63	5	5.4	5.4.3	7255	3.4	L	99.9	99.7	N
64	5	5.5	5.5.1	5001	3.2	R	103.6		
65	5	5.5	5.5.2	5061	2.3	R	101.5		
66	5	5.5	5.5.3	5243	2.8	R	98.5	99.8	N
67	5	5.6	5.6.1	5454	2.6	R	99.8		
68	5	5.6	5.6.2	5630	1.5	R	99.7		
69	5	5.6	5.6.3	5813	3.2	R	98.6	99.0	N
70	5	5.7	5.7.1	6014	1.2	R	100.2		
71	5	5.7	5.7.2	6332	0.7	R	97.1		
72	5	5.7	5.7.3	6641	1.6	R	99.7	98.1	N
73	5	5.8	5.8.1	6770	0.7	R	98.8		
74	5	5.8	5.8.2	7031	3.0	R	99.5		
75	5	5.8	5.8.3	7242	2.6	R	99.2	99.0	N
76	6	6.1	6.1.1	153632	3.2	L	95.3		
77	6	6.1	6.1.2	153828	2.1	L	104.1		
78	6	6.1	6.1.3	154170	1.7	L	96.4		
79	6	6.1	6.1.4	154296	1.7	L	103.1		
80	6	6.1	6.1.5	154646	2.1	L	96.1		
81	6	6.1	6.1.6	154910	3.6	L	93.4	94.9	N
82	6	6.2	6.2.1	153649	0.8	R	101.2		
83	6	6.2	6.2.2	153784	2.4	R	101.4		

Test No.	Site ID No	Lot ID	Test ID	Chain m	Offset m	Side L/R	RDD %	CV %	Pass Y/N
84	6	6.2	6.2.3	154227	1.9	R	96.2		
85	6	6.2	6.2.4	154291	1.1	R	98.5		
86	6	6.2	6.2.5	154607	2.6	R	100.1		
87	6	6.2	6.2.6	154918	2.0	R	104.1	98.3	N

APPENDIX E – PROJECT DATA & INSPECTIONS

E.1 Insitu Stabilisation Design Data

Pre-design investigations of each site were carried out to determine the layer thicknesses and strengths of the existing soils. Once the depth of stabilisation and the thickness of grade-fixing gravel was decided, UCS tests with varying cement content were carried out on sample of the final mix to determine cement content to reach or exceed the target 600 – 2000 MPa.

Table E.1 shows the design thicknesses of the sub-base and top modified base layers which were assessed from the results of the pre-design investigation, the modului of the existing subgrade and sub-base, and the target modulus for the modified layer. The last column shows the cement content to be added to achieve the target modulus.

Table E.2 shows the design traffic values assessed from the best available data and the target total Standard Axle Repetitions (SAR) values expressed in Equivalent Standard Axles (ESAs). The pavement layer design attempted to achieve the SAR values.

Table E.3 shows the capability of the designed rehabilitated pavement forecast by CIRCLY and the consequential forecast design life. Even though the forecast design life generally fell short of the desired target design life, other factors dictated that the projects proceed.

E.2 Rehabilitated Pavement Visual Inspection Results

Table E.4 shows the results of the visual inspections of the road pavements carried out for this project in August 2007. The rehabilitated road pavements had been in service for periods between six and eighteen months.

Table E.1 - Pavement Design Parameters

Site ID No.	Thickness mm		Modulus MPa			Cement (Base Layer) %
	Sub-base	Base	Subgrade	Sub-base	Base	
1	200	200	40	69	1000	2%
2	200	150	50	650	1000	2.5%
3	0	300	30	-	2000	3%
4	100	200	71.4	71.0	600	2%
5	0	200	50	-	1000	3%
6	200	225	50	186	600	2%

Table E.2 - Design Traffic Assumptions

1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
Site ID No.	**AADT Initial vpd***	**Heavy Vehicles %**	**Growth % pa**	**F_1 #**	**Design Life years**	**f ****	**SAR ## ($\times 10^6$) ESAs**
1	2590	16.0	16.0	3.2	10 20	25 133	7.000 37.900
2	4121	8.3	5.3	3.2	10 20	13.4 25.9	2.700 7.200
3	878	7.5	2.4	3.2	10 20	12 28	0.113
4	750	14.0	5.0	3.2	10 20	13 25	0.658 1.780
5	618	13.1	4.0	3.2	20	29.8	1.400
6	273	11.0	5.2	3.2	10 20	18.56 71.27	1.100 4.200

* total vehicles per day at rehabilitation completion date, both ways.

\# Standard axles per heavy vehicle - Culway Data, Main Roads Mackay Memo issued 19/12/2001

** Total axles factor for design life and growth factor Equation 7.6 MRD

$$f = (1 + 0.01i)\frac{(1 + 0.01i)^y - 1}{0.01i}$$

where i = growth rate percentage
y = years

\## Total Standard Axle Repetitions for assumed design life

Table E.3 - CIRCLY Design Life Forecast

1	*2*	*3*	*4*	*5*
Site ID No.	**Forecast SAR Capability x 10^6**	**Desired Design Life yr**	**SAR for Desired Design Life x 10^6**	**CIRCLY Forecast Design Life yr**
1	1.00	10	7.0	2
2	4.50	20	7.2	12
3	0.93	20	1.1	16
4	1.05	20	1.78	12
5	3.70	20	2.50	29
6	1.90	20	4.20	9

Table E.4 - Visual Inspection Results
Post-construction Inspection Results - August 2007

Site ID No.	Chainage	Visual Description
1	154.261 – 154.278 (LHS)	Edge line longitudinal cracking with adjacent rutting
	153.871 – 153.893 (LHS)	Longitudinal cracking on shoulder line
	153.680 – 153.780 (LHS)	Bitumen stripping has occurred and patched with a asphalt pothole mix
	153.432 – 153.440 (LHS)	Longitudinal cracking
2	All	No obvious signs of deterioration or distress of the pavement. Significant signs of bleeding especially in the inner and outer wheel paths.
3	3.314 – 3.333 (LHS)	18 mm rut near the centre line Significant signs of bleeding especially in the inner and outer wheel paths.
4	All	No obvious signs of deterioration or distress of the pavement. Signs of bleeding in the inner and outer wheel paths in small sections throughout site.
5	All	No obvious signs of deterioration or distress of the pavement. Significant signs of bleeding especially in the inner and outer wheel paths.
6	153.892 – 153.933 (RHS)	Longitudinal cracking in shoulder
	154.237 – 154.281 (LHS)	Longitudinal cracking in shoulder.
	154.416 – 154.388 (LHS)	Longitudinal cracking in shoulder.
	154.640 – 154.651 (RHS)	Longitudinal cracking in shoulder.

APPENDIX F – RAW FWD DEFLECTION RESULTS

The following Table F.1 lists the Falling Weight Deflectometer deflection readings for nominal 60, 80 and 110 kPa impacts taken as close as possible to the location where the Relative Dry Density tests were conducted during construction ie Site Specific locations. These pressures are equivalent to nominal 40, 60 and 80 kN total force application on the base plate.

Table F.2 lists the Falling Weight Deflectometer deflection readings taken for each site at regular intervals (50 m or 100 m) along the outer wheel track in each lane over the full length of the site.

Table F.1 - Site Specific Deflection Results

Table F.1(a) - Site 1 - Chainage 153.300 - 154.600 – Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
1.1.2	153.221	598	0.756	0.461	0.367	0.268	0.200	0.129	0.069
		600	0.742	0.452	0.361	0.264	0.198	0.129	0.069
		599	0.737	0.448	0.358	0.263	0.195	0.127	0.069
		874	1.043	0.650	0.519	0.388	0.295	0.194	0.103
		1138	1.364	0.837	0.667	0.498	0.387	0.254	0.135
1.1.3	153.537	599	1.072	0.812	0.660	0.464	0.312	0.166	0.080
		601	1.048	0.796	0.654	0.462	0.312	0.171	0.086
		590	1.027	0.780	0.641	0.454	0.309	0.168	0.084
		860	1.380	1.066	0.881	0.635	0.439	0.245	0.120
		1126	1.737	1.335	1.107	0.805	0.567	0.318	0.154
1.1.4	153.602	601	0.661	0.501	0.420	0.295	0.208	0.116	0.059
		600	0.649	0.490	0.410	0.289	0.204	0.122	0.062
		603	0.649	0.490	0.409	0.288	0.204	0.125	0.065
		864	0.893	0.680	0.572	0.405	0.289	0.174	0.088
		1133	1.143	0.864	0.696	0.515	0.370	0.226	0.115
1.1.5	153.821	575	0.191	0.159	0.146	0.122	0.099	0.070	0.039
		567	0.189	0.156	0.143	0.120	0.097	0.068	0.041
		564	0.187	0.156	0.142	0.118	0.096	0.069	0.038
		828	0.283	0.241	0.221	0.184	0.150	0.107	0.057
		1119	0.384	0.322	0.298	0.246	0.201	0.144	0.080
1.1.6	153.946	561	0.621	0.580	0.547	0.480	0.413	0.309	0.155
		555	0.610	0.564	0.536	0.465	0.399	0.299	0.149
		559	0.611	0.564	0.534	0.465	0.399	0.298	0.149
		833	0.835	0.794	0.727	0.650	0.556	0.413	0.206
		1107	1.087	1.001	0.883	0.813	0.696	0.513	0.256

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
1.1.7	154.186	571	1.257	0.904	0.698	0.469	0.314	0.191	0.138
		573	1.192	0.877	0.684	0.469	0.322	0.202	0.138
		570	1.172	0.868	0.680	0.470	0.328	0.205	0.133
		842	1.673	1.252	0.998	0.697	0.488	0.320	0.194
		1101	2.109	1.606	1.283	0.914	0.660	0.425	0.252
1.1.8	154.441	601	0.792	0.716	0.651	0.519	0.407	0.264	0.104
		592	0.767	0.693	0.626	0.504	0.396	0.258	0.104
		589	0.758	0.683	0.618	0.498	0.390	0.255	0.105
		853	1.105	0.983	0.882	0.712	0.566	0.372	0.156
		1112	1.430	1.254	1.126	0.904	0.718	0.474	0.201

Table F.1(b) - Site 1 - Chainage 153.300 - 154.600 - Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
1.2.2	153.313	607	0.499	0.342	0.268	0.206	0.159	0.092	0.050
		611	0.490	0.336	0.268	0.197	0.150	0.102	0.062
		610	0.490	0.334	0.267	0.197	0.148	0.103	0.061
		894	0.653	0.482	0.389	0.294	0.228	0.154	0.090
		1161	0.861	0.614	0.501	0.374	0.291	0.205	0.121
1.2.3	153.462	577	0.470	0.381	0.336	0.273	0.215	0.143	0.092
		576	0.465	0.376	0.338	0.266	0.212	0.139	0.087
		573	0.462	0.373	0.334	0.263	0.209	0.139	0.087
		844	0.677	0.554	0.491	0.401	0.319	0.219	0.132
		1116	0.895	0.721	0.643	0.519	0.417	0.290	0.171
1.2.4	153.588	569	1.150	0.912	0.784	0.600	0.464	0.290	0.134
		561	1.117	0.885	0.761	0.585	0.454	0.289	0.128
		562	1.109	0.880	0.755	0.582	0.454	0.288	0.127
		835	1.524	1.224	1.045	0.823	0.649	0.419	0.189
		1104	1.889	1.513	1.297	1.031	0.820	0.534	0.245
1.2.5	153.755	578	0.237	0.214	0.203	0.164	0.132	0.092	0.046
		574	0.234	0.210	0.198	0.162	0.132	0.090	0.045
		568	0.234	0.208	0.197	0.161	0.131	0.090	0.046
		837	0.336	0.316	0.299	0.246	0.199	0.137	0.078
		1119	0.461	0.411	0.389	0.323	0.264	0.184	0.095
1.2.6	153.871	582	1.012	0.755	0.599	0.415	0.290	0.161	0.079
		574	0.952	0.718	0.575	0.403	0.280	0.160	0.079
		572	0.940	0.712	0.571	0.402	0.276	0.163	0.070
		846	1.318	1.011	0.826	0.591	0.420	0.244	0.115
		1109	1.638	1.267	1.035	0.761	0.554	0.327	0.146
1.2.7	154.115	574	0.957	0.615	0.489	0.323	0.225	0.145	0.091
		568	0.907	0.595	0.475	0.320	0.225	0.145	0.093
		567	0.898	0.592	0.472	0.320	0.225	0.146	0.094
		851	1.238	0.851	0.680	0.474	0.338	0.221	0.137
		1127	1.550	1.082	0.862	0.616	0.449	0.296	0.184

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
1.2.8	154.204	572	0.413	0.362	0.327	0.273	0.221	0.154	0.091
		566	0.406	0.357	0.321	0.268	0.213	0.153	0.091
		568	0.408	0.358	0.323	0.269	0.217	0.153	0.091
		834	0.601	0.532	0.481	0.398	0.322	0.232	0.136
		1108	0.786	0.692	0.610	0.518	0.424	0.303	0.178
1.2.9	154.365	573	0.411	0.358	0.325	0.270	0.218	0.152	0.091
		560	0.409	0.353	0.320	0.266	0.215	0.154	0.091
		557	0.408	0.352	0.319	0.265	0.214	0.155	0.090
		833	0.597	0.525	0.479	0.397	0.324	0.230	0.134
		1112	0.792	0.682	0.625	0.518	0.425	0.304	0.172

Table F.1(c) - Site 2 - Chainage 66.800 - 67.300 - Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
2.1.1	66.837	574	0.128	0.118	0.112	0.099	0.085	0.066	0.035
		568	0.126	0.117	0.111	0.098	0.084	0.064	0.034
		564	0.127	0.117	0.110	0.098	0.084	0.064	0.035
		862	0.191	0.180	0.170	0.150	0.130	0.099	0.053
		1153	0.263	0.242	0.227	0.201	0.175	0.132	0.072
2.1.2	66.913	602	0.194	0.126	0.113	0.094	0.075	0.054	0.027
		600	0.191	0.125	0.112	0.093	0.075	0.053	0.029
		601	0.191	0.126	0.113	0.093	0.076	0.053	0.031
		867	0.280	0.196	0.176	0.145	0.117	0.082	0.044
		1147	0.381	0.271	0.243	0.200	0.161	0.112	0.058
2.1.3	66.980	585	0.243	0.128	0.115	0.093	0.073	0.048	0.023
		579	0.236	0.125	0.113	0.092	0.072	0.049	0.026
		577	0.235	0.124	0.112	0.092	0.072	0.049	0.024
		863	0.348	0.201	0.181	0.145	0.114	0.078	0.040
		1150	0.460	0.280	0.252	0.201	0.159	0.107	0.055
2.1.4	67.171	608	0.150	0.137	0.132	0.120	0.112	0.045	0.029
		607	0.151	0.136	0.131	0.120	0.110	0.048	0.030
		598	0.149	0.135	0.129	0.118	0.109	0.047	0.030
		866	0.213	0.207	0.199	0.181	0.167	0.072	0.045
		1130	0.299	0.280	0.268	0.244	0.224	0.100	0.061

Table F.1(d) - Site 2 - Chainage 66.800 - 67.300 - Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
2.2.1	66.826	590	0.256	0.191	0.161	0.118	0.087	0.052	0.020
		585	0.249	0.188	0.157	0.116	0.085	0.051	0.023
		584	0.249	0.187	0.157	0.115	0.085	0.051	0.022
		858	0.357	0.276	0.234	0.175	0.130	0.079	0.030
		1141	0.464	0.362	0.306	0.232	0.175	0.106	0.042

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
2.2.2	67.028	569	0.127	0.110	0.095	0.077	0.063	0.048	0.026
		566	0.123	0.106	0.093	0.075	0.062	0.044	0.026
		564	0.123	0.105	0.092	0.075	0.062	0.045	0.027
		843	0.187	0.166	0.146	0.120	0.099	0.071	0.041
		1135	0.251	0.226	0.200	0.164	0.137	0.099	0.056
2.2.3	67.045	577	0.105	0.093	0.088	0.079	0.071	0.057	0.025
		573	0.103	0.093	0.088	0.078	0.069	0.056	0.026
		570	0.103	0.092	0.087	0.078	0.069	0.054	0.027
		841	0.157	0.145	0.136	0.122	0.109	0.087	0.040
		1137	0.211	0.196	0.186	0.165	0.147	0.118	0.056
2.2.4	67.254	574	0.131	0.116	0.106	0.091	0.077	0.058	0.031
		570	0.128	0.114	0.104	0.089	0.077	0.057	0.030
		566	0.128	0.112	0.103	0.089	0.076	0.056	0.031
		856	0.196	0.179	0.166	0.142	0.123	0.093	0.049
		1143	0.269	0.244	0.227	0.196	0.170	0.129	0.070

Table F.1(e) - Site 3 - Chainage 2.200 - 3.800 - Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
3.1.1	2.334	596	0.262	0.241	0.225	0.201	0.172	0.128	0.067
		588	0.258	0.236	0.223	0.196	0.167	0.126	0.066
		588	0.258	0.238	0.222	0.197	0.168	0.126	0.065
		855	0.419	0.385	0.358	0.317	0.271	0.201	0.104
		1121	0.595	0.542	0.504	0.440	0.374	0.277	0.139
3.1.2	2.596	606	0.519	0.395	0.322	0.238	0.172	0.114	0.058
		600	0.513	0.389	0.316	0.235	0.171	0.112	0.059
		597	0.508	0.386	0.314	0.234	0.170	0.111	0.059
		879	0.777	0.598	0.488	0.363	0.265	0.171	0.090
		1148	1.059	0.804	0.658	0.491	0.361	0.231	0.121
3.1.3	2.660	604	0.253	0.210	0.195	0.174	0.148	0.113	0.066
		613	0.256	0.212	0.197	0.177	0.149	0.114	0.065
		606	0.250	0.209	0.194	0.174	0.147	0.112	0.062
		873	0.370	0.317	0.293	0.262	0.223	0.169	0.088
		1127	0.495	0.420	0.391	0.347	0.298	0.224	0.123
3.1.4	2.913	583	0.360	0.305	0.258	0.213	0.170	0.117	0.060
		579	0.358	0.300	0.256	0.212	0.169	0.117	0.059
		581	0.361	0.301	0.257	0.213	0.170	0.118	0.061
		848	0.540	0.464	0.400	0.331	0.265	0.183	0.093
		1123	0.750	0.621	0.539	0.445	0.359	0.245	0.124
3.2.1	3.115	589	0.219	0.201	0.183	0.160	0.133	0.096	0.047
		587	0.217	0.199	0.181	0.159	0.134	0.093	0.052
		583	0.218	0.199	0.181	0.158	0.131	0.095	0.047
		851	0.335	0.311	0.284	0.248	0.208	0.145	0.075
		1122	0.462	0.427	0.371	0.338	0.282	0.197	0.098

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
3.2.2	3.314	559	0.243	0.216	0.204	0.174	0.142	0.095	0.048
		552	0.240	0.216	0.201	0.172	0.140	0.095	0.047
		551	0.240	0.216	0.201	0.172	0.140	0.095	0.047
		838	0.388	0.350	0.325	0.278	0.225	0.151	0.074
		1126	0.543	0.485	0.414	0.383	0.308	0.207	0.101
3.2.3	3.495	539	0.171	0.179	0.167	0.153	0.135	0.106	0.057
		538	0.173	0.177	0.167	0.152	0.135	0.106	0.057
		537	0.175	0.176	0.167	0.153	0.135	0.106	0.057
		824	0.270	0.280	0.265	0.242	0.215	0.168	0.090
		1106	0.366	0.384	0.364	0.329	0.294	0.230	0.122
3.2.4	3.665	565	0.530	0.442	0.390	0.304	0.232	0.143	0.074
		562	0.526	0.439	0.385	0.302	0.232	0.144	0.075
		559	0.524	0.436	0.383	0.301	0.230	0.143	0.074
		836	0.792	0.675	0.601	0.472	0.362	0.226	0.112
		1116	1.078	0.912	0.801	0.641	0.496	0.311	0.150

Table F.1(f) - Site 3 - Chainage 2.200 - 3.800 - Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
3.3.1	2.296	596	0.234	0.219	0.210	0.191	0.167	0.119	0.068
		591	0.233	0.216	0.207	0.188	0.162	0.117	0.068
		589	0.235	0.215	0.207	0.188	0.162	0.118	0.071
		868	0.368	0.344	0.328	0.295	0.253	0.181	0.106
		1133	0.517	0.474	0.452	0.398	0.343	0.242	0.139
3.3.2	2.423	591	0.255	0.235	0.221	0.192	0.160	0.116	0.060
		589	0.253	0.232	0.215	0.189	0.159	0.116	0.060
		589	0.254	0.234	0.218	0.190	0.159	0.116	0.059
		857	0.391	0.369	0.341	0.298	0.253	0.184	0.091
		1121	0.552	0.509	0.471	0.409	0.347	0.250	0.125
3.3.3	2.723	603	0.199	0.176	0.165	0.146	0.126	0.094	0.050
		597	0.197	0.173	0.163	0.144	0.125	0.091	0.049
		593	0.198	0.174	0.164	0.144	0.125	0.092	0.049
		858	0.291	0.270	0.253	0.221	0.191	0.141	0.073
		1121	0.423	0.374	0.352	0.304	0.263	0.191	0.099
3.3.4	2.904	607	0.176	0.162	0.147	0.129	0.109	0.077	0.048
		605	0.175	0.160	0.145	0.128	0.110	0.075	0.053
		605	0.177	0.160	0.144	0.128	0.110	0.075	0.052
		871	0.278	0.251	0.227	0.197	0.166	0.118	0.066
		1129	0.377	0.344	0.312	0.267	0.223	0.162	0.084
3.4.1	3.122	579	0.264	0.227	0.211	0.172	0.139	0.094	0.046
		577	0.262	0.225	0.210	0.172	0.139	0.093	0.047
		573	0.262	0.225	0.208	0.172	0.139	0.091	0.050
		846	0.416	0.365	0.336	0.275	0.221	0.147	0.073
		1125	0.589	0.505	0.466	0.378	0.302	0.200	0.099

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
3.4.2	3.351	601	0.163	0.145	0.134	0.112	0.090	0.056	0.016
		603	0.162	0.144	0.134	0.111	0.088	0.054	0.018
		600	0.161	0.144	0.133	0.110	0.088	0.055	0.015
		867	0.239	0.227	0.211	0.172	0.138	0.085	0.025
		1132	0.349	0.322	0.295	0.240	0.191	0.118	0.029
3.4.3	3.544	563	0.179	0.156	0.148	0.131	0.114	0.088	0.055
		560	0.178	0.154	0.146	0.130	0.113	0.085	0.056
		557	0.178	0.153	0.146	0.130	0.113	0.086	0.055
		826	0.264	0.239	0.228	0.202	0.176	0.135	0.086
		1117	0.368	0.326	0.308	0.273	0.239	0.184	0.114
3.4.4	3.718	607	0.333	0.296	0.270	0.228	0.183	0.128	0.070
		595	0.326	0.290	0.264	0.221	0.178	0.124	0.068
		595	0.326	0.289	0.265	0.222	0.179	0.126	0.067
		861	0.507	0.457	0.416	0.346	0.280	0.193	0.097
		1130	0.710	0.638	0.572	0.477	0.390	0.262	0.126

Table F.1(g) - Site 4 - Chainage 28.500 - 29.600 – Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
4.1.1	28.579	617	0.452	0.350	0.302	0.244	0.195	0.124	0.052
		613	0.446	0.346	0.298	0.241	0.192	0.122	0.054
		614	0.445	0.346	0.298	0.241	0.192	0.122	0.049
		899	0.651	0.511	0.436	0.353	0.280	0.180	0.071
		1172	0.893	0.681	0.578	0.466	0.370	0.238	0.096
4.1.2	28.929	615	0.314	0.263	0.229	0.168	0.120	0.063	0.031
		614	0.308	0.258	0.223	0.165	0.115	0.059	0.026
		614	0.307	0.258	0.224	0.165	0.114	0.058	0.027
		892	0.456	0.377	0.323	0.239	0.169	0.082	0.032
		1162	0.604	0.490	0.418	0.307	0.216	0.110	0.039
4.1.3	29.035	625	0.578	0.358	0.242	0.118	0.060	0.028	0.017
		619	0.557	0.348	0.234	0.117	0.061	0.029	0.016
		615	0.551	0.343	0.231	0.116	0.061	0.028	0.017
		912	0.791	0.506	0.344	0.173	0.092	0.043	0.026
		1194	1.035	0.655	0.445	0.229	0.129	0.056	0.034
4.3.1	29.255	593	0.539	0.432	0.378	0.303	0.239	0.157	0.083
		588	0.530	0.422	0.372	0.297	0.234	0.154	0.083
		589	0.531	0.423	0.371	0.297	0.234	0.153	0.085
		863	0.787	0.648	0.574	0.457	0.359	0.239	0.129
		1129	1.062	0.866	0.764	0.610	0.481	0.319	0.168
4.3.3	29.486	596	0.455	0.334	0.265	0.189	0.134	0.082	0.039
		596	0.448	0.330	0.264	0.189	0.136	0.084	0.041
		592	0.445	0.328	0.263	0.188	0.135	0.083	0.040
		873	0.683	0.508	0.406	0.292	0.211	0.128	0.060
		1150	0.922	0.686	0.550	0.397	0.288	0.174	0.079

Table F.1(h) - Site 4 - Chainage 28.500 - 29.600 – Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
4.2.1	28.643	565	1.311	0.968	0.755	0.545	0.385	0.231	0.111
		564	1.268	0.931	0.738	0.545	0.378	0.227	0.113
		563	1.254	0.924	0.733	0.535	0.384	0.231	0.112
		837	1.750	1.326	1.065	0.795	0.571	0.345	0.171
		1094	2.201	1.674	1.360	1.017	0.743	0.454	0.223
4.2.2	28.726	581	1.308	0.957	0.755	0.538	0.382	0.207	0.093
		581	1.299	0.952	0.753	0.538	0.383	0.209	0.092
		577	1.289	0.945	0.747	0.534	0.381	0.209	0.092
		854	1.805	1.359	1.091	0.796	0.577	0.317	0.136
		1110	2.259	1.720	1.395	1.029	0.752	0.418	0.176
4.2.3	29.062	627	1.113	0.707	0.499	0.269	0.148	0.062	0.034
		636	1.080	0.699	0.500	0.273	0.152	0.067	0.038
		634	1.063	0.696	0.492	0.271	0.153	0.065	0.035
		933	1.436	0.952	0.670	0.383	0.222	0.094	0.044
		1215	1.794	1.178	0.813	0.487	0.292	0.125	0.058
4.3.2	29.313	595	0.779	0.495	0.365	0.243	0.172	0.100	0.043
		593	0.743	0.481	0.355	0.239	0.170	0.099	0.050
		592	0.736	0.481	0.354	0.239	0.170	0.100	0.051
		877	1.051	0.696	0.520	0.357	0.257	0.151	0.074
		1146	1.346	0.892	0.673	0.468	0.337	0.200	0.097
4.3.4	29.521	623	0.683	0.438	0.355	0.246	0.170	0.092	0.033
		620	0.646	0.430	0.346	0.242	0.169	0.092	0.034
		616	0.642	0.430	0.342	0.240	0.168	0.092	0.038
		903	0.910	0.617	0.495	0.350	0.250	0.136	0.052
		1175	1.184	0.798	0.639	0.455	0.328	0.179	0.065

Table F.1(i) - Site 5 - Chainage 4.800 - 7.400 – Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
5.1.1	4.981	580	0.308	0.252	0.226	0.188	0.149	0.106	0.053
		572	0.305	0.247	0.219	0.183	0.145	0.103	0.053
		568	0.305	0.245	0.220	0.182	0.144	0.102	0.053
		837	0.449	0.384	0.344	0.285	0.227	0.158	0.081
		1112	0.631	0.515	0.462	0.382	0.306	0.213	0.111
5.1.2	5.060	584	0.233	0.206	0.192	0.163	0.134	0.093	0.042
		577	0.230	0.204	0.189	0.161	0.131	0.092	0.043
		574	0.228	0.203	0.188	0.159	0.131	0.091	0.042
		838	0.341	0.308	0.288	0.244	0.198	0.137	0.063
		1116	0.465	0.413	0.379	0.320	0.261	0.181	0.080
5.1.3	5.321	581	0.348	0.314	0.279	0.229	0.177	0.110	0.041
		583	0.346	0.312	0.278	0.227	0.176	0.108	0.042
		579	0.346	0.312	0.278	0.226	0.176	0.108	0.043
		853	0.518	0.479	0.428	0.345	0.268	0.164	0.063
		1123	0.710	0.635	0.566	0.454	0.353	0.217	0.085

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
5.2.1	5.359	590	0.181	0.164	0.151	0.130	0.111	0.082	0.041
		588	0.182	0.163	0.151	0.130	0.110	0.084	0.040
		585	0.181	0.162	0.150	0.129	0.109	0.082	0.041
		855	0.266	0.244	0.227	0.197	0.166	0.125	0.063
		1128	0.365	0.327	0.305	0.263	0.223	0.163	0.086
5.2.2	5.645	592	0.236	0.217	0.205	0.179	0.149	0.108	0.054
		588	0.235	0.215	0.203	0.177	0.148	0.107	0.054
		587	0.236	0.215	0.203	0.179	0.149	0.109	0.054
		860	0.358	0.334	0.313	0.272	0.228	0.167	0.082
		1127	0.495	0.454	0.421	0.363	0.309	0.223	0.107
5.2.3	5.921	602	0.400	0.326	0.282	0.220	0.165	0.097	0.047
		599	0.396	0.320	0.279	0.218	0.163	0.095	0.049
		598	0.396	0.322	0.277	0.217	0.162	0.097	0.046
		876	0.588	0.477	0.411	0.320	0.240	0.142	0.065
		1145	0.773	0.619	0.531	0.412	0.309	0.184	0.083
5.3.1	6.004	588	0.282	0.268	0.246	0.206	0.170	0.123	0.069
		581	0.278	0.263	0.240	0.203	0.166	0.118	0.062
		585	0.278	0.264	0.242	0.204	0.167	0.119	0.063
		850	0.428	0.403	0.368	0.308	0.252	0.177	0.093
		1111	0.575	0.532	0.482	0.404	0.330	0.234	0.123
5.3.2	6.424	588	0.353	0.298	0.267	0.216	0.173	0.110	0.046
		586	0.353	0.295	0.265	0.214	0.172	0.108	0.048
		584	0.355	0.295	0.263	0.214	0.170	0.109	0.047
		859	0.519	0.434	0.386	0.311	0.246	0.163	0.066
		1135	0.710	0.572	0.506	0.406	0.320	0.212	0.088
5.3.3	6.498	595	0.188	0.172	0.159	0.135	0.115	0.079	0.035
		590	0.185	0.169	0.158	0.134	0.109	0.076	0.038
		588	0.184	0.168	0.154	0.133	0.108	0.078	0.037
		856	0.271	0.252	0.232	0.196	0.165	0.115	0.053
		1121	0.363	0.336	0.309	0.259	0.217	0.151	0.070
5.4.1	6.765	599	0.250	0.226	0.211	0.179	0.147	0.099	0.047
		601	0.252	0.227	0.212	0.179	0.148	0.099	0.047
		603	0.253	0.228	0.212	0.179	0.147	0.098	0.048
		880	0.383	0.349	0.323	0.273	0.224	0.151	0.071
		1151	0.528	0.478	0.440	0.369	0.303	0.203	0.093
5.4.2	6.965	584	0.266	0.245	0.224	0.198	0.168	0.117	0.058
		581	0.264	0.243	0.225	0.197	0.165	0.116	0.058
		580	0.263	0.243	0.223	0.196	0.165	0.116	0.058
		846	0.402	0.373	0.347	0.299	0.247	0.174	0.084
		1116	0.534	0.491	0.456	0.390	0.324	0.228	0.110
5.4.3	7.255	591	0.145	0.138	0.132	0.112	0.097	0.070	0.034
		587	0.146	0.136	0.128	0.110	0.095	0.070	0.034
		590	0.146	0.137	0.130	0.111	0.095	0.071	0.034
		857	0.220	0.216	0.206	0.176	0.152	0.113	0.054
		1123	0.300	0.298	0.283	0.244	0.209	0.156	0.074

Table F.1(j) - Site 5 - Chainage 4.800 - 7.400 – Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
5.5.1	5.001	578	0.163	0.147	0.143	0.136	0.123	0.107	0.069
		582	0.160	0.148	0.143	0.135	0.122	0.106	0.068
		583	0.161	0.144	0.140	0.134	0.125	0.105	0.069
		845	0.238	0.231	0.223	0.208	0.190	0.163	0.106
		1103	0.331	0.309	0.299	0.279	0.256	0.218	0.139
5.5.2	5.061	590	0.383	0.325	0.296	0.238	0.190	0.126	0.053
		582	0.377	0.322	0.285	0.233	0.186	0.124	0.056
		581	0.375	0.320	0.285	0.233	0.186	0.124	0.058
		860	0.552	0.472	0.421	0.343	0.274	0.181	0.083
		1127	0.725	0.611	0.542	0.441	0.352	0.234	0.106
5.5.3	5.243	578	0.162	0.154	0.146	0.129	0.110	0.083	0.045
		573	0.161	0.154	0.144	0.129	0.111	0.083	0.042
		575	0.163	0.153	0.145	0.129	0.110	0.083	0.044
		834	0.252	0.235	0.224	0.198	0.171	0.130	0.065
		1102	0.329	0.315	0.299	0.263	0.230	0.174	0.084
5.6.1	5.454	594	0.248	0.226	0.209	0.183	0.152	0.111	0.052
		588	0.247	0.223	0.207	0.181	0.150	0.110	0.053
		586	0.246	0.222	0.207	0.181	0.150	0.110	0.052
		861	0.374	0.343	0.320	0.279	0.234	0.169	0.080
		1125	0.515	0.464	0.430	0.374	0.315	0.225	0.105
5.6.2	5.630	607	0.533	0.376	0.298	0.209	0.141	0.076	0.030
		603	0.522	0.368	0.293	0.206	0.140	0.076	0.031
		602	0.520	0.366	0.292	0.206	0.140	0.076	0.031
		880	0.762	0.556	0.445	0.317	0.217	0.116	0.047
		1154	1.022	0.740	0.593	0.425	0.294	0.156	0.064
5.6.3	5.813	600	0.135	0.148	0.136	0.114	0.089	0.065	0.044
		598	0.140	0.148	0.134	0.113	0.093	0.067	0.044
		599	0.144	0.147	0.135	0.113	0.090	0.064	0.045
		865	0.218	0.220	0.202	0.168	0.154	0.101	0.067
		1127	0.290	0.294	0.271	0.223	0.185	0.136	0.089
5.7.1	6.014	570	0.238	0.185	0.154	0.112	0.080	0.045	0.023
		565	0.236	0.184	0.152	0.112	0.079	0.045	0.023
		564	0.234	0.182	0.153	0.111	0.079	0.045	0.023
		839	0.357	0.279	0.232	0.170	0.122	0.069	0.034
		1121	0.488	0.375	0.309	0.229	0.165	0.095	0.047
5.7.2	6.332	598	0.136	0.122	0.112	0.094	0.077	0.049	0.024
		596	0.134	0.120	0.110	0.093	0.075	0.049	0.024
		595	0.134	0.119	0.110	0.092	0.074	0.049	0.025
		865	0.205	0.181	0.167	0.140	0.112	0.075	0.034
		1131	0.274	0.243	0.222	0.186	0.151	0.098	0.047
5.7.3	6.641	603	0.303	0.260	0.241	0.197	0.158	0.099	0.044
		604	0.302	0.257	0.236	0.195	0.156	0.102	0.044
		606	0.302	0.257	0.237	0.195	0.157	0.104	0.045
		878	0.457	0.395	0.362	0.295	0.235	0.154	0.063
		1149	0.649	0.542	0.502	0.399	0.316	0.205	0.083

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
5.8.1	6.770	609	0.240	0.210	0.179	0.130	0.092	0.052	0.026
		603	0.239	0.209	0.178	0.130	0.092	0.052	0.024
		600	0.238	0.208	0.177	0.129	0.091	0.051	0.026
		887	0.368	0.325	0.275	0.199	0.141	0.079	0.037
		1166	0.504	0.443	0.375	0.270	0.190	0.106	0.047
5.8.2	7.031	596	0.401	0.301	0.246	0.171	0.110	0.047	0.014
		598	0.398	0.299	0.246	0.171	0.111	0.049	0.015
		601	0.398	0.302	0.248	0.172	0.111	0.047	0.013
		870	0.575	0.458	0.377	0.267	0.173	0.074	0.017
		1147	0.803	0.623	0.516	0.368	0.241	0.102	0.017
5.8.3	7.242	590	0.517	0.372	0.298	0.205	0.147	0.083	0.033
		585	0.510	0.368	0.295	0.203	0.146	0.084	0.032
		585	0.510	0.366	0.294	0.203	0.146	0.080	0.035
		859	0.736	0.540	0.440	0.307	0.224	0.131	0.050
		1131	0.949	0.696	0.573	0.402	0.296	0.174	0.071

Table F.1(k) - Site 6 - Chainage 153.600 - 155.100 – Left Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
6.1.1	153.632	603	0.900	0.700	0.598	0.465	0.346	0.201	0.090
		598	0.876	0.682	0.586	0.454	0.348	0.201	0.090
		601	0.873	0.678	0.584	0.457	0.348	0.200	0.093
		882	1.214	0.964	0.839	0.667	0.520	0.307	0.136
		1151	1.542	1.230	1.055	0.867	0.684	0.411	0.183
6.1.2	153.828	591	0.711	0.508	0.426	0.305	0.229	0.151	0.082
		587	0.691	0.496	0.416	0.301	0.226	0.149	0.082
		583	0.687	0.493	0.413	0.299	0.226	0.151	0.086
		864	1.006	0.750	0.626	0.464	0.349	0.231	0.126
		1137	1.326	0.991	0.826	0.619	0.468	0.308	0.164
6.1.3	154.170	617	0.394	0.332	0.302	0.250	0.200	0.137	0.075
		615	0.391	0.329	0.299	0.247	0.199	0.138	0.072
		614	0.389	0.328	0.298	0.247	0.197	0.135	0.078
		893	0.570	0.500	0.452	0.372	0.301	0.205	0.104
		1162	0.801	0.673	0.608	0.498	0.400	0.272	0.140
6.1.4	154.296	607	0.752	0.583	0.486	0.346	0.254	0.160	0.078
		607	0.732	0.570	0.477	0.339	0.249	0.160	0.079
		608	0.730	0.568	0.474	0.338	0.250	0.160	0.078
		889	1.067	0.837	0.699	0.510	0.385	0.242	0.117
		1155	1.411	1.100	0.917	0.672	0.511	0.322	0.153
6.1.5	154.646	585	0.779	0.476	0.349	0.243	0.181	0.123	0.067
		589	0.779	0.476	0.351	0.243	0.181	0.122	0.067
		585	0.779	0.477	0.351	0.244	0.181	0.123	0.068
		876	1.102	0.700	0.527	0.374	0.281	0.189	0.099
		1153	1.416	0.900	0.694	0.500	0.380	0.251	0.129

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
6.1.6	154.910	593	1.247	0.782	0.563	0.368	0.271	0.175	0.100
		592	1.187	0.768	0.563	0.376	0.275	0.181	0.103
		590	1.168	0.768	0.565	0.379	0.276	0.182	0.107
		873	1.667	1.133	0.848	0.581	0.427	0.275	0.157
		1140	2.143	1.496	1.114	0.776	0.575	0.369	0.207

Table F.1(l) - Site 6 - Chainage 153.600 - 155.100 – Right Side

ID	Chain (km)	Press (kPa)	Deflections (mm)						
			0	200	300	450	600	900	1500
6.2.1	153.649	608	0.594	0.462	0.373	0.279	0.213	0.139	0.071
		603	0.578	0.451	0.366	0.274	0.213	0.138	0.070
		605	0.576	0.451	0.365	0.275	0.213	0.139	0.072
		888	0.824	0.658	0.544	0.414	0.325	0.211	0.108
		1160	1.080	0.854	0.709	0.545	0.430	0.281	0.141
6.2.2	153.784	601	0.731	0.494	0.388	0.289	0.217	0.148	0.086
		601	0.711	0.487	0.383	0.287	0.216	0.148	0.085
		599	0.704	0.482	0.380	0.285	0.215	0.147	0.085
		883	0.993	0.715	0.570	0.433	0.330	0.227	0.126
		1154	1.309	0.935	0.752	0.574	0.441	0.303	0.165
6.2.3	154.227	604	0.432	0.348	0.307	0.248	0.190	0.133	0.073
		606	0.430	0.346	0.305	0.248	0.190	0.132	0.076
		604	0.429	0.344	0.304	0.246	0.189	0.132	0.073
		882	0.642	0.521	0.458	0.373	0.287	0.199	0.109
		1152	0.879	0.697	0.610	0.494	0.382	0.262	0.148
6.2.4	154.291	588	0.380	0.316	0.288	0.239	0.187	0.117	0.064
		583	0.376	0.313	0.285	0.235	0.185	0.118	0.066
		581	0.377	0.312	0.286	0.237	0.182	0.117	0.066
		861	0.572	0.484	0.439	0.362	0.289	0.184	0.099
		1141	0.774	0.650	0.587	0.483	0.386	0.248	0.131
6.2.5	154.607	611	0.640	0.501	0.417	0.311	0.230	0.144	0.089
		607	0.632	0.493	0.410	0.307	0.228	0.143	0.086
		606	0.630	0.491	0.409	0.305	0.227	0.144	0.083
		890	0.927	0.728	0.608	0.455	0.339	0.218	0.123
		1159	1.240	0.955	0.800	0.596	0.446	0.289	0.161
6.2.6	154.918	605	0.802	0.627	0.522	0.396	0.296	0.189	0.097
		605	0.781	0.606	0.511	0.394	0.298	0.189	0.097
		602	0.773	0.602	0.505	0.388	0.293	0.189	0.103
		880	1.121	0.876	0.738	0.575	0.437	0.281	0.149
		1148	1.479	1.141	0.960	0.749	0.574	0.370	0.205

Table F.2 - Sequential FWD Deflection Results

Table F.2(a) - Site 1 – Chainage 153.300 – 154.600 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
153.300	609	0.570	0.460	0.390	0.289	0.211	0.133	0.081
	603	0.554	0.449	0.378	0.280	0.207	0.134	0.076
	595	0.547	0.441	0.372	0.277	0.204	0.132	0.076
	860	0.752	0.615	0.525	0.397	0.299	0.197	0.111
	1135	0.967	0.781	0.668	0.508	0.388	0.258	0.144
153.400	580	1.154	0.953	0.792	0.565	0.387	0.215	0.110
	573	1.123	0.927	0.769	0.557	0.385	0.219	0.111
	575	1.115	0.921	0.762	0.555	0.387	0.220	0.115
	849	1.532	1.290	1.022	0.802	0.575	0.329	0.168
	1118	1.934	1.620	1.261	1.028	0.752	0.440	0.232
153.500	590	1.267	0.940	0.744	0.503	0.333	0.203	0.102
	593	1.224	0.912	0.726	0.497	0.339	0.208	0.114
	594	1.212	0.904	0.722	0.496	0.339	0.209	0.116
	866	1.640	1.246	1.006	0.708	0.500	0.310	0.168
	1125	2.007	1.539	1.243	0.895	0.646	0.406	0.220
153.600	587	1.018	0.785	0.651	0.474	0.334	0.202	0.101
	581	0.985	0.754	0.633	0.465	0.327	0.200	0.103
	579	0.978	0.755	0.629	0.464	0.327	0.202	0.101
	853	1.278	0.999	0.838	0.630	0.455	0.291	0.151
	1129	1.562	1.218	1.029	0.781	0.574	0.378	0.204
153.701	575	0.381	0.340	0.316	0.265	0.214	0.146	0.076
	569	0.376	0.336	0.312	0.262	0.211	0.144	0.075
	569	0.374	0.335	0.311	0.260	0.210	0.143	0.074
	848	0.549	0.491	0.455	0.380	0.310	0.212	0.111
	1130	0.715	0.636	0.588	0.490	0.402	0.276	0.145
153.800	568	0.347	0.286	0.259	0.213	0.169	0.109	0.048
	559	0.342	0.280	0.253	0.208	0.164	0.107	0.047
	559	0.341	0.279	0.253	0.208	0.164	0.106	0.048
	841	0.503	0.425	0.384	0.316	0.254	0.166	0.071
	1125	0.665	0.559	0.507	0.416	0.335	0.219	0.097
153.900	571	0.449	0.395	0.366	0.306	0.248	0.167	0.079
	569	0.445	0.392	0.361	0.303	0.246	0.165	0.078
	566	0.442	0.391	0.360	0.302	0.243	0.164	0.076
	850	0.643	0.571	0.530	0.443	0.362	0.247	0.115
	1131	0.832	0.740	0.686	0.574	0.472	0.323	0.154
154.000	587	0.388	0.332	0.311	0.264	0.221	0.162	0.080
	583	0.383	0.330	0.307	0.259	0.215	0.154	0.076
	580	0.379	0.328	0.305	0.258	0.214	0.155	0.077
	851	0.532	0.466	0.431	0.365	0.304	0.219	0.108
	1127	0.684	0.590	0.546	0.462	0.386	0.280	0.142
154.100	593	0.809	0.656	0.570	0.446	0.349	0.220	0.114
	591	0.800	0.652	0.569	0.450	0.349	0.230	0.118
	593	0.789	0.640	0.557	0.437	0.341	0.219	0.110
	855	1.105	0.912	0.763	0.642	0.510	0.335	0.180
	1117	1.386	1.142	0.925	0.810	0.646	0.434	0.218
154.200	594	0.999	0.761	0.632	0.463	0.323	0.195	0.114
	585	0.954	0.731	0.610	0.450	0.319	0.194	0.119
	587	0.955	0.730	0.610	0.448	0.318	0.196	0.119
	853	1.272	1.022	0.845	0.643	0.470	0.293	0.170
	1111	1.998	1.281	1.050	0.817	0.613	0.388	0.220
154.300	601	0.983	0.743	0.598	0.418	0.304	0.201	0.117
	602	0.971	0.737	0.592	0.419	0.308	0.203	0.119
	597	0.962	0.732	0.588	0.414	0.302	0.197	0.117
	870	1.327	1.022	0.829	0.598	0.449	0.299	0.175
	1127	1.651	1.275	1.045	0.766	0.578	0.389	0.223

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
154.400	607	1.059	0.862	0.677	0.472	0.342	0.229	0.121
	607	1.031	0.834	0.672	0.475	0.345	0.232	0.130
	606	1.024	0.834	0.656	0.474	0.346	0.231	0.128
	878	1.406	1.159	0.917	0.677	0.505	0.342	0.189
	1142	1.778	1.454	1.158	0.862	0.659	0.449	0.246
154.500	586	0.806	0.638	0.545	0.409	0.299	0.176	0.082
	581	0.790	0.620	0.531	0.398	0.295	0.172	0.087
	582	0.790	0.625	0.535	0.403	0.293	0.175	0.084
	852	1.083	0.882	0.758	0.580	0.433	0.261	0.116
	1116	1.376	1.103	0.925	0.739	0.561	0.344	0.157
154.510	581	0.888	0.689	0.595	0.453	0.326	0.187	0.081
	581	0.865	0.669	0.583	0.443	0.321	0.182	0.078
	580	0.857	0.663	0.579	0.440	0.319	0.182	0.078
	854	1.193	0.944	0.825	0.640	0.470	0.272	0.111
	1117	1.514	1.203	1.055	0.827	0.615	0.366	0.151
154.520	571	1.032	0.840	0.643	0.478	0.314	0.160	0.078
	572	1.008	0.822	0.641	0.474	0.309	0.165	0.080
	572	1.001	0.818	0.640	0.473	0.307	0.167	0.080
	848	1.383	1.148	0.894	0.688	0.453	0.252	0.118
	1115	1.748	1.447	1.160	0.885	0.590	0.337	0.156
154.530	577	0.926	0.717	0.619	0.462	0.331	0.164	0.078
	578	0.906	0.711	0.609	0.459	0.328	0.166	0.083
	580	0.904	0.712	0.603	0.458	0.326	0.169	0.081
	852	1.241	1.013	0.859	0.665	0.485	0.261	0.121
	1112	1.573	1.276	1.091	0.854	0.633	0.350	0.165
154.540	577	0.967	0.786	0.684	0.514	0.360	0.182	0.081
	576	0.943	0.769	0.673	0.512	0.361	0.187	0.086
	574	0.932	0.766	0.671	0.507	0.362	0.187	0.084
	846	1.304	1.084	0.955	0.736	0.537	0.287	0.126
	1103	1.636	1.368	1.199	0.941	0.696	0.386	0.169
154.550	577	1.118	0.914	0.721	0.507	0.328	0.172	0.096
	573	1.075	0.886	0.703	0.498	0.331	0.179	0.103
	573	1.066	0.880	0.704	0.500	0.333	0.180	0.097
	843	1.480	1.230	1.005	0.724	0.496	0.273	0.143
	1096	1.863	1.531	1.292	0.941	0.646	0.369	0.186
154.600	590	1.120	0.765	0.584	0.378	0.254	0.141	0.075
	585	1.075	0.741	0.569	0.373	0.252	0.141	0.074
	588	1.068	0.737	0.570	0.373	0.255	0.145	0.070
	872	1.452	1.031	0.808	0.547	0.377	0.217	0.113
	1134	1.807	1.277	1.012	0.699	0.491	0.286	0.148

Table F.2(b) - Site 1 - Chainage 153.300 - 154.600 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
153.300	578	0.595	0.459	0.389	0.287	0.201	0.125	0.077
	576	0.584	0.452	0.381	0.284	0.203	0.128	0.069
	578	0.582	0.450	0.384	0.285	0.199	0.124	0.075
	846	0.813	0.655	0.555	0.424	0.312	0.198	0.106
	1121	1.031	0.821	0.713	0.547	0.402	0.256	0.144
153.400	566	0.878	0.670	0.564	0.430	0.319	0.201	0.105
	561	0.866	0.648	0.554	0.418	0.319	0.198	0.108
	562	0.859	0.647	0.551	0.418	0.316	0.200	0.105
	831	1.134	0.917	0.785	0.604	0.464	0.298	0.162
	1090	1.448	1.156	0.984	0.771	0.595	0.390	0.214
153.500	567	0.928	0.763	0.654	0.497	0.369	0.220	0.098
	564	0.904	0.742	0.638	0.488	0.363	0.219	0.101
	564	0.897	0.738	0.635	0.486	0.364	0.218	0.098

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	834	1.195	1.003	0.870	0.677	0.518	0.319	0.146
	1106	1.472	1.236	1.077	0.847	0.656	0.416	0.196
153.599	568	0.693	0.576	0.507	0.399	0.294	0.180	0.082
	564	0.675	0.564	0.496	0.391	0.287	0.177	0.083
	564	0.673	0.560	0.495	0.389	0.287	0.176	0.082
	840	0.948	0.807	0.705	0.567	0.428	0.269	0.125
	1112	1.203	1.017	0.900	0.722	0.548	0.350	0.166
153.700	588	0.742	0.604	0.531	0.420	0.319	0.192	0.092
	582	0.727	0.592	0.520	0.412	0.312	0.190	0.091
	578	0.723	0.589	0.516	0.411	0.309	0.188	0.095
	852	0.993	0.818	0.706	0.579	0.449	0.278	0.131
	1118	1.228	1.016	0.871	0.725	0.568	0.359	0.179
153.800	586	0.373	0.309	0.270	0.210	0.164	0.106	0.051
	581	0.367	0.304	0.266	0.208	0.163	0.105	0.052
	576	0.364	0.301	0.263	0.205	0.160	0.104	0.052
	849	0.533	0.449	0.397	0.310	0.242	0.156	0.077
	1125	0.708	0.582	0.510	0.401	0.317	0.207	0.103
153.900	579	0.851	0.620	0.505	0.336	0.223	0.143	0.072
	573	0.825	0.603	0.493	0.328	0.223	0.137	0.074
	570	0.813	0.596	0.488	0.323	0.214	0.136	0.075
	842	1.113	0.846	0.699	0.486	0.340	0.209	0.110
	1105	1.403	1.063	0.886	0.629	0.449	0.281	0.144
153.999	562	1.141	0.823	0.629	0.405	0.264	0.158	0.087
	561	1.091	0.796	0.620	0.405	0.267	0.158	0.089
	560	1.079	0.791	0.621	0.405	0.268	0.159	0.089
	832	1.469	1.115	0.894	0.598	0.405	0.241	0.132
	1099	1.831	1.394	1.124	0.775	0.535	0.323	0.174
154.100	572	0.946	0.556	0.418	0.272	0.199	0.140	0.087
	571	0.890	0.542	0.412	0.275	0.204	0.142	0.090
	570	0.877	0.544	0.410	0.276	0.205	0.143	0.091
	851	1.205	0.775	0.588	0.405	0.304	0.214	0.132
	1128	1.508	0.983	0.756	0.529	0.404	0.288	0.178
154.200	582	0.996	0.662	0.514	0.336	0.235	0.157	0.094
	577	0.948	0.640	0.505	0.333	0.236	0.158	0.096
	577	0.934	0.635	0.500	0.333	0.236	0.158	0.095
	854	1.285	0.889	0.702	0.483	0.351	0.234	0.139
	1121	1.594	1.113	0.886	0.621	0.459	0.310	0.185
154.300	584	0.466	0.444	0.423	0.380	0.332	0.257	0.141
	583	0.462	0.441	0.418	0.376	0.328	0.251	0.154
	582	0.460	0.436	0.419	0.374	0.328	0.252	0.155
	845	0.680	0.644	0.612	0.547	0.479	0.373	0.202
	1107	0.881	0.833	0.788	0.699	0.611	0.473	0.255
154.400	557	0.868	0.720	0.638	0.506	0.388	0.237	0.114
	556	0.850	0.706	0.630	0.499	0.384	0.235	0.115
	556	0.850	0.704	0.629	0.498	0.385	0.237	0.113
	824	1.146	0.978	0.877	0.704	0.553	0.351	0.168
	1095	1.437	1.223	1.042	0.885	0.709	0.453	0.220
154.500	577	0.700	0.547	0.467	0.341	0.237	0.136	0.060
	580	0.689	0.541	0.461	0.339	0.237	0.136	0.063
	578	0.684	0.539	0.460	0.339	0.237	0.136	0.058
	849	0.931	0.760	0.658	0.491	0.355	0.206	0.088
	1115	1.167	0.952	0.850	0.625	0.460	0.271	0.121
154.600	586	0.592	0.501	0.447	0.346	0.264	0.161	0.070
	582	0.588	0.497	0.444	0.343	0.262	0.161	0.073
	580	0.586	0.496	0.443	0.342	0.262	0.160	0.073
	856	0.812	0.705	0.630	0.495	0.384	0.240	0.112
	1123	1.022	0.881	0.772	0.625	0.489	0.312	0.147

Table F.2(c) - Site 2 – Chainage 66.800 – 67.300 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
66.800	656	0.159	0.129	0.115	0.096	0.081	0.061	0.030
	652	0.158	0.129	0.113	0.094	0.080	0.060	0.029
	644	0.157	0.127	0.113	0.094	0.080	0.060	0.028
	929	0.230	0.189	0.171	0.142	0.121	0.092	0.044
	1203	0.305	0.250	0.229	0.190	0.162	0.123	0.057
66.850	622	0.261	0.250	0.247	0.229	0.213	0.184	0.055
	626	0.261	0.251	0.248	0.229	0.214	0.183	0.056
	622	0.260	0.249	0.245	0.227	0.212	0.181	0.056
	907	0.355	0.343	0.337	0.312	0.290	0.247	0.086
	1192	0.449	0.430	0.420	0.387	0.358	0.305	0.116
66.900	637	0.156	0.125	0.117	0.098	0.081	0.059	0.034
	632	0.154	0.124	0.115	0.097	0.081	0.059	0.036
	632	0.154	0.123	0.115	0.097	0.080	0.059	0.035
	913	0.224	0.182	0.170	0.144	0.120	0.087	0.052
	1192	0.297	0.241	0.224	0.191	0.160	0.117	0.069
66.950	640	0.199	0.164	0.146	0.104	0.078	0.058	0.035
	639	0.196	0.162	0.145	0.103	0.077	0.057	0.035
	636	0.194	0.161	0.143	0.102	0.077	0.058	0.035
	913	0.278	0.236	0.213	0.154	0.117	0.086	0.051
	1190	0.360	0.312	0.282	0.205	0.160	0.116	0.067
67.000	612	0.136	0.121	0.115	0.102	0.089	0.071	0.045
	614	0.137	0.121	0.115	0.102	0.089	0.072	0.045
	616	0.138	0.122	0.115	0.103	0.090	0.071	0.045
	895	0.208	0.186	0.175	0.156	0.137	0.107	0.068
	1181	0.278	0.248	0.235	0.208	0.182	0.143	0.089
67.050	591	0.190	0.153	0.139	0.119	0.100	0.073	0.044
	595	0.188	0.153	0.139	0.119	0.100	0.074	0.045
	590	0.187	0.150	0.137	0.118	0.098	0.072	0.047
	882	0.273	0.226	0.206	0.177	0.150	0.111	0.069
	1173	0.360	0.298	0.272	0.233	0.198	0.148	0.090
67.101	614	0.171	0.162	0.163	0.152	0.143	0.092	0.040
	612	0.172	0.162	0.162	0.150	0.143	0.091	0.038
	610	0.172	0.161	0.162	0.149	0.141	0.092	0.041
	885	0.260	0.249	0.246	0.229	0.216	0.143	0.060
	1166	0.351	0.331	0.330	0.301	0.289	0.188	0.079
67.150	623	0.143	0.126	0.118	0.105	0.088	0.071	0.040
	621	0.142	0.126	0.118	0.104	0.088	0.071	0.040
	621	0.142	0.127	0.119	0.104	0.090	0.072	0.041
	903	0.216	0.195	0.183	0.161	0.140	0.110	0.063
	1172	0.293	0.261	0.246	0.217	0.188	0.147	0.083
67.200	622	0.141	0.127	0.111	0.098	0.082	0.069	0.042
	619	0.135	0.115	0.108	0.091	0.077	0.060	0.036
	620	0.139	0.122	0.111	0.093	0.078	0.065	0.042
	895	0.205	0.179	0.166	0.145	0.124	0.093	0.053
	1169	0.280	0.245	0.228	0.199	0.171	0.129	0.073
67.250	626	0.102	0.087	0.081	0.070	0.059	0.034	0.016
	623	0.102	0.088	0.080	0.069	0.059	0.035	0.017
	623	0.102	0.086	0.080	0.067	0.059	0.033	0.016
	904	0.155	0.134	0.122	0.107	0.089	0.054	0.027
	1176	0.209	0.182	0.168	0.144	0.122	0.074	0.035
67.300	626	0.208	0.167	0.150	0.120	0.096	0.061	0.027
	624	0.207	0.166	0.150	0.119	0.096	0.061	0.027
	626	0.206	0.166	0.149	0.120	0.095	0.061	0.028
	910	0.292	0.241	0.217	0.175	0.140	0.092	0.041
	1186	0.382	0.313	0.283	0.229	0.184	0.121	0.054

Table F.2(d) - Site 2 - Chainage 66.800 - 67.300 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
66.800	608	0.164	0.127	0.077	0.056	0.044	0.033	0.014
	601	0.156	0.124	0.074	0.054	0.045	0.030	0.018
	602	0.155	0.124	0.073	0.054	0.044	0.031	0.017
	874	0.221	0.181	0.119	0.088	0.071	0.048	0.026
	1159	0.297	0.235	0.161	0.120	0.098	0.066	0.035
66.850	600	0.133	0.114	0.105	0.086	0.074	0.052	0.027
	596	0.130	0.110	0.102	0.085	0.071	0.051	0.028
	593	0.129	0.109	0.101	0.083	0.071	0.051	0.026
	868	0.198	0.175	0.159	0.134	0.113	0.082	0.042
	1156	0.273	0.239	0.219	0.185	0.158	0.115	0.059
66.900	624	0.237	0.194	0.174	0.144	0.118	0.080	0.040
	618	0.232	0.190	0.170	0.141	0.116	0.078	0.042
	620	0.232	0.190	0.169	0.140	0.116	0.078	0.042
	898	0.340	0.289	0.258	0.215	0.180	0.120	0.063
	1173	0.469	0.388	0.347	0.291	0.244	0.165	0.082
66.950	572	0.212	0.175	0.156	0.128	0.103	0.070	0.031
	568	0.207	0.172	0.153	0.125	0.100	0.068	0.031
	568	0.206	0.171	0.152	0.124	0.100	0.068	0.031
	863	0.312	0.264	0.236	0.194	0.159	0.108	0.052
	1158	0.409	0.354	0.317	0.261	0.217	0.149	0.070
67.000	575	0.192	0.166	0.149	0.126	0.103	0.075	0.039
	572	0.190	0.163	0.148	0.125	0.105	0.074	0.038
	571	0.189	0.163	0.146	0.123	0.101	0.073	0.040
	855	0.287	0.255	0.232	0.197	0.167	0.118	0.061
	1148	0.388	0.343	0.315	0.266	0.223	0.161	0.083
67.050	606	0.190	0.157	0.141	0.118	0.097	0.074	0.051
	603	0.189	0.155	0.141	0.117	0.095	0.074	0.050
	600	0.188	0.155	0.140	0.117	0.095	0.073	0.049
	874	0.271	0.231	0.209	0.177	0.150	0.115	0.072
	1152	0.359	0.306	0.280	0.238	0.202	0.157	0.097
67.100	605	0.191	0.152	0.132	0.103	0.076	0.058	0.033
	603	0.189	0.151	0.131	0.102	0.077	0.055	0.034
	600	0.188	0.150	0.131	0.101	0.076	0.055	0.034
	871	0.264	0.219	0.196	0.151	0.121	0.089	0.050
	1150	0.344	0.287	0.260	0.203	0.164	0.120	0.067
67.150	615	0.106	0.086	0.079	0.067	0.057	0.042	0.024
	612	0.106	0.089	0.080	0.068	0.059	0.045	0.026
	609	0.106	0.086	0.079	0.067	0.058	0.045	0.027
	880	0.152	0.130	0.119	0.102	0.087	0.065	0.039
	1152	0.206	0.175	0.162	0.140	0.120	0.090	0.052
67.200	603	0.127	0.106	0.098	0.087	0.075	0.053	0.029
	599	0.126	0.105	0.097	0.085	0.074	0.050	0.032
	597	0.126	0.104	0.095	0.087	0.073	0.053	0.029
	867	0.184	0.158	0.147	0.131	0.113	0.081	0.042
	1147	0.246	0.212	0.198	0.176	0.154	0.111	0.056
67.250	605	0.241	0.202	0.172	0.137	0.112	0.082	0.048
	604	0.237	0.200	0.171	0.135	0.112	0.080	0.050
	603	0.237	0.200	0.169	0.135	0.111	0.081	0.049
	877	0.343	0.292	0.255	0.206	0.171	0.124	0.075
	1151	0.444	0.382	0.335	0.274	0.229	0.167	0.099
67.300	574	0.147	0.124	0.110	0.086	0.065	0.045	0.019
	568	0.143	0.122	0.108	0.083	0.063	0.044	0.019
	567	0.143	0.121	0.107	0.083	0.063	0.043	0.019
	859	0.212	0.185	0.165	0.130	0.100	0.069	0.030
	1156	0.280	0.245	0.220	0.176	0.139	0.095	0.041

Table F.2(e) - Site 3 - Chainage 2.100 - 3.800 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
2.200	634	0.121	0.125	0.121	0.110	0.100	0.080	0.051
	627	0.121	0.124	0.120	0.109	0.099	0.081	0.052
	630	0.122	0.124	0.121	0.110	0.099	0.080	0.052
	890	0.181	0.187	0.182	0.165	0.149	0.122	0.075
	1146	0.243	0.250	0.242	0.222	0.199	0.165	0.099
2.250	588	0.262	0.221	0.204	0.178	0.150	0.111	0.060
	584	0.260	0.218	0.203	0.177	0.149	0.112	0.060
	584	0.260	0.219	0.203	0.177	0.149	0.109	0.061
	848	0.369	0.318	0.297	0.259	0.219	0.164	0.085
	1128	0.485	0.415	0.386	0.333	0.284	0.209	0.114
2.300	607	0.313	0.282	0.262	0.237	0.203	0.152	0.085
	605	0.315	0.281	0.262	0.234	0.202	0.153	0.086
	603	0.314	0.279	0.260	0.233	0.201	0.152	0.083
	861	0.458	0.428	0.402	0.352	0.304	0.230	0.124
	1128	0.638	0.570	0.536	0.467	0.402	0.304	0.160
2.400	610	0.316	0.274	0.249	0.209	0.170	0.119	0.064
	608	0.314	0.272	0.247	0.207	0.169	0.120	0.063
	608	0.315	0.272	0.247	0.207	0.169	0.119	0.063
	874	0.481	0.417	0.380	0.317	0.260	0.182	0.094
	1143	0.652	0.565	0.514	0.426	0.350	0.245	0.125
2.450	614	0.367	0.313	0.276	0.223	0.175	0.116	0.063
	630	0.376	0.319	0.284	0.229	0.181	0.121	0.066
	626	0.374	0.313	0.285	0.226	0.179	0.120	0.065
	891	0.543	0.464	0.418	0.336	0.268	0.179	0.094
	1152	0.717	0.611	0.553	0.444	0.356	0.238	0.126
2.500	621	0.175	0.164	0.156	0.142	0.122	0.089	0.054
	617	0.175	0.162	0.155	0.141	0.120	0.089	0.047
	614	0.174	0.162	0.155	0.140	0.121	0.088	0.053
	883	0.263	0.251	0.239	0.215	0.182	0.136	0.072
	1145	0.372	0.345	0.328	0.291	0.248	0.184	0.098
2.550	626	0.165	0.153	0.147	0.133	0.118	0.093	0.053
	613	0.162	0.149	0.143	0.131	0.117	0.091	0.052
	612	0.162	0.149	0.143	0.129	0.116	0.090	0.052
	874	0.236	0.228	0.219	0.197	0.176	0.138	0.077
	1132	0.326	0.309	0.296	0.267	0.236	0.184	0.103
2.600	600	0.319	0.282	0.261	0.223	0.186	0.135	0.079
	601	0.318	0.280	0.261	0.223	0.186	0.136	0.077
	600	0.318	0.280	0.261	0.222	0.187	0.135	0.076
	867	0.482	0.431	0.401	0.345	0.288	0.209	0.113
	1135	0.666	0.587	0.550	0.463	0.391	0.280	0.149
2.650	611	0.226	0.208	0.201	0.180	0.158	0.121	0.070
	611	0.226	0.208	0.200	0.179	0.157	0.122	0.070
	607	0.223	0.206	0.198	0.179	0.154	0.122	0.070
	866	0.337	0.313	0.299	0.270	0.235	0.185	0.105
	1127	0.447	0.421	0.400	0.356	0.316	0.243	0.138
2.700	605	0.285	0.253	0.234	0.200	0.167	0.116	0.053
	598	0.283	0.252	0.232	0.198	0.166	0.115	0.052
	601	0.284	0.253	0.233	0.198	0.165	0.117	0.053
	869	0.436	0.390	0.359	0.306	0.255	0.179	0.078
	1133	0.598	0.531	0.492	0.416	0.347	0.241	0.105
2.750	612	0.141	0.152	0.143	0.130	0.115	0.093	0.058
	606	0.141	0.149	0.140	0.128	0.114	0.092	0.062
	608	0.142	0.150	0.141	0.128	0.115	0.093	0.058
	869	0.208	0.228	0.216	0.196	0.177	0.144	0.088
	1135	0.263	0.311	0.293	0.265	0.235	0.192	0.117
2.800	612	0.158	0.147	0.140	0.127	0.114	0.089	0.053
	609	0.157	0.142	0.141	0.125	0.113	0.087	0.053
	609	0.157	0.144	0.140	0.126	0.113	0.087	0.050

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	874	0.244	0.225	0.215	0.195	0.173	0.137	0.081
	1134	0.334	0.307	0.293	0.264	0.234	0.183	0.102
2.850	616	0.367	0.316	0.284	0.219	0.163	0.104	0.050
	609	0.365	0.308	0.275	0.216	0.161	0.104	0.053
	608	0.358	0.307	0.274	0.215	0.160	0.106	0.052
	874	0.488	0.456	0.410	0.325	0.244	0.155	0.078
	1134	0.678	0.600	0.529	0.432	0.327	0.208	0.104
2.900	606	0.215	0.199	0.190	0.170	0.151	0.118	0.071
	599	0.210	0.196	0.189	0.172	0.149	0.117	0.071
	598	0.211	0.196	0.189	0.171	0.149	0.117	0.071
	857	0.316	0.303	0.289	0.261	0.231	0.180	0.107
	1126	0.438	0.408	0.392	0.352	0.310	0.242	0.140
2.950	610	0.190	0.181	0.171	0.153	0.134	0.103	0.057
	605	0.188	0.180	0.169	0.152	0.130	0.101	0.058
	607	0.189	0.179	0.170	0.152	0.134	0.103	0.057
	865	0.293	0.279	0.266	0.235	0.207	0.158	0.087
	1131	0.404	0.382	0.364	0.320	0.281	0.214	0.116
3.000	602	0.239	0.240	0.198	0.176	0.150	0.115	0.064
	595	0.233	0.237	0.196	0.174	0.149	0.114	0.062
	597	0.233	0.238	0.198	0.176	0.150	0.114	0.065
	856	0.353	0.360	0.311	0.277	0.240	0.181	0.092
	1121	0.478	0.481	0.425	0.377	0.327	0.242	0.128
3.050	610	0.172	0.155	0.145	0.129	0.112	0.088	0.051
	606	0.171	0.154	0.143	0.127	0.111	0.089	0.053
	605	0.169	0.153	0.143	0.127	0.111	0.088	0.052
	869	0.256	0.237	0.221	0.198	0.172	0.134	0.076
	1127	0.367	0.328	0.305	0.270	0.235	0.182	0.103
3.100	598	0.227	0.198	0.184	0.160	0.136	0.101	0.054
	590	0.223	0.195	0.182	0.158	0.134	0.100	0.056
	590	0.222	0.193	0.181	0.158	0.135	0.099	0.053
	854	0.332	0.310	0.288	0.249	0.212	0.157	0.081
	1125	0.479	0.426	0.397	0.342	0.291	0.215	0.109
3.150	566	0.223	0.190	0.179	0.159	0.141	0.107	0.058
	560	0.223	0.187	0.177	0.158	0.139	0.107	0.056
	559	0.225	0.187	0.177	0.157	0.139	0.106	0.057
	833	0.329	0.296	0.278	0.247	0.218	0.168	0.091
	1120	0.466	0.402	0.377	0.334	0.296	0.228	0.123
3.200	571	0.207	0.186	0.177	0.157	0.134	0.099	0.053
	566	0.205	0.183	0.176	0.157	0.133	0.097	0.055
	566	0.206	0.185	0.176	0.156	0.133	0.098	0.053
	841	0.328	0.295	0.278	0.244	0.208	0.155	0.084
	1124	0.454	0.406	0.381	0.333	0.286	0.211	0.114
3.250	580	0.300	0.256	0.228	0.184	0.142	0.085	0.034
	577	0.298	0.255	0.227	0.183	0.141	0.086	0.033
	575	0.299	0.254	0.227	0.182	0.141	0.087	0.033
	858	0.456	0.391	0.350	0.283	0.219	0.133	0.048
	1140	0.615	0.525	0.471	0.382	0.297	0.183	0.068
3.300	610	0.329	0.272	0.244	0.201	0.158	0.107	0.055
	602	0.321	0.266	0.239	0.198	0.156	0.105	0.055
	601	0.321	0.265	0.239	0.198	0.155	0.104	0.056
	868	0.487	0.409	0.366	0.304	0.243	0.163	0.084
	1138	0.660	0.552	0.493	0.409	0.331	0.221	0.110
3.350	597	0.373	0.325	0.295	0.218	0.168	0.104	0.045
	595	0.372	0.322	0.292	0.216	0.166	0.102	0.051
	592	0.371	0.319	0.290	0.213	0.164	0.101	0.049
	867	0.544	0.482	0.437	0.331	0.256	0.158	0.074
	1138	0.745	0.646	0.555	0.448	0.347	0.216	0.097
3.400	560	0.264	0.292	0.226	0.196	0.166	0.119	0.058
	554	0.261	0.283	0.225	0.195	0.160	0.116	0.063

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	555	0.264	0.284	0.225	0.195	0.164	0.119	0.059
	832	0.405	0.446	0.372	0.319	0.272	0.195	0.094
	1115	0.542	0.605	0.521	0.445	0.380	0.272	0.132
3.450	540	0.177	0.177	0.165	0.145	0.124	0.091	0.047
	539	0.177	0.175	0.165	0.144	0.122	0.091	0.046
	536	0.177	0.174	0.164	0.145	0.123	0.090	0.047
	825	0.277	0.283	0.263	0.230	0.195	0.143	0.070
	1111	0.380	0.390	0.366	0.318	0.270	0.198	0.099
3.500	543	0.183	0.175	0.167	0.153	0.133	0.105	0.061
	539	0.181	0.170	0.167	0.149	0.135	0.104	0.063
	538	0.181	0.171	0.167	0.151	0.132	0.104	0.060
	824	0.255	0.272	0.266	0.237	0.215	0.167	0.097
	1106	0.369	0.375	0.361	0.324	0.288	0.227	0.133
3.550	565	0.113	0.104	0.099	0.094	0.084	0.068	0.042
	556	0.112	0.100	0.098	0.091	0.080	0.065	0.042
	555	0.112	0.101	0.098	0.090	0.080	0.065	0.042
	822	0.169	0.161	0.154	0.144	0.130	0.105	0.066
	1111	0.231	0.218	0.208	0.192	0.174	0.140	0.088
3.600	547	0.192	0.171	0.158	0.139	0.114	0.085	0.043
	541	0.190	0.169	0.156	0.136	0.111	0.082	0.043
	539	0.190	0.167	0.154	0.135	0.112	0.082	0.043
	832	0.295	0.264	0.243	0.214	0.179	0.132	0.066
	1122	0.400	0.362	0.333	0.291	0.244	0.180	0.090
3.650	576	0.495	0.423	0.373	0.288	0.213	0.123	0.056
	573	0.487	0.417	0.367	0.284	0.211	0.124	0.056
	572	0.487	0.419	0.367	0.286	0.212	0.125	0.053
	851	0.745	0.651	0.571	0.451	0.342	0.201	0.074
	1126	1.003	0.870	0.767	0.608	0.461	0.270	0.106
3.700	574	0.631	0.448	0.357	0.263	0.197	0.124	0.071
	569	0.619	0.441	0.350	0.263	0.196	0.125	0.070
	568	0.618	0.439	0.350	0.261	0.195	0.124	0.070
	851	0.904	0.680	0.547	0.415	0.309	0.197	0.107
	1124	1.206	0.912	0.719	0.567	0.423	0.269	0.142
3.750	559	0.646	0.494	0.384	0.267	0.189	0.111	0.052
	555	0.631	0.485	0.379	0.265	0.187	0.112	0.054
	556	0.631	0.484	0.380	0.266	0.189	0.112	0.054
	841	0.929	0.732	0.586	0.421	0.305	0.180	0.085
	1121	1.222	0.970	0.784	0.573	0.421	0.250	0.116
3.800	580	0.240	0.219	0.202	0.177	0.150	0.110	0.056
	574	0.237	0.214	0.200	0.174	0.147	0.107	0.057
	573	0.237	0.213	0.201	0.174	0.148	0.108	0.055
	843	0.383	0.348	0.318	0.280	0.237	0.172	0.089
	1116	0.537	0.479	0.443	0.384	0.328	0.237	0.122

Table F.2(f) - Site 3 - Chainage 2.100 - 3.800 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
2.200	585	0.090	0.119	0.116	0.109	0.097	0.078	0.047
	584	0.105	0.117	0.115	0.107	0.096	0.078	0.049
	579	0.107	0.117	0.114	0.107	0.096	0.078	0.048
	837	0.176	0.182	0.180	0.169	0.151	0.122	0.074
	1103	0.235	0.245	0.243	0.227	0.202	0.162	0.098
2.250	591	0.157	0.142	0.135	0.123	0.110	0.087	0.051
	587	0.159	0.141	0.134	0.123	0.107	0.085	0.051
	586	0.160	0.141	0.135	0.122	0.110	0.088	0.050
	853	0.235	0.222	0.210	0.192	0.167	0.131	0.078
	1113	0.321	0.305	0.291	0.260	0.230	0.179	0.102

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
2.300	593	0.161	0.153	0.147	0.136	0.121	0.102	0.063
	592	0.161	0.152	0.147	0.138	0.122	0.099	0.067
	592	0.161	0.153	0.147	0.136	0.122	0.101	0.063
	858	0.247	0.238	0.228	0.211	0.189	0.156	0.098
	1119	0.336	0.323	0.308	0.283	0.255	0.208	0.129
2.341	569	0.141	0.137	0.127	0.112	0.096	0.072	0.047
	564	0.138	0.134	0.125	0.111	0.095	0.071	0.043
	563	0.139	0.135	0.126	0.111	0.096	0.072	0.042
	827	0.213	0.212	0.198	0.174	0.151	0.113	0.065
	1108	0.295	0.287	0.266	0.234	0.203	0.153	0.087
2.400	594	0.262	0.232	0.213	0.180	0.149	0.108	0.057
	594	0.263	0.232	0.213	0.181	0.150	0.108	0.064
	591	0.264	0.232	0.213	0.181	0.149	0.108	0.061
	862	0.401	0.364	0.332	0.283	0.235	0.168	0.088
	1128	0.559	0.498	0.453	0.385	0.320	0.227	0.125
2.450	585	0.190	0.170	0.161	0.140	0.120	0.093	0.052
	584	0.192	0.170	0.160	0.143	0.124	0.093	0.053
	581	0.190	0.168	0.159	0.142	0.122	0.092	0.052
	846	0.276	0.264	0.248	0.219	0.189	0.144	0.080
	1108	0.393	0.361	0.338	0.298	0.258	0.195	0.107
2.500	578	0.185	0.174	0.163	0.145	0.125	0.094	0.049
	575	0.185	0.173	0.165	0.144	0.127	0.091	0.053
	579	0.187	0.172	0.163	0.146	0.127	0.092	0.051
	840	0.285	0.273	0.256	0.226	0.195	0.147	0.071
	1112	0.397	0.375	0.349	0.308	0.265	0.199	0.100
2.550	599	0.239	0.211	0.198	0.173	0.148	0.106	0.053
	596	0.238	0.210	0.197	0.171	0.147	0.104	0.052
	593	0.240	0.211	0.197	0.171	0.148	0.104	0.053
	858	0.358	0.326	0.304	0.265	0.224	0.160	0.079
	1117	0.515	0.450	0.419	0.360	0.304	0.215	0.105
2.600	592	0.285	0.258	0.237	0.207	0.174	0.125	0.065
	589	0.283	0.255	0.236	0.205	0.173	0.123	0.065
	592	0.286	0.258	0.237	0.206	0.173	0.124	0.062
	859	0.445	0.397	0.368	0.319	0.266	0.191	0.093
	1122	0.611	0.540	0.504	0.434	0.361	0.257	0.126
2.650	593	0.223	0.217	0.200	0.177	0.152	0.115	0.059
	590	0.225	0.213	0.201	0.177	0.149	0.112	0.061
	585	0.225	0.211	0.200	0.176	0.147	0.110	0.060
	852	0.336	0.339	0.311	0.273	0.238	0.175	0.089
	1116	0.485	0.468	0.428	0.373	0.324	0.237	0.120
2.700	596	0.190	0.170	0.160	0.143	0.123	0.094	0.051
	592	0.189	0.171	0.159	0.142	0.123	0.093	0.052
	591	0.189	0.168	0.159	0.142	0.123	0.093	0.052
	855	0.282	0.266	0.249	0.219	0.189	0.143	0.076
	1121	0.400	0.370	0.344	0.304	0.261	0.196	0.101
2.750	579	0.240	0.210	0.194	0.169	0.141	0.101	0.051
	578	0.240	0.209	0.193	0.169	0.142	0.101	0.052
	579	0.240	0.210	0.193	0.169	0.142	0.101	0.051
	849	0.360	0.325	0.298	0.260	0.218	0.155	0.078
	1117	0.499	0.443	0.406	0.349	0.292	0.208	0.103
2.800	579	0.189	0.169	0.158	0.139	0.118	0.089	0.048
	576	0.187	0.167	0.157	0.138	0.117	0.089	0.050
	573	0.186	0.167	0.157	0.138	0.117	0.089	0.050
	838	0.297	0.263	0.246	0.216	0.184	0.138	0.078
	1107	0.409	0.363	0.337	0.294	0.252	0.187	0.106
2.850	590	0.491	0.406	0.352	0.260	0.190	0.111	0.044
	586	0.480	0.400	0.341	0.259	0.191	0.112	0.049
	585	0.479	0.397	0.341	0.257	0.189	0.112	0.047
	854	0.708	0.591	0.503	0.388	0.290	0.170	0.072

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	1115	0.926	0.769	0.661	0.508	0.382	0.226	0.096
2.900	580	0.163	0.149	0.140	0.125	0.110	0.081	0.042
	578	0.162	0.146	0.138	0.123	0.108	0.080	0.042
	576	0.162	0.147	0.139	0.123	0.108	0.080	0.043
	835	0.250	0.231	0.217	0.193	0.168	0.126	0.066
	1099	0.342	0.315	0.295	0.261	0.227	0.167	0.086
2.950	594	0.180	0.166	0.156	0.141	0.123	0.093	0.051
	593	0.179	0.165	0.155	0.140	0.122	0.091	0.052
	591	0.179	0.166	0.156	0.140	0.122	0.093	0.051
	852	0.268	0.259	0.243	0.217	0.189	0.143	0.079
	1115	0.377	0.355	0.333	0.294	0.255	0.193	0.106
2.999	602	0.239	0.213	0.203	0.179	0.157	0.115	0.065
	598	0.236	0.212	0.201	0.177	0.155	0.114	0.063
	595	0.233	0.211	0.200	0.178	0.153	0.115	0.065
	857	0.363	0.329	0.310	0.273	0.236	0.176	0.096
	1119	0.499	0.446	0.422	0.367	0.317	0.237	0.129
3.050	598	0.198	0.180	0.171	0.149	0.125	0.090	0.050
	601	0.197	0.182	0.172	0.150	0.126	0.091	0.051
	601	0.197	0.182	0.171	0.150	0.126	0.090	0.051
	860	0.301	0.282	0.263	0.230	0.194	0.142	0.073
	1122	0.417	0.387	0.360	0.312	0.264	0.189	0.102
3.100	576	0.183	0.171	0.162	0.145	0.124	0.095	0.050
	573	0.182	0.169	0.160	0.143	0.122	0.094	0.051
	572	0.181	0.169	0.160	0.143	0.122	0.092	0.052
	832	0.287	0.265	0.250	0.223	0.192	0.144	0.077
	1108	0.392	0.361	0.339	0.302	0.260	0.192	0.106
3.150	581	0.260	0.227	0.214	0.193	0.164	0.126	0.070
	580	0.259	0.226	0.214	0.192	0.163	0.126	0.072
	575	0.260	0.224	0.212	0.191	0.162	0.125	0.072
	833	0.391	0.357	0.337	0.301	0.258	0.197	0.109
	1104	0.555	0.491	0.463	0.411	0.352	0.267	0.146
3.200	591	0.216	0.193	0.188	0.160	0.139	0.099	0.047
	590	0.215	0.194	0.183	0.159	0.137	0.099	0.047
	590	0.216	0.195	0.182	0.160	0.137	0.100	0.052
	849	0.336	0.309	0.285	0.250	0.213	0.155	0.074
	1113	0.475	0.426	0.391	0.341	0.290	0.211	0.104
3.250	592	0.275	0.240	0.218	0.182	0.144	0.091	0.038
	591	0.272	0.239	0.218	0.182	0.143	0.091	0.038
	591	0.274	0.239	0.219	0.183	0.144	0.091	0.038
	861	0.434	0.381	0.342	0.285	0.226	0.142	0.057
	1129	0.611	0.531	0.474	0.393	0.310	0.195	0.077
3.300	600	0.240	0.228	0.220	0.200	0.143	0.100	0.050
	598	0.239	0.225	0.218	0.197	0.143	0.100	0.051
	597	0.238	0.225	0.216	0.197	0.143	0.099	0.050
	863	0.371	0.352	0.337	0.303	0.226	0.157	0.077
	1129	0.524	0.488	0.438	0.408	0.313	0.216	0.103
3.350	590	0.216	0.188	0.177	0.160	0.134	0.103	0.051
	595	0.214	0.189	0.178	0.159	0.136	0.103	0.051
	594	0.216	0.189	0.178	0.157	0.136	0.102	0.037
	856	0.316	0.293	0.273	0.248	0.206	0.156	0.067
	1119	0.450	0.399	0.370	0.338	0.277	0.211	0.095
3.399	573	0.186	0.173	0.164	0.142	0.123	0.090	0.048
	568	0.185	0.172	0.162	0.142	0.122	0.091	0.048
	567	0.186	0.171	0.162	0.141	0.122	0.090	0.048
	829	0.290	0.274	0.256	0.225	0.193	0.143	0.076
	1111	0.409	0.383	0.357	0.310	0.266	0.196	0.104
3.449	554	0.215	0.192	0.175	0.149	0.123	0.089	0.041
	553	0.214	0.191	0.174	0.148	0.122	0.087	0.043
	558	0.215	0.193	0.176	0.150	0.124	0.088	0.043

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	841	0.329	0.305	0.277	0.236	0.197	0.140	0.068
	1124	0.453	0.418	0.380	0.323	0.271	0.192	0.093
3.500	560	0.166	0.167	0.156	0.134	0.109	0.075	0.034
	557	0.166	0.164	0.154	0.132	0.108	0.075	0.036
	553	0.169	0.164	0.154	0.132	0.109	0.076	0.036
	838	0.268	0.269	0.250	0.215	0.177	0.123	0.058
	1125	0.375	0.378	0.351	0.301	0.250	0.174	0.081
3.546	583	0.166	0.157	0.150	0.140	0.126	0.101	0.065
	587	0.170	0.157	0.151	0.141	0.128	0.103	0.066
	580	0.168	0.157	0.150	0.138	0.125	0.100	0.063
	840	0.255	0.249	0.240	0.221	0.199	0.161	0.100
	1119	0.354	0.339	0.327	0.299	0.271	0.219	0.135
3.600	562	0.372	0.313	0.271	0.213	0.163	0.101	0.047
	560	0.370	0.311	0.269	0.212	0.164	0.102	0.047
	561	0.373	0.308	0.269	0.215	0.163	0.102	0.047
	846	0.560	0.477	0.418	0.334	0.262	0.165	0.073
	1133	0.761	0.648	0.569	0.457	0.363	0.230	0.100
3.650	580	0.232	0.203	0.192	0.178	0.152	0.120	0.071
	577	0.232	0.204	0.193	0.179	0.153	0.120	0.073
	574	0.231	0.202	0.191	0.178	0.152	0.120	0.073
	843	0.371	0.331	0.310	0.275	0.244	0.189	0.116
	1122	0.508	0.456	0.424	0.372	0.327	0.252	0.148
3.700	559	0.265	0.232	0.205	0.173	0.146	0.109	0.065
	555	0.266	0.230	0.204	0.172	0.145	0.106	0.065
	556	0.268	0.231	0.205	0.172	0.146	0.107	0.065
	846	0.416	0.361	0.318	0.267	0.224	0.165	0.098
	1134	0.573	0.493	0.436	0.364	0.303	0.222	0.129
3.750	576	0.296	0.255	0.223	0.189	0.156	0.111	0.059
	566	0.292	0.251	0.218	0.186	0.153	0.110	0.058
	563	0.291	0.250	0.218	0.185	0.153	0.109	0.059
	841	0.462	0.396	0.344	0.292	0.242	0.173	0.089
	1125	0.650	0.545	0.469	0.399	0.331	0.236	0.122
3.800	599	0.224	0.192	0.182	0.160	0.139	0.106	0.052
	594	0.224	0.191	0.180	0.159	0.138	0.105	0.055
	596	0.225	0.192	0.183	0.160	0.139	0.106	0.055
	857	0.334	0.305	0.285	0.252	0.218	0.165	0.083
	1133	0.488	0.425	0.393	0.348	0.299	0.224	0.115

Table F.2(g) - Site 4 - Chainage 28.500 - 29.600 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
28.500	630	1.095	0.809	0.593	0.349	0.212	0.100	0.044
	632	1.070	0.793	0.577	0.346	0.213	0.106	0.043
	632	1.064	0.793	0.574	0.344	0.214	0.105	0.045
	915	1.410	1.074	0.802	0.501	0.313	0.153	0.066
	1186	1.742	1.323	0.961	0.654	0.411	0.203	0.081
28.601	626	1.000	0.762	0.623	0.442	0.315	0.176	0.062
	621	0.979	0.747	0.610	0.451	0.314	0.178	0.069
	626	0.980	0.751	0.613	0.454	0.316	0.177	0.073
	900	1.356	1.057	0.874	0.653	0.461	0.261	0.095
	1157	1.737	1.358	1.128	0.845	0.598	0.341	0.127
28.700	567	1.626	1.178	0.863	0.530	0.322	0.163	0.087
	566	1.566	1.158	0.849	0.541	0.338	0.172	0.087
	567	1.551	1.152	0.843	0.547	0.341	0.172	0.090
	841	1.999	1.638	1.229	0.820	0.520	0.264	0.135
	1095	1.999	1.999	1.577	1.077	0.696	0.356	0.177
28.800	613	0.646	0.539	0.468	0.364	0.272	0.167	0.077

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	608	0.636	0.531	0.464	0.361	0.270	0.164	0.076
	607	0.636	0.532	0.465	0.362	0.270	0.163	0.078
	878	0.934	0.790	0.697	0.549	0.415	0.252	0.115
	1139	1.232	1.043	0.920	0.731	0.559	0.343	0.152
28.900	602	1.008	0.725	0.544	0.370	0.257	0.133	0.066
	598	0.966	0.710	0.534	0.370	0.258	0.137	0.064
	599	0.959	0.710	0.536	0.372	0.259	0.140	0.065
	879	1.359	1.024	0.789	0.564	0.396	0.211	0.092
	1143	1.731	1.309	0.962	0.746	0.526	0.287	0.117
29.000	633	0.445	0.314	0.242	0.176	0.127	0.068	0.027
	628	0.439	0.311	0.237	0.174	0.128	0.068	0.028
	627	0.438	0.311	0.237	0.174	0.128	0.068	0.028
	916	0.604	0.443	0.339	0.249	0.182	0.097	0.039
	1193	0.788	0.567	0.441	0.322	0.232	0.124	0.050
29.110	634	0.877	0.543	0.370	0.208	0.116	0.043	0.022
	632	0.856	0.537	0.364	0.210	0.118	0.045	0.025
	633	0.851	0.536	0.363	0.208	0.118	0.045	0.024
	923	1.169	0.735	0.503	0.291	0.167	0.067	0.031
	1197	1.501	0.919	0.616	0.371	0.213	0.084	0.042
29.200	574	0.573	0.465	0.404	0.312	0.236	0.154	0.085
	570	0.567	0.462	0.401	0.312	0.239	0.159	0.083
	566	0.562	0.458	0.399	0.309	0.237	0.158	0.082
	850	0.827	0.683	0.597	0.471	0.368	0.247	0.125
	1126	1.074	0.892	0.781	0.620	0.487	0.329	0.165
29.300	586	1.196	0.799	0.592	0.375	0.244	0.142	0.076
	583	1.150	0.775	0.606	0.377	0.247	0.145	0.078
	579	1.134	0.770	0.600	0.375	0.246	0.144	0.079
	868	1.572	1.119	0.875	0.563	0.376	0.223	0.117
	1142	1.972	1.430	1.100	0.743	0.524	0.303	0.159
29.400	622	0.368	0.280	0.237	0.176	0.129	0.068	0.030
	622	0.364	0.276	0.234	0.175	0.127	0.071	0.029
	620	0.365	0.276	0.234	0.175	0.127	0.070	0.029
	891	0.535	0.407	0.344	0.258	0.188	0.105	0.040
	1165	0.742	0.549	0.462	0.345	0.255	0.139	0.052
29.500	589	1.163	0.863	0.706	0.462	0.306	0.169	0.091
	584	1.133	0.843	0.683	0.456	0.305	0.171	0.094
	583	1.130	0.842	0.679	0.456	0.306	0.172	0.096
	872	1.589	1.225	0.998	0.680	0.460	0.259	0.136
	1141	1.999	1.578	1.276	0.892	0.607	0.343	0.177
29.600	614	0.515	0.429	0.385	0.303	0.229	0.133	0.047
	613	0.510	0.426	0.382	0.301	0.227	0.133	0.049
	608	0.510	0.426	0.381	0.301	0.226	0.132	0.050
	893	0.734	0.616	0.550	0.433	0.329	0.194	0.073
	1164	0.978	0.809	0.722	0.564	0.436	0.258	0.093

Table F.2(h) - Site 4 - Chainage 28.500 - 29.600 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
28.500	624	0.796	0.580	0.465	0.314	0.216	0.109	0.046
	625	0.780	0.568	0.449	0.312	0.224	0.114	0.042
	621	0.778	0.563	0.448	0.311	0.221	0.112	0.044
	917	1.053	0.780	0.619	0.437	0.313	0.164	0.060
	1184	1.315	0.965	0.755	0.544	0.390	0.203	0.083
28.600	586	1.446	1.020	0.723	0.464	0.303	0.174	0.101
	587	1.390	0.994	0.713	0.457	0.302	0.176	0.103
	584	1.368	0.982	0.706	0.452	0.301	0.177	0.101
	859	1.839	1.363	1.007	0.662	0.446	0.263	0.150

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	1113	2.267	1.703	1.280	0.855	0.591	0.350	0.189
28.699	600	1.259	0.917	0.726	0.499	0.335	0.178	0.084
	596	1.246	0.910	0.716	0.495	0.334	0.177	0.085
	599	1.243	0.909	0.715	0.494	0.333	0.177	0.084
	880	1.665	1.245	0.997	0.706	0.484	0.263	0.122
	1145	2.042	1.539	1.236	0.893	0.626	0.340	0.159
28.800	613	0.927	0.572	0.413	0.243	0.141	0.063	0.047
	609	0.901	0.560	0.404	0.242	0.143	0.066	0.047
	608	0.896	0.557	0.403	0.240	0.143	0.067	0.048
	888	1.192	0.769	0.567	0.346	0.210	0.100	0.066
	1158	1.502	0.966	0.720	0.450	0.278	0.134	0.088
28.900	634	1.083	0.667	0.456	0.281	0.185	0.075	0.039
	629	1.046	0.647	0.438	0.276	0.178	0.079	0.043
	628	1.042	0.643	0.438	0.273	0.176	0.078	0.039
	922	1.361	0.856	0.594	0.385	0.247	0.116	0.060
	1191	1.658	1.032	0.724	0.478	0.308	0.151	0.075
29.000	627	0.456	0.345	0.266	0.178	0.114	0.051	0.021
	626	0.444	0.334	0.263	0.173	0.114	0.052	0.024
	625	0.443	0.335	0.261	0.174	0.113	0.052	0.024
	909	0.603	0.456	0.355	0.239	0.155	0.075	0.034
	1179	0.767	0.570	0.441	0.299	0.194	0.094	0.040
29.100	615	0.864	0.623	0.473	0.312	0.207	0.095	0.040
	611	0.843	0.603	0.463	0.310	0.206	0.100	0.046
	612	0.840	0.603	0.463	0.312	0.206	0.100	0.041
	912	1.151	0.821	0.630	0.431	0.282	0.136	0.060
	1194	1.447	1.019	0.763	0.530	0.345	0.167	0.073
29.200	590	0.636	0.531	0.410	0.296	0.218	0.127	0.051
	586	0.622	0.520	0.407	0.293	0.216	0.126	0.052
	586	0.619	0.517	0.406	0.291	0.215	0.125	0.049
	873	0.870	0.729	0.587	0.431	0.324	0.190	0.076
	1149	1.096	0.921	0.733	0.560	0.426	0.253	0.099
29.300	623	0.797	0.470	0.366	0.256	0.175	0.092	0.034
	623	0.779	0.460	0.360	0.250	0.170	0.090	0.035
	623	0.776	0.458	0.357	0.249	0.170	0.091	0.037
	914	1.109	0.654	0.506	0.359	0.250	0.134	0.050
	1193	1.418	0.838	0.645	0.458	0.320	0.170	0.064
29.400	616	0.608	0.460	0.379	0.277	0.195	0.098	0.039
	614	0.596	0.453	0.372	0.272	0.191	0.098	0.040
	612	0.593	0.450	0.370	0.270	0.192	0.098	0.036
	901	0.848	0.643	0.525	0.384	0.268	0.138	0.053
	1178	1.099	0.820	0.668	0.488	0.345	0.177	0.061
29.500	616	0.542	0.430	0.366	0.270	0.183	0.097	0.043
	612	0.525	0.415	0.355	0.264	0.181	0.099	0.041
	611	0.523	0.415	0.354	0.264	0.180	0.100	0.044
	896	0.753	0.614	0.521	0.389	0.269	0.144	0.058
	1167	0.991	0.800	0.682	0.509	0.355	0.187	0.073
29.600	613	0.659	0.481	0.384	0.273	0.192	0.110	0.042
	612	0.647	0.469	0.376	0.269	0.191	0.112	0.048
	613	0.645	0.468	0.375	0.269	0.191	0.113	0.047
	903	0.927	0.683	0.549	0.398	0.285	0.169	0.069
	1177	1.196	0.882	0.712	0.521	0.374	0.224	0.091

Table F.2(i) - Site 5 - Chainage 4.800 - 7.400 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
4.800	631	0.187	0.176	0.168	0.152	0.135	0.105	0.074
	627	0.186	0.175	0.166	0.152	0.133	0.105	0.073

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	627	0.187	0.175	0.165	0.154	0.133	0.102	0.073
	891	0.270	0.264	0.251	0.223	0.199	0.162	0.100
	1152	0.375	0.355	0.337	0.301	0.265	0.208	0.128
4.900	605	0.231	0.211	0.198	0.171	0.145	0.106	0.048
	604	0.232	0.209	0.196	0.169	0.144	0.105	0.050
	599	0.229	0.208	0.195	0.168	0.143	0.106	0.053
	871	0.336	0.320	0.300	0.258	0.220	0.160	0.083
	1143	0.474	0.433	0.404	0.346	0.296	0.216	0.128
5.000	610	0.200	0.191	0.176	0.153	0.133	0.097	0.065
	610	0.206	0.190	0.175	0.152	0.134	0.103	0.060
	613	0.206	0.190	0.176	0.153	0.138	0.106	0.059
	888	0.286	0.287	0.270	0.240	0.210	0.160	0.095
	1148	0.404	0.389	0.361	0.316	0.275	0.210	0.124
5.100	623	0.189	0.182	0.172	0.149	0.127	0.090	0.050
	620	0.195	0.184	0.171	0.148	0.128	0.089	0.045
	623	0.197	0.184	0.171	0.149	0.127	0.089	0.053
	885	0.258	0.277	0.257	0.223	0.191	0.135	0.069
	1148	0.381	0.374	0.347	0.298	0.255	0.179	0.089
5.201	622	0.162	0.144	0.136	0.117	0.097	0.067	0.031
	621	0.160	0.143	0.134	0.115	0.096	0.066	0.032
	622	0.160	0.142	0.135	0.116	0.097	0.066	0.031
	894	0.230	0.213	0.200	0.172	0.143	0.100	0.047
	1157	0.317	0.282	0.264	0.226	0.190	0.130	0.061
5.300	604	0.277	0.242	0.225	0.186	0.156	0.101	0.036
	600	0.273	0.238	0.220	0.185	0.153	0.102	0.038
	601	0.273	0.238	0.222	0.183	0.154	0.100	0.035
	874	0.415	0.363	0.336	0.278	0.233	0.152	0.050
	1144	0.564	0.487	0.446	0.373	0.309	0.203	0.069
5.400	602	0.191	0.172	0.161	0.138	0.115	0.081	0.039
	602	0.191	0.172	0.160	0.139	0.117	0.085	0.045
	603	0.191	0.168	0.163	0.136	0.113	0.080	0.041
	875	0.291	0.262	0.247	0.211	0.178	0.128	0.064
	1147	0.405	0.358	0.336	0.286	0.241	0.172	0.087
5.500	597	0.189	0.173	0.165	0.147	0.130	0.095	0.044
	598	0.188	0.174	0.164	0.147	0.128	0.095	0.044
	596	0.187	0.174	0.164	0.147	0.128	0.095	0.044
	868	0.277	0.260	0.247	0.219	0.191	0.140	0.063
	1138	0.378	0.348	0.328	0.290	0.251	0.183	0.082
5.600	604	0.171	0.167	0.156	0.136	0.115	0.082	0.040
	600	0.175	0.167	0.155	0.134	0.113	0.082	0.039
	601	0.177	0.168	0.157	0.134	0.113	0.081	0.039
	870	0.253	0.252	0.233	0.202	0.171	0.125	0.056
	1143	0.357	0.339	0.313	0.269	0.226	0.159	0.078
5.700	606	0.129	0.124	0.117	0.102	0.088	0.065	0.037
	605	0.129	0.124	0.116	0.101	0.087	0.065	0.036
	602	0.128	0.123	0.116	0.101	0.088	0.065	0.036
	870	0.200	0.189	0.177	0.154	0.134	0.099	0.054
	1140	0.277	0.259	0.242	0.210	0.181	0.134	0.073
5.800	606	0.312	0.237	0.202	0.162	0.128	0.085	0.044
	605	0.308	0.236	0.200	0.160	0.127	0.083	0.046
	608	0.309	0.236	0.201	0.161	0.127	0.083	0.048
	884	0.461	0.358	0.303	0.241	0.194	0.131	0.066
	1158	0.615	0.476	0.405	0.322	0.259	0.175	0.091
5.906	596	0.221	0.205	0.197	0.177	0.155	0.117	0.053
	597	0.220	0.203	0.196	0.175	0.154	0.115	0.051
	593	0.219	0.205	0.195	0.174	0.154	0.115	0.052
	865	0.325	0.311	0.298	0.265	0.232	0.173	0.077
	1133	0.450	0.420	0.401	0.353	0.309	0.228	0.098
6.000	595	0.284	0.255	0.241	0.210	0.182	0.136	0.066

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	595	0.283	0.253	0.240	0.209	0.180	0.135	0.066
	596	0.283	0.253	0.240	0.208	0.180	0.135	0.066
	863	0.423	0.383	0.361	0.315	0.271	0.203	0.104
	1133	0.580	0.511	0.481	0.417	0.359	0.268	0.138
6.100	614	0.579	0.493	0.442	0.362	0.298	0.213	0.100
	609	0.572	0.485	0.435	0.356	0.288	0.206	0.100
	608	0.573	0.486	0.436	0.356	0.288	0.206	0.101
	892	0.797	0.678	0.604	0.495	0.406	0.291	0.143
	1165	1.027	0.864	0.743	0.625	0.514	0.368	0.185
6.201	600	0.253	0.228	0.211	0.181	0.152	0.112	0.062
	594	0.249	0.224	0.208	0.178	0.150	0.110	0.061
	593	0.249	0.224	0.208	0.178	0.150	0.110	0.061
	866	0.380	0.344	0.317	0.272	0.229	0.168	0.092
	1134	0.516	0.461	0.425	0.364	0.305	0.223	0.122
6.300	610	0.425	0.381	0.351	0.291	0.240	0.144	0.059
	613	0.423	0.377	0.349	0.289	0.237	0.143	0.059
	610	0.422	0.376	0.348	0.289	0.237	0.142	0.060
	886	0.594	0.530	0.487	0.404	0.331	0.204	0.085
	1160	0.779	0.682	0.626	0.518	0.424	0.262	0.112
6.400	602	0.247	0.206	0.187	0.153	0.124	0.082	0.038
	599	0.248	0.202	0.184	0.151	0.122	0.085	0.035
	601	0.249	0.202	0.184	0.151	0.122	0.082	0.039
	874	0.351	0.304	0.275	0.226	0.182	0.123	0.059
	1144	0.505	0.412	0.369	0.302	0.242	0.162	0.077
6.500	598	0.193	0.169	0.155	0.128	0.104	0.069	0.030
	599	0.195	0.170	0.156	0.129	0.106	0.070	0.028
	599	0.196	0.170	0.157	0.130	0.106	0.070	0.030
	869	0.290	0.257	0.237	0.196	0.160	0.108	0.044
	1135	0.408	0.351	0.322	0.266	0.216	0.146	0.059
6.607	587	0.194	0.179	0.172	0.154	0.136	0.106	0.062
	588	0.193	0.178	0.172	0.154	0.136	0.106	0.061
	586	0.192	0.178	0.172	0.153	0.136	0.105	0.062
	858	0.294	0.276	0.264	0.237	0.208	0.164	0.093
	1130	0.404	0.372	0.355	0.317	0.281	0.217	0.124
6.700	607	0.203	0.181	0.165	0.138	0.112	0.071	0.026
	607	0.203	0.179	0.163	0.137	0.110	0.070	0.025
	604	0.204	0.179	0.162	0.136	0.110	0.069	0.026
	886	0.300	0.272	0.247	0.204	0.164	0.103	0.037
	1164	0.427	0.370	0.332	0.274	0.217	0.136	0.048
6.800	600	0.278	0.248	0.232	0.190	0.150	0.097	0.038
	600	0.276	0.247	0.228	0.188	0.150	0.096	0.038
	597	0.275	0.245	0.227	0.187	0.148	0.096	0.039
	876	0.422	0.379	0.348	0.287	0.228	0.147	0.058
	1154	0.589	0.515	0.474	0.387	0.308	0.197	0.077
6.900	587	0.198	0.181	0.172	0.152	0.131	0.092	0.053
	580	0.196	0.180	0.169	0.151	0.130	0.092	0.049
	579	0.196	0.180	0.169	0.150	0.128	0.094	0.050
	852	0.298	0.274	0.260	0.229	0.199	0.139	0.080
	1126	0.401	0.365	0.343	0.302	0.262	0.185	0.100
7.000	601	0.183	0.168	0.156	0.130	0.107	0.072	0.037
	600	0.183	0.166	0.153	0.129	0.104	0.072	0.035
	596	0.183	0.166	0.152	0.128	0.102	0.072	0.035
	868	0.267	0.254	0.233	0.195	0.159	0.109	0.054
	1144	0.366	0.345	0.314	0.261	0.211	0.144	0.070
7.100	587	0.260	0.239	0.209	0.168	0.130	0.080	0.035
	587	0.258	0.236	0.209	0.167	0.129	0.080	0.033
	586	0.261	0.233	0.207	0.168	0.129	0.078	0.035
	851	0.406	0.367	0.326	0.256	0.202	0.128	0.050
	1130	0.567	0.498	0.441	0.348	0.274	0.173	0.068

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
7.200	586	0.249	0.216	0.193	0.155	0.122	0.079	0.034
	583	0.248	0.211	0.191	0.154	0.119	0.079	0.037
	583	0.248	0.213	0.190	0.153	0.121	0.077	0.035
	849	0.378	0.333	0.298	0.240	0.190	0.123	0.054
	1126	0.528	0.455	0.404	0.325	0.259	0.169	0.071
7.300	579	0.252	0.224	0.201	0.158	0.121	0.076	0.033
	574	0.252	0.221	0.195	0.155	0.117	0.075	0.032
	571	0.252	0.220	0.196	0.155	0.118	0.075	0.032
	842	0.385	0.342	0.308	0.245	0.190	0.118	0.053
	1117	0.532	0.465	0.413	0.329	0.254	0.162	0.071

Table F.2(j) - Site 5 - Chainage 4.800 - 7.400 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
4.799	586	0.162	0.147	0.140	0.123	0.105	0.079	0.049
	587	0.162	0.147	0.138	0.122	0.104	0.078	0.048
	585	0.162	0.146	0.138	0.121	0.103	0.077	0.049
	849	0.248	0.227	0.213	0.188	0.161	0.120	0.082
	1114	0.342	0.310	0.290	0.254	0.217	0.161	0.111
4.899	585	0.161	0.151	0.144	0.133	0.117	0.093	0.051
	585	0.159	0.150	0.144	0.131	0.116	0.092	0.054
	586	0.158	0.149	0.144	0.131	0.115	0.093	0.051
	847	0.247	0.233	0.225	0.206	0.183	0.147	0.079
	1112	0.339	0.320	0.309	0.282	0.254	0.203	0.104
5.000	579	0.246	0.235	0.223	0.203	0.177	0.139	0.083
	574	0.244	0.232	0.221	0.200	0.175	0.137	0.083
	572	0.246	0.232	0.221	0.199	0.174	0.136	0.079
	839	0.372	0.359	0.341	0.306	0.268	0.209	0.121
	1105	0.505	0.476	0.454	0.405	0.354	0.276	0.159
5.100	555	0.175	0.151	0.134	0.110	0.086	0.057	0.022
	548	0.171	0.147	0.131	0.106	0.085	0.054	0.023
	547	0.171	0.146	0.130	0.107	0.084	0.055	0.021
	830	0.260	0.228	0.205	0.165	0.133	0.085	0.036
	1119	0.358	0.311	0.276	0.224	0.178	0.116	0.047
5.200	548	0.235	0.217	0.198	0.170	0.141	0.094	0.040
	544	0.233	0.213	0.196	0.168	0.138	0.094	0.040
	545	0.235	0.213	0.196	0.168	0.139	0.095	0.040
	826	0.358	0.329	0.304	0.259	0.214	0.146	0.061
	1112	0.477	0.435	0.403	0.342	0.283	0.194	0.082
5.299	593	0.264	0.237	0.217	0.182	0.147	0.099	0.041
	594	0.262	0.234	0.215	0.180	0.146	0.098	0.042
	591	0.262	0.234	0.215	0.180	0.146	0.098	0.042
	868	0.401	0.357	0.330	0.275	0.221	0.149	0.063
	1135	0.541	0.478	0.439	0.364	0.293	0.196	0.085
5.397	591	0.225	0.196	0.182	0.162	0.140	0.105	0.057
	592	0.225	0.197	0.183	0.164	0.140	0.108	0.055
	587	0.224	0.193	0.181	0.160	0.140	0.103	0.057
	854	0.339	0.304	0.282	0.250	0.216	0.162	0.086
	1120	0.466	0.417	0.385	0.340	0.293	0.219	0.112
5.499	600	0.214	0.193	0.180	0.158	0.133	0.096	0.045
	598	0.211	0.190	0.178	0.156	0.132	0.094	0.043
	595	0.210	0.189	0.178	0.156	0.132	0.094	0.044
	862	0.316	0.287	0.269	0.235	0.198	0.142	0.066
	1120	0.425	0.385	0.359	0.312	0.264	0.187	0.084
5.600	558	0.196	0.185	0.168	0.139	0.109	0.067	0.033
	555	0.199	0.183	0.166	0.138	0.107	0.067	0.034
	555	0.200	0.183	0.166	0.137	0.108	0.067	0.033

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	838	0.302	0.284	0.258	0.212	0.167	0.103	0.050
	1129	0.422	0.385	0.350	0.285	0.224	0.137	0.066
5.694	559	0.419	0.345	0.303	0.238	0.178	0.107	0.045
	552	0.412	0.336	0.297	0.232	0.173	0.105	0.045
	549	0.411	0.336	0.295	0.230	0.174	0.104	0.043
	838	0.612	0.502	0.439	0.346	0.261	0.159	0.066
	1119	0.804	0.654	0.572	0.452	0.342	0.210	0.089
5.800	588	0.393	0.264	0.211	0.147	0.102	0.064	0.035
	582	0.388	0.260	0.207	0.145	0.102	0.064	0.034
	584	0.388	0.260	0.209	0.145	0.101	0.062	0.037
	864	0.560	0.391	0.314	0.221	0.157	0.098	0.052
	1137	0.741	0.513	0.416	0.293	0.207	0.126	0.074
5.900	572	0.479	0.351	0.291	0.216	0.159	0.097	0.044
	569	0.470	0.345	0.287	0.214	0.158	0.097	0.044
	567	0.470	0.344	0.286	0.214	0.158	0.097	0.045
	849	0.675	0.505	0.420	0.315	0.234	0.144	0.066
	1131	0.861	0.649	0.542	0.409	0.305	0.190	0.089
6.000	581	0.282	0.223	0.196	0.155	0.121	0.075	0.040
	576	0.279	0.220	0.193	0.154	0.120	0.076	0.039
	571	0.274	0.219	0.193	0.153	0.120	0.077	0.037
	842	0.388	0.332	0.290	0.232	0.181	0.117	0.056
	1116	0.545	0.444	0.387	0.307	0.241	0.157	0.076
6.100	604	0.287	0.233	0.198	0.146	0.108	0.062	0.031
	606	0.287	0.229	0.196	0.145	0.106	0.063	0.032
	607	0.287	0.228	0.196	0.146	0.105	0.063	0.032
	880	0.421	0.341	0.293	0.220	0.162	0.097	0.048
	1148	0.572	0.454	0.391	0.294	0.217	0.131	0.062
6.200	604	0.282	0.247	0.226	0.192	0.155	0.109	0.061
	602	0.280	0.243	0.223	0.189	0.153	0.107	0.061
	603	0.278	0.243	0.223	0.189	0.153	0.107	0.061
	877	0.420	0.376	0.344	0.291	0.236	0.166	0.094
	1141	0.589	0.510	0.466	0.393	0.318	0.223	0.126
6.300	617	0.257	0.211	0.190	0.149	0.113	0.070	0.022
	621	0.255	0.210	0.187	0.148	0.114	0.067	0.029
	620	0.255	0.210	0.189	0.148	0.112	0.067	0.025
	885	0.357	0.305	0.275	0.216	0.165	0.098	0.035
	1144	0.484	0.404	0.359	0.283	0.218	0.131	0.042
6.398	611	0.213	0.166	0.144	0.110	0.082	0.048	0.022
	605	0.209	0.163	0.141	0.108	0.080	0.049	0.023
	609	0.211	0.164	0.142	0.108	0.081	0.047	0.022
	880	0.333	0.259	0.223	0.170	0.125	0.074	0.034
	1145	0.471	0.360	0.308	0.234	0.170	0.101	0.044
6.500	603	0.402	0.352	0.329	0.277	0.228	0.161	0.075
	596	0.394	0.348	0.319	0.271	0.226	0.159	0.075
	596	0.395	0.347	0.320	0.271	0.225	0.159	0.075
	869	0.571	0.510	0.466	0.393	0.327	0.231	0.110
	1138	0.775	0.666	0.609	0.509	0.422	0.298	0.144
6.600	593	0.267	0.244	0.239	0.220	0.203	0.166	0.100
	594	0.262	0.242	0.236	0.218	0.201	0.165	0.099
	595	0.263	0.242	0.236	0.218	0.200	0.165	0.100
	856	0.369	0.364	0.351	0.323	0.295	0.239	0.143
	1127	0.507	0.486	0.467	0.425	0.385	0.309	0.186
6.700	600	0.260	0.230	0.214	0.174	0.140	0.090	0.035
	597	0.259	0.230	0.210	0.174	0.138	0.089	0.036
	596	0.259	0.228	0.213	0.172	0.139	0.088	0.034
	871	0.390	0.344	0.316	0.259	0.205	0.132	0.049
	1150	0.537	0.466	0.424	0.347	0.273	0.175	0.064
6.800	615	0.845	0.633	0.500	0.338	0.224	0.130	0.065
	622	0.835	0.631	0.488	0.333	0.225	0.134	0.067

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	618	0.830	0.624	0.482	0.329	0.223	0.134	0.067
	901	1.148	0.869	0.674	0.471	0.329	0.204	0.102
	1172	1.499	1.105	0.833	0.604	0.430	0.276	0.139
6.899	600	0.348	0.300	0.276	0.235	0.192	0.142	0.064
	604	0.346	0.296	0.273	0.235	0.194	0.136	0.068
	608	0.351	0.295	0.274	0.236	0.195	0.141	0.064
	872	0.503	0.447	0.411	0.353	0.292	0.197	0.103
	1139	0.700	0.603	0.549	0.464	0.380	0.264	0.126
7.000	588	0.259	0.204	0.170	0.135	0.098	0.061	0.023
	585	0.256	0.202	0.169	0.134	0.097	0.060	0.023
	590	0.258	0.204	0.171	0.135	0.099	0.061	0.023
	872	0.387	0.307	0.267	0.207	0.153	0.091	0.035
	1149	0.521	0.413	0.355	0.277	0.205	0.122	0.045
7.100	593	0.325	0.272	0.242	0.188	0.142	0.080	0.026
	594	0.323	0.271	0.238	0.187	0.141	0.082	0.033
	592	0.321	0.270	0.239	0.185	0.141	0.082	0.029
	862	0.461	0.403	0.354	0.278	0.212	0.127	0.046
	1132	0.615	0.531	0.470	0.366	0.281	0.167	0.061
7.200	600	0.253	0.221	0.201	0.166	0.133	0.087	0.036
	602	0.253	0.221	0.200	0.167	0.133	0.086	0.037
	599	0.251	0.221	0.199	0.165	0.132	0.085	0.036
	867	0.377	0.338	0.304	0.250	0.201	0.131	0.054
	1137	0.530	0.456	0.410	0.333	0.269	0.175	0.072
7.300	588	0.418	0.303	0.260	0.196	0.145	0.088	0.036
	582	0.415	0.299	0.257	0.195	0.145	0.089	0.037
	579	0.412	0.299	0.256	0.194	0.144	0.088	0.036
	848	0.583	0.442	0.380	0.290	0.219	0.135	0.053
	1127	0.769	0.575	0.497	0.379	0.290	0.181	0.072

Table F.2(k) - Site 6 - Chainage 153.600 - 155.100 – Left Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
153.600	622	0.592	0.491	0.429	0.340	0.267	0.163	0.081
	620	0.585	0.485	0.424	0.338	0.265	0.163	0.082
	618	0.583	0.484	0.424	0.338	0.266	0.163	0.083
	905	0.824	0.700	0.617	0.498	0.396	0.249	0.122
	1182	1.049	0.891	0.789	0.642	0.515	0.326	0.160
153.700	639	0.497	0.430	0.385	0.278	0.225	0.154	0.089
	635	0.488	0.422	0.378	0.272	0.223	0.154	0.086
	634	0.487	0.421	0.377	0.271	0.223	0.155	0.085
	920	0.701	0.611	0.548	0.412	0.338	0.233	0.128
	1195	0.917	0.792	0.709	0.543	0.447	0.309	0.167
153.800	623	0.680	0.550	0.466	0.338	0.251	0.163	0.089
	621	0.665	0.539	0.455	0.330	0.250	0.165	0.091
	618	0.659	0.532	0.451	0.330	0.248	0.163	0.092
	902	0.939	0.776	0.661	0.493	0.378	0.250	0.134
	1181	1.210	0.994	0.824	0.646	0.499	0.330	0.175
153.900	624	0.771	0.564	0.469	0.343	0.254	0.167	0.093
	620	0.751	0.546	0.458	0.338	0.252	0.168	0.095
	619	0.746	0.547	0.458	0.337	0.252	0.169	0.095
	904	1.071	0.796	0.673	0.501	0.384	0.256	0.140
	1179	1.397	1.027	0.870	0.656	0.507	0.341	0.187
154.000	604	0.832	0.574	0.461	0.327	0.239	0.156	0.085
	602	0.813	0.563	0.454	0.324	0.238	0.156	0.085
	600	0.806	0.560	0.451	0.322	0.238	0.156	0.084
	891	1.137	0.817	0.667	0.485	0.362	0.237	0.129
	1171	1.463	1.056	0.870	0.636	0.480	0.315	0.170

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
154.100	613	0.611	0.491	0.421	0.329	0.252	0.168	0.088
	617	0.604	0.486	0.419	0.328	0.253	0.169	0.090
	612	0.601	0.483	0.416	0.325	0.252	0.168	0.092
	896	0.870	0.712	0.615	0.487	0.378	0.253	0.132
	1177	1.134	0.932	0.807	0.641	0.501	0.336	0.174
154.200	625	0.691	0.615	0.544	0.439	0.345	0.219	0.101
	620	0.678	0.602	0.533	0.431	0.339	0.216	0.101
	617	0.675	0.599	0.531	0.430	0.340	0.218	0.100
	902	0.929	0.830	0.735	0.602	0.479	0.314	0.151
	1178	1.192	1.045	0.927	0.763	0.613	0.410	0.200
154.300	621	0.932	0.633	0.479	0.315	0.225	0.140	0.083
	618	0.887	0.610	0.464	0.312	0.226	0.143	0.082
	617	0.876	0.609	0.459	0.311	0.226	0.143	0.081
	898	1.251	0.893	0.682	0.474	0.342	0.217	0.117
	1170	1.609	1.158	0.900	0.642	0.463	0.290	0.156
154.401	604	0.984	0.603	0.437	0.299	0.221	0.144	0.081
	604	0.938	0.596	0.433	0.306	0.227	0.149	0.082
	603	0.924	0.594	0.431	0.307	0.227	0.148	0.080
	889	1.311	0.884	0.652	0.469	0.349	0.229	0.124
	1160	1.662	1.153	0.855	0.627	0.467	0.305	0.167
154.500	616	0.938	0.709	0.510	0.315	0.217	0.134	0.082
	612	0.901	0.690	0.504	0.317	0.213	0.131	0.084
	613	0.896	0.691	0.508	0.319	0.218	0.136	0.083
	901	1.277	0.999	0.750	0.484	0.333	0.205	0.121
	1177	1.633	1.284	0.988	0.646	0.450	0.277	0.155
154.600	614	0.964	0.645	0.479	0.329	0.238	0.157	0.089
	612	0.924	0.634	0.473	0.329	0.240	0.159	0.086
	611	0.915	0.633	0.473	0.329	0.241	0.159	0.089
	895	1.293	0.915	0.700	0.497	0.365	0.239	0.130
	1165	1.646	1.181	0.916	0.657	0.527	0.317	0.169
154.700	603	1.061	0.695	0.497	0.329	0.236	0.157	0.089
	597	1.005	0.671	0.487	0.331	0.240	0.161	0.090
	596	0.995	0.668	0.485	0.333	0.242	0.162	0.089
	886	1.389	0.972	0.725	0.508	0.369	0.245	0.134
	1166	1.771	1.250	0.948	0.673	0.492	0.327	0.173
154.800	627	0.720	0.480	0.356	0.251	0.189	0.133	0.082
	623	0.700	0.469	0.351	0.250	0.189	0.135	0.083
	620	0.694	0.466	0.351	0.250	0.189	0.136	0.083
	910	0.984	0.686	0.528	0.387	0.294	0.207	0.126
	1186	1.286	0.894	0.695	0.515	0.394	0.274	0.161
154.900	611	1.050	0.730	0.536	0.354	0.254	0.167	0.096
	606	1.002	0.705	0.524	0.351	0.254	0.168	0.096
	603	0.991	0.699	0.521	0.351	0.253	0.168	0.097
	896	1.403	1.017	0.774	0.531	0.390	0.257	0.147
	1166	1.800	1.299	1.003	0.697	0.520	0.341	0.194
155.000	613	0.860	0.542	0.392	0.274	0.207	0.145	0.089
	610	0.829	0.528	0.384	0.274	0.206	0.146	0.091
	607	0.821	0.524	0.383	0.272	0.204	0.145	0.091
	897	1.160	0.767	0.571	0.412	0.316	0.221	0.132
	1173	1.515	1.001	0.754	0.548	0.425	0.297	0.174
155.101	597	1.240	0.823	0.610	0.405	0.293	0.187	0.101
	594	1.178	0.790	0.600	0.403	0.294	0.186	0.103
	589	1.162	0.784	0.594	0.404	0.299	0.193	0.106
	876	1.635	1.152	0.883	0.616	0.463	0.292	0.160
	1146	2.106	1.487	1.154	0.818	0.622	0.393	0.215

Table F.2(l) - Site 6 - Chainage 153.600 - 155.100 – Right Side

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
153.600	604	0.528	0.415	0.357	0.285	0.222	0.147	0.080
	598	0.517	0.406	0.350	0.281	0.220	0.144	0.080
	598	0.517	0.406	0.350	0.280	0.219	0.146	0.080
	879	0.764	0.622	0.541	0.432	0.341	0.225	0.124
	1153	1.029	0.825	0.717	0.575	0.457	0.303	0.163
153.700	598	0.773	0.589	0.487	0.341	0.243	0.149	0.086
	596	0.756	0.579	0.480	0.340	0.243	0.154	0.085
	598	0.752	0.575	0.478	0.340	0.243	0.155	0.089
	882	1.075	0.839	0.706	0.509	0.368	0.237	0.131
	1152	1.395	1.092	0.924	0.673	0.490	0.311	0.172
153.800	605	0.647	0.440	0.359	0.267	0.203	0.136	0.083
	599	0.628	0.430	0.353	0.260	0.201	0.139	0.082
	596	0.623	0.428	0.349	0.262	0.201	0.137	0.083
	884	0.891	0.639	0.527	0.398	0.307	0.210	0.124
	1160	1.171	0.840	0.697	0.530	0.410	0.280	0.163
153.900	593	0.816	0.609	0.500	0.349	0.273	0.175	0.087
	588	0.793	0.593	0.490	0.346	0.268	0.174	0.089
	589	0.788	0.593	0.490	0.346	0.269	0.176	0.087
	872	1.132	0.881	0.736	0.531	0.416	0.271	0.131
	1143	1.487	1.159	0.974	0.711	0.568	0.365	0.177
153.992	593	0.889	0.567	0.425	0.308	0.222	0.146	0.093
	593	0.862	0.556	0.420	0.305	0.224	0.147	0.092
	589	0.852	0.550	0.417	0.303	0.224	0.147	0.092
	879	1.200	0.809	0.627	0.460	0.344	0.227	0.137
	1156	1.528	1.049	0.823	0.608	0.464	0.306	0.180
154.100	607	0.454	0.406	0.358	0.283	0.221	0.147	0.085
	601	0.449	0.397	0.355	0.279	0.219	0.145	0.080
	597	0.446	0.395	0.352	0.278	0.218	0.145	0.084
	882	0.667	0.599	0.528	0.421	0.332	0.223	0.127
	1158	0.909	0.796	0.702	0.560	0.444	0.299	0.168
154.200	600	0.517	0.412	0.370	0.296	0.231	0.152	0.086
	598	0.509	0.406	0.364	0.292	0.229	0.152	0.087
	596	0.506	0.404	0.363	0.291	0.228	0.152	0.086
	876	0.740	0.604	0.542	0.439	0.349	0.232	0.128
	1149	0.981	0.798	0.720	0.580	0.460	0.309	0.171
154.300	598	0.794	0.479	0.374	0.279	0.220	0.149	0.080
	595	0.764	0.467	0.367	0.277	0.217	0.149	0.084
	591	0.757	0.463	0.366	0.275	0.217	0.148	0.082
	877	1.055	0.695	0.561	0.426	0.339	0.230	0.123
	1145	1.362	0.913	0.746	0.569	0.457	0.309	0.161
154.400	607	0.993	0.656	0.489	0.312	0.218	0.133	0.083
	602	0.957	0.645	0.472	0.308	0.218	0.136	0.081
	601	0.949	0.648	0.469	0.305	0.217	0.139	0.079
	889	1.338	0.936	0.694	0.464	0.332	0.211	0.119
	1165	1.717	1.201	0.907	0.616	0.444	0.283	0.154
154.499	595	1.033	0.677	0.522	0.351	0.257	0.167	0.096
	589	0.990	0.655	0.504	0.348	0.256	0.168	0.093
	587	0.980	0.654	0.499	0.349	0.256	0.168	0.098
	883	1.389	0.972	0.751	0.536	0.397	0.261	0.144
	1156	1.772	1.264	0.990	0.714	0.535	0.352	0.193
154.599	614	0.689	0.541	0.448	0.317	0.220	0.142	0.083
	610	0.680	0.534	0.441	0.313	0.220	0.142	0.080
	607	0.675	0.531	0.437	0.312	0.220	0.143	0.081
	890	0.956	0.766	0.634	0.468	0.341	0.221	0.123
	1165	1.248	0.987	0.806	0.616	0.456	0.296	0.163
154.700	604	0.764	0.558	0.466	0.341	0.246	0.165	0.087
	609	0.750	0.551	0.461	0.343	0.248	0.165	0.091
	608	0.746	0.547	0.459	0.343	0.248	0.167	0.092

Chain (km)	Press (kPa)	Deflections (mm)						
		0	200	300	450	600	900	1500
	889	1.057	0.791	0.674	0.512	0.372	0.247	0.142
	1160	1.362	1.022	0.875	0.671	0.494	0.330	0.190
154.799	611	0.446	0.342	0.295	0.231	0.181	0.128	0.075
	608	0.439	0.339	0.290	0.228	0.180	0.128	0.073
	608	0.438	0.339	0.290	0.228	0.180	0.129	0.073
	894	0.667	0.518	0.448	0.353	0.281	0.199	0.109
	1173	0.903	0.695	0.599	0.473	0.378	0.267	0.146
154.900	609	0.892	0.654	0.508	0.345	0.251	0.162	0.086
	602	0.865	0.636	0.499	0.342	0.250	0.162	0.086
	602	0.857	0.630	0.495	0.341	0.250	0.163	0.088
	892	1.213	0.909	0.732	0.526	0.386	0.251	0.135
	1170	1.564	1.172	0.926	0.695	0.517	0.337	0.183
155.000	599	0.835	0.602	0.497	0.367	0.264	0.165	0.096
	597	0.818	0.592	0.492	0.364	0.265	0.169	0.095
	597	0.814	0.591	0.490	0.364	0.264	0.167	0.096
	883	1.152	0.864	0.726	0.549	0.405	0.258	0.146
	1157	1.475	1.116	0.942	0.725	0.537	0.341	0.195
155.100	604	1.039	0.741	0.593	0.437	0.314	0.199	0.099
	599	1.003	0.722	0.581	0.433	0.310	0.199	0.100
	599	0.993	0.718	0.580	0.433	0.310	0.200	0.102
	881	1.389	1.035	0.849	0.646	0.472	0.303	0.151
	1150	1.769	1.340	1.110	0.855	0.631	0.403	0.204

APPENDIX G – FWD COMPARISON TEST RESULTS

Table G.1 lists the calculated moduli for the selected test locations at the six sites as listed in Table E.4 - Acceptance Test Data – Selected Sites (Appendix E), together with the Relative Dry Density test results for each location as measured at the time of construction. The FWD Subgrade Modulus in MPa (Column 4) was obtained from the pulse time analysis during the Falling Weight Deflectometer tests. The moduli for the Subgrade, Subbase and Base (the Stabilised top layer) listed were calculated using the CIRCDEF program to match the deflection bowl measured by the Falling Weight Deflectometer tests – refer Appendix F.

Table G.1 - FWD Comparison Test Data – Selected Sites

Site 1

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**40**	**40**	**69**	**1000**
1.1.2	153221	93.0	130	111	107	1220
1.1.3	153537	96.9	80	96	24	1150
1.1.4	153602	100.4	130	133	52	1620
1.1.5	153821	95.1	250	200	260	10400
1.1.6	153946	98.9	50	74	20	8180
1.1.7	154186	97.4	60	82	26	750
1.1.8	154441	96.4	50	80	25	1270
1.2.2	153313	95.3	180	145	225	1290
1.2.3	153462	98.2	110	77	24	1876
1.2.4	153588	91.6	40	50	44	1115
1.2.5	153755	99.0	80	88	29	2496
1.2.6	153871	96.6	80	92	33	1022
1.2.7	154115	93.2	100	94	70	640
1.2.8	154204	95.2	90	90	73	534
1.2.9	154365	97.1	90	72	21	2410

Site 2

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**50**	**50**	**650**	**1000**
2.1.1	66837	101.8	250	154	8217	20000
2.1.2	66913	100.0	250	286	5744	1468
2.1.3	66980	102.8	250	202	10000	78
2.1.4	67171	100.4	250	150	6000	20000
2.2.1	66826	93.1	250	264	220	8871
2.2.2	67028	101.4	250	279	1970	15000
2.2.3	67045	100.7	250	243	6000	15000
2.2.4	67254	99.3	250	212	3654	15000

Site 3

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**30**	**30**	**-**	**2000**
3.1.1	2334	100.8	130	110		4212
3.1.2	2596	98.0	160	151		1122
3.1.3	2660	102.7	160	133		5096
3.1.4	2913	99.5	140	117		1879
3.2.1	3115	100.3	190	149		3864
3.2.2	3314	98.5	170	138		2636
3.2.3	3495	99.2	140	120		8351
3.2.4	3665	93.5	100	80		1182
3.3.1	2296	102.1	140	115		5357
3.3.2	2424	97.5	140	122		3665
3.3.3	2723	98.1	200	154		6040
3.3.4	2904	97.9	250	187		5225
3.4.1	3122	96.9	200	144		2121
3.4.2	3351	97.9	250	244		3071
3.4.3	3544	100.7	210	154		7900
3.4.4	3718	93.0	130	105		2910

Site 4

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**71**	**71**	**71**	**600**
4.1.1	28579	95.2	140	165	23	4600
4.1.2	28929	96.6	250	340	25	5015
4.1.3	29035	96.1	250	490	34	937
4.2.1	28643	94.0	50	60	26	688
4.2.2	28726	95.5	60	68	15	786
4.2.3	29062	98.4	250	228	16	626
4.3.1	29255	98.7	100	93	107	2689
4.3.2	29313	101.9	180	136	64	735
4.3.3	29486	98.6	240	171	74	1825
4.3.4	29521	99.4	220	154	53	1184

Site 5

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**50**	**50**	**-**	**1000**
5.1.1	4981	103.7	160	132		23854
5.1.2	5060	100.7	200	155		11167
5.1.3	5321	100.2	160	140		8253
5.2.1	5359	97.3	240	176		15704
5.2.2	5645	102.6	160	132		14525
5.2.3	5921	105.9	190	124		23291
5.3.1	6004	97.6	140	108		13578
5.3.2	6424	99.2	160	178		7953
5.3.3	6498	100.3	250	160		20824
5.4.1	6765	99.8	190	147		7286
5.4.2	6965	99.7	140	123		11234
5.4.3	7255	99.9	250	211		21701
5.5.1	5001	103.6	160	104		19963
5.5.2	5061	101.5	130	117		4723
5.5.3	5243	98.5	230	144		11192
5.6.1	5454	99.8	150	132		13169
5.6.2	5630	99.7	250	149		1136
5.6.3	5813	98.6	250	215		17984
5.7.1	6014	100.2	250	260		3050
5.7.2	6332	97.1	250	293		16028
5.7.3	6641	99.7	180	141		6307
5.8.1	6770	98.8	250	238		3721
5.8.2	7031	99.5	250	182		1673
5.8.3	7242	99.2	240	146		1180

Site 6

Test ID	Chain m	RDD %	FWD Subgrade Modulus	Modulus MPa		
				Subgrade	Subbase	Base
Design	**-**	**-**	**50**	**50**	**186**	**600**
6.1.1	153632	95.3	70	89	23	1518
6.1.2	153828	104.1	100	95	87	957
6.1.3	154170	96.4	120	90	22	2822
6.1.4	154296	103.1	90	103	43	1233
6.1.5	154646	96.1	130	93	69	553
6.1.6	154910	93.4	80	80	63	360
6.2.1	153649	101.2	120	114	76	1578
6.2.2	153784	101.4	100	99	136	738
6.2.3	154227	96.2	120	115	121	2598
6.2.4	154291	98.5	140	95	220	473
6.2.5	154607	100.1	110	104	70	1326
6.2.6	154918	104.1	70	81	60	1076