MATERIALS & METHODS NEEDED TO PREPARE SUBGRADES SUITABLE FOR *CAPTIF*

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AN IMPORTANT NOTE FOR THE READER

The research detailed in this report was commissioned by Transfund New Zealand.

Transfund New Zealand is a Crown entity established under the Transit New Zealand Act 1989. Its principal objective is to allocate resources to achieve a safe and efficient roading system. Each year, Transfund New Zealand invests a portion of its funds on research that contributes to this objective.

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Introduction

The Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) is a state-of-the-art pavement testing facility located at Christchurch, New Zealand. Pavements are constructed in it, using materials and thicknesses similar to actual roads, for testing. This report presents the results of the third stage, carried out in 1998, of a three stage project to determine the materials and methods necessary to prepare appropriate subgrades of a desired strength to be used at CAPTIF.

For CAPTIF to fully perform its function as a pavement engineering research and development tool, it is important that pavements can be tested at CAPTIF under a variety of conditions. One of the most fundamental variables is the strength or elastic modulus of the pavement subgrade. Experience has shown that a good quality subgrade is relatively easy to construct at CAPTIF, e.g. a subgrade with a California Bearing Ratio (CBR) of say 15% or higher. However it is a much greater challenge to construct a subgrade with low or moderate CBR.

Method

This project investigated materials and methods needed to construct subgrades at CAPTIF that have a pre-determined subgrade strength. It involved the construction of a trial subgrade based on the recommendations of previous stages of the project. These recommendations primarily involved controlling the water content of the subgrade to achieve different subgrade strengths. A trial pavement was constructed at CAPTIF and subjected to 147,000 load repetitions. The properties of each pavement layer were carefully monitored during construction, at intervals during the loading, and at the completion of the loading sequence.

This generated a large quantity of data in the monitoring of the construction and performance of the trial subgrade. In some cases several test procedures were used to monitor similar material parameters. Irrespective of these factors, most of the test data provided a consistent and credible characterisation of the pavement materials.

Results

The results of the monitoring programme showed that the Tod Clay soil (a silty clay from Hyde in North Otago) used in the test subgrade performed well, and its water content and dry density did not change significantly from the start to the finish of the loading sequence. They also showed that controlling the water content is a legitimate way to control the strength and stiffness of a test subgrade, but improvements to the construction technique are required.

Conclusions

The main conclusions that can be drawn for the project are as follows:

- Tod Clay soil is suitable for constructing test pavement subgrades at CAPTIF. It is workable over a reasonable range of water contents and did not suffer significant changes in water content or dry density after the application of a large number of repeated loads.
- Using the soil water content to control the strength/stiffness properties of the subgrade is an appropriate technique.

However, the method of achieving the desired water content requires improvement as the sprinkler approach used in this project produced some variability.

- The Loadman portable falling weight deflectometer (FWD) is a valuable tool for characterising pavement materials, requiring minimum of effort both during the testing and in postmortem analyses.

Recommendations
The experience gained during the construction, loading and monitoring of the test pavement has resulted in the identification of areas where the subgrade construction at CAPTIF can be improved, i.e:

- The manner in which the soil is conditioned needs improvement. The sprinkler system used during this trial did not provide the uniformity required. A pug mill or similar equipment should achieve a much more consistent water content and avoid the soft spots caused by the concentration of water around the sprinkler heads.

- The construction of a covered storage area should be an objective to strive for at CAPTIF. A covered storage facility would allow the equilibration of soil water content during CAPTIF's down-time and would protect the soil stockpile from erosion and loss. A covered facility would also provide a buffer area for soil that has already been conditioned in the pug mill, to be stockpiled before it is placed in the test pavement.

- A self-propelled trench roller is appropriate for the compaction of the CAPTIF subgrade soil. While alternative plant should not be excluded from consideration, the trench roller has proved to be effective in this project.

- The Loadman apparatus is recommended for future use in CAPTIF projects.

These issues have been addressed in a revised specification for the construction of subgrades at CAPTIF.

ABSTRACT

The Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) is a state-of-the-art pavement testing facility located at Christchurch, New Zealand. This report presents the results of the third stage, carried out in 1998, of a three stage project to determine the materials and methods necessary to prepare appropriate subgrades of a desired strength to be used at CAPTIF.

A subgrade construction specification was developed using the recommendations of reports from previous stages of the project. This primarily involved controlling the water content of the subgrade to achieve different subgrade strengths. A trial pavement was constructed at CAPTIF and subjected to 147,000 load repetitions. The properties of each pavement layer were carefully monitored during construction, at intervals during the loading and at the completion of the loading sequence.

1. INTRODUCTION

1.1 General

The Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) is a state-of-the-art pavement testing facility located at Christchurch, New Zealand. It is owned and managed by Transit New Zealand with the University of Canterbury providing the technical expertise for its operation and research.

This report presents the results of the third stage, carried out in 1998, of a three stage project to determine the materials and methods necessary to prepare appropriate subgrades of a desired strength to be used at CAPTIF.

1.2 Objective

For CAPTIF to fully perform its function as a pavement engineering research and development tool, it is important that pavements can be tested at CAPTIF under a variety of conditions. One of the most fundamental variables is the strength or elastic modulus of the pavement subgrade. Experience has shown that a good quality subgrade is relatively easy to construct at CAPTIF, e.g. a subgrade with a California Bearing Ratio (CBR) of say 15% or higher. However it is a much greater challenge to construct a subgrade with low or moderate CBR.

The objective of this project was to investigate materials and methods necessary to construct subgrades at CAPTIF that have a pre-determined subgrade strength. Also important is that the performance of the subgrade is reasonably consistent throughout the duration of a test. For the purposes of this project, three subgrade strengths have been defined, i.e. weak (CBR < 5%), moderate (5% < CBR < 10%), and strong (CBR > 10%). Note that the CBR parameter has been used to characterise the subgrade, but a direct correlation with the more fundamental parameter of subgrade elastic modulus, ($E_{vertical}$), i.e. $E_{vertical}$ = 10CBR (AUSTROADS 1992), is generally accepted for most cohesive subgrade soils in New Zealand. This relationship is based on the soil responding anistropically, where the ratio of the vertical to horizontal elastic moduli is 2.0.

The objective of this (third and final) stage of the project was to take the recommendations of the previous stage and construct a test subgrade at CAPTIF. The performance of the subgrade was monitored to ensure that the pre-defined properties were achieved and that they persisted through an extended loading sequence.

1.3 Previous Work

The first stage of this project comprised a general review of the literature on the performance of subgrade soils. The second stage of the project involved the selection of a number of candidate CAPTIF subgrade soils (Bartley Consultants Ltd 1995).

Each soil was thoroughly investigated including the performance of repeated load triaxial tests to determine their response to dynamic loading.

One approach to constructing subgrades in a weak, moderate or strong condition would be to use varying degrees of construction quality. However, this approach was rejected because it is inappropriate for the performance of a test pavement to be influenced by construction procedures. It is much more desirable for the fundamental response of the materials to be the controlling influence.

The initial testing carried out in the second stage of the project demonstrated that virtually all soils exhibit a high degree of strength and stiffness when thoroughly compacted at their optimum water content. It was postulated that the best way to control the strength of a CAPTIF subgrade would be to vary the soil water content while still compacting the soil as far as possible towards saturation. Minimising the air voids gives the best chance of achieving consistent performance under repeated loading.

Choosing the candidate soils for further testing included a number of considerations. The most important of these were as follows:

- The soils should be located as close as possible to CAPTIF to minimise transportation costs.
- The soils should exhibit a gradual change in strength or stiffness with changes in water content. This means that any small errors in achieving the desired water contents do not have significant effect on the resulting subgrade strength or stiffness.
- The soil should be workable over a wide range of water contents.

The first of the considerations listed above meant that soils originating from the South Island of New Zealand, and in particular those close to the Canterbury area, were given priority.

The second consideration was achieved by taking cognizance of the definition of the soil Plasticity Index (PI) parameter. The PI is defined as the range of water contents over which a soil maintains a plastic state. Typically this also corresponds to the change in water content that is required to achieve an approximate 100 to 200-fold change in undrained shear strength of the soil. Therefore, the higher the PI, the more likely it is that the soil will demonstrate a gradual change in strength with variations in water content.

The third consideration was somewhat related to the second. While a high PI was desirable in one sense, it can also be associated with difficulty in handling. Therefore, a soil with a moderate to high PI was considered to be best suited for the CAPTIF subgrade.

The second stage report concluded that a silty clay from Hyde in North Otago gave the best performance in terms of the considerations listed above. This material has been named *Tod Clay* after the owner of the pit from which it was excavated.

The soil has a workable consistency related to the mica content and has a relatively low susceptibility to shrinkage and swelling because of the predominant kaolin clay mineral. A summary of the engineering characteristics of the Tod Clay samples tested in the laboratory is presented in Appendix A of this report.

1.4 Brief Description of CAPTIF

CAPTIF consists of an annular concrete trough (median diameter 18.5 m) with a trapezoidal cross-section (Figure 2.1). The trough is 4 m wide at the top, 1.5 m wide at the base and 1.5 m deep. The test pavement is constructed in the trough and is subsequently loaded using the Simulated Loading and Vehicle Emulator (SLAVE).

The test track component of CAPTIF is housed in an octagonal enclosure which protects it from environmental factors. The enclosure also allows electronic measuring equipment and instrumentation to be used without detrimental influences from the environment. The facility includes a stockpile area for construction materials which is not enclosed.

The SLAVE comprises two bogies fitted to a radial arm that rotates to produce bogey speeds of up to approximately 50 km/h. The bogies can be fitted with a variety of suspension types and loads. In addition, the bogies can be set so they traffic different portions of the test pavement as well as wandering either side of a pre-set centre line.

A variety of pavement structures have been investigated using CAPTIF. These have included unbound aggregate bases, stabilised materials and asphalt. Typically, a test monitoring programme may include strain measurements within the subgrade and/or base and sub-base layers, surface deflection, transverse profiles, longitudinal profiles, crack lengths and/or widths, layer water contents and densities, etc. Tests are generally performed before, during, and after testing so that a full history of pavement performance can be developed.

1.5 CAPTIF Instrumentation and Testing Capabilities

CAPTIF provides the opportunity to subject a variety of pavement types to large numbers of axle loads in a manner that accurately simulates real pavement structures and loadings. However, this is only one aspect of the facility. To accurately monitor pavement performance, CAPTIF staff have developed sophisticated instrumentation and data acquisition systems. There is also the capability to perform fundamental pavement materials testing by utilising the resources of the Department of Civil Engineering at the University of Canterbury.

1.5.1 Fundamental Soil / Aggregate Properties

CAPTIF and the University of Canterbury have the capability to undertake a wide range of fundamental soil and aggregate tests, for example:

- density by sand replacement and nuclear methods;
- water content;

- particle size distribution;
- CBR - laboratory or in situ;
- triaxial compression test - static or repeated loading;
- dynamic cone penetration; and
- vane shear strength.

1.5.2 Loading

Dynamic loads are measured by a system developed by Industrial Research Ltd, New Zealand, and adapted for use at CAPTIF. The SLAVE vehicles are fitted with accelerometers on the axle and chassis that, together with a distance gauge between the two assemblies, measure the vertical movement of the vehicle components as they travel around the track circumference. Dynamic loads are calculated by relating the mass of the components to their vertical accelerations. The data are read remotely while the vehicles are travelling at speeds up to 50km/h.

1.5.3 Pavement Profiles

Longitudinal profiles can be measured at any transverse position within 800 mm of the track centreline. A laser camera is mounted behind a SLAVE vehicle which is measuring pavement height, and a rotorpulser attached to the wheel hub measures longitudinal distance. An accelerometer inside the laser camera box compensates for vehicle suspension movement. The data are read remotely at a vehicle speed of 25 km/h.

Transverse profiles are measured by the CAPTIF Profilometer, a beam that straddles the tank walls with an electric-powered carriage that measures pavement heights at 25 mm intervals. Data are logged to a hand-held computer and downloaded to the main data acquisition system.

1.5.4 Pavement Deflections

Surface deflections are measured by the CAPTIF Deflectometer, a device based on the principle of the Benkelman Beam but with fewer moving parts. It has an electronic data capture link to a hand-held computer. Surface deflections are taken automatically every 50 mm as the SLAVE vehicle moves away from the sensor.

A Loadman portable falling weight deflectometer is also available. The Loadman comprises an aluminium tube with a steel base plate bolted to the lower end. A 10 kg mass is contained within the tube and is free to slide up and down. To perform a test the Loadman is placed on the test layer so that the base plate is in full contact with the surface of the layer. When the device is activated the weight falls within the tube from a height of 800 mm and impacts the base plate. An accelerometer records the resulting motion of the Loadman and the deflection of the test layer is calculated. The Loadman gives an instant readout of inferred layer elastic modulus using the Boussinesq theory for the response of a single layer of elastic material (Bartley Consultants Limited 1998). Alternatively, the Loadman deflection can be used to back-calculate the elastic modulus using a separate analysis procedure.

2. CAPTIF TEST PARAMETERS

2.1 General

Construction of the CAPTIF subgrade was divided into three segments, as described in Table 2.1.

Table 2.1 Subgrade conditions for the CAPTIF test pavement.

Segment	Station	Subgrade Condition
A	00 - 20	Tod clay with target water content of 30%
B	20 - 40	Tod clay placed at stockpile water content (approx. 23%)
C	40 - 57.8	Tod clay with a target water content equal to the higher of the stockpile water content or 25% (this was later amended to simply the stockpile water content (see Section 3.1))

Only Segments A and C were tested in detail. Segment B included the access ramps for construction equipment and therefore was subject to inconsistent compaction.

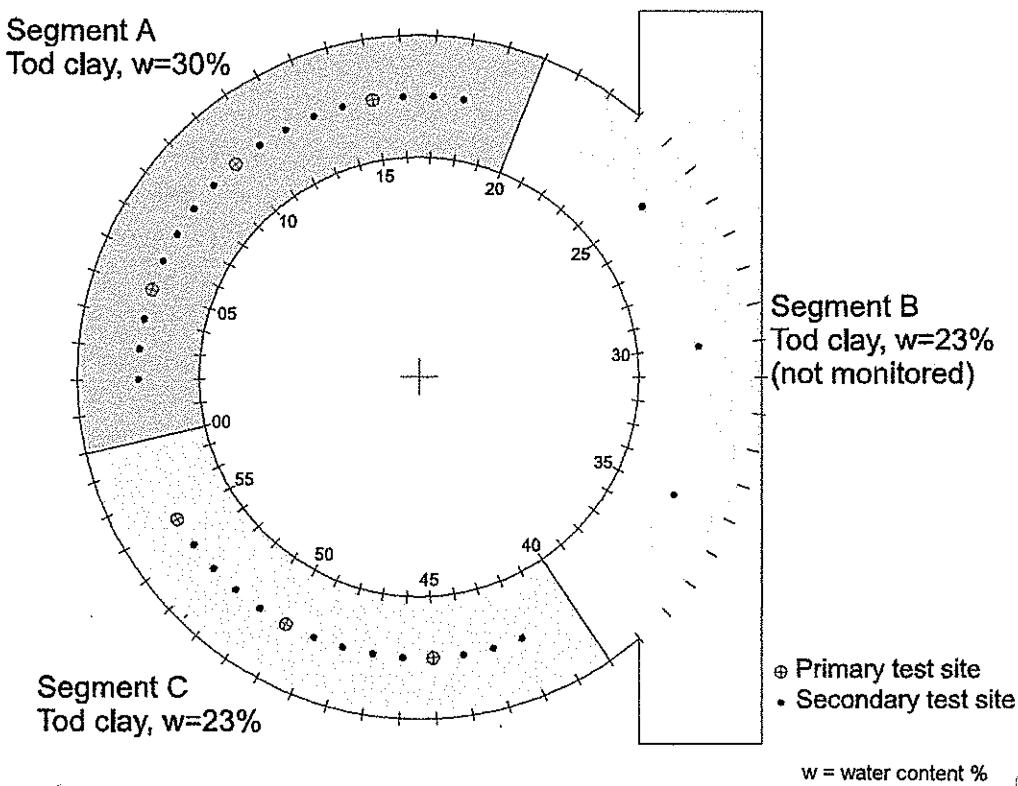


Figure 2.1 Plan view of the CAPTIF test subgrade layout.

2.2 Pavement Structure

A specification for construction of the trial pavement was prepared by Bartley Consultants Limited. Some modifications to the original specification were necessary to suit existing conditions and test pavement design requirements, as well as time and equipment constraints. The following describes the test procedures that were adopted.

At the time of construction the lower part of the CAPTIF trough contained approximately 560 mm of clay soil sourced from Waikari, Canterbury. This material was retained in the trough as a lower subgrade. The overlying thickness of Tod Clay was approximately 590 mm. The upper pavement structure comprised about 165 mm of AP40 sub-base aggregate overlain by 160 mm of AP20 basecourse aggregate and about 30 mm of Mix 7 asphalt. This structure is summarised in Figure 2.2, which also shows that the pavement layers were constructed in 8 lifts.

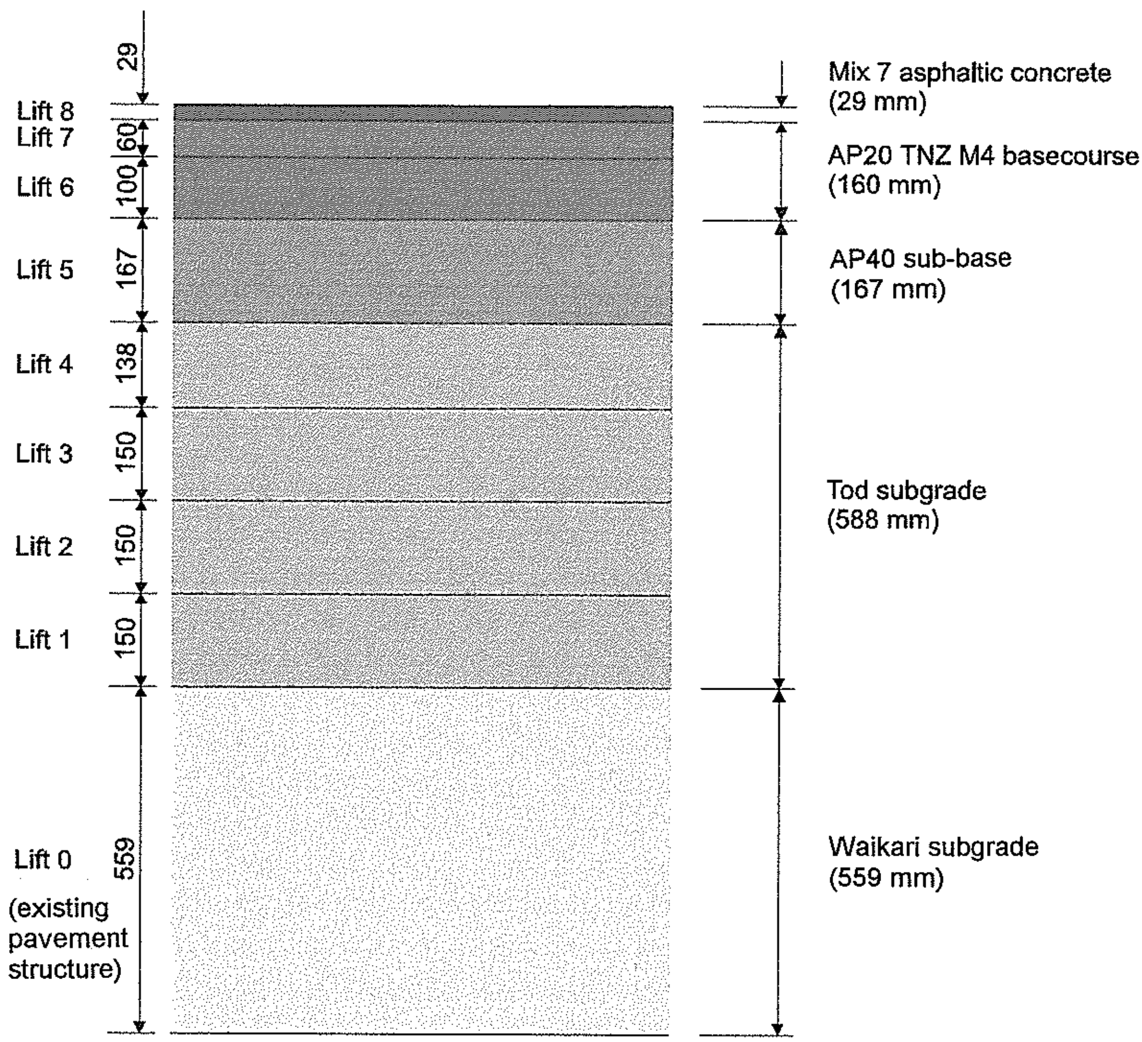


Figure 2.2 Test pavement structure for CAPTIF.

2.3 Loading

The testing procedure included the following loading specifications:
- both SLAVE vehicles laden to 40 kN gross weight;
- both SLAVE vehicles fitted with 10R20 tyres inflated to 700 kPa (measured cold);
- SLAVE vehicle A fitted with a steel multi-leaf suspension unit;
- SLAVE vehicle B fitted with an air suspension unit;
- SLAVE vehicle speed set to 45 km/h;
- SLAVE vehicle centreline set at lateral offset of 175 cm; and
- SLAVE vehicle wander set to +/- 200 mm with a normal distribution.

Loading was paused after 5,000, 15,000, 25,000, 75,000, and 100,000 cycles to allow pavement tests to be carried out. Loading was discontinued after 147,000 cycles.

2.4 Testing Schedule

A suite of tests was performed to achieve the following objectives:
- monitor the construction of the test pavement;
- monitor the performance of the pavement during loading; and
- determine the terminal condition of the pavement (postmortem).

As described in Section 2.1, the CAPTIF trough was divided into three subgrade segments, and segments A and C were of most interest. These segments were further divided into *primary* test sites and *secondary* test sites. In all there were six primary test sites and 28 secondary test sites located as shown in Figure 2.1.

2.4.1 Construction Testing

A number of tests were carried out on the top of the subgrade during construction to establish the initial condition of the soil. Table 2.2 describes the tests that were carried out at the respective test sites.

Table 2.2 Summary of subgrade testing schedule.

Subgrade Construction Tests Specified	
Primary Test Sites	**Secondary Test Sites**
Loadman tests at C/L[(1)], +/- 0.5 m & +/- 1.0 m Water content tests at C/L Vane shear tests at C/L & +/- 0.5 m In situ CBR at C/L Dynamic cone penetrometer tests at C/L NDM[(2)] tests at C/L, +/- 0.5 m & +/- 1.0 m 38 mm diameter core sample recovery	Loadman tests at C/L NDM[(2)] tests at C/L

(1) C/L - centreline of loading
(2) NDM - Troxler nuclear density meter

A number of tests were carried out on the top of the basecourse layer to establish the initial condition of the top part of the pavement. Table 2.3 describes the tests that were carried out at the respective test sites.

Table 2.3 Summary of basecourse testing schedule.

Basecourse Construction Tests Specified	
Primary Test Sites	**Secondary Test Sites**
Loadman tests at C/L, +/- 0.5 m & +/- 1.0 m NDM tests at C/L, +/- 0.5 m & +/- 1.0 m Sand replacement tests at +/- 1.0 from C/L Water content tests at +/- 1.0 from C/L	Loadman tests at C/L NDM tests at C/L

At the completion of pavement construction, transverse profiles were recorded at each station. Longitudinal profiles were recorded at the centreline and offsets of +/- 40 cm and +/- 80 cm.

2.4.2 Intermediate Testing

At "milestones" of 5,000, 15,000, 25,000, 50,000, 75,000 and 100,000 load cycles a limited number of tests were carried out. These involved measurement of:

- dynamic wheel forces at the pavement centreline;
- transverse profile at each station; and
- longitudinal profiles at the centreline and offsets of +/- 40 cm and +/- 80 cm.

2.4.3 Postmortem Testing

At the end of the loading sequence, trenches were excavated across the test pavement at Stations 4, 12 and 15 in Segment A and Stations 47, 50 and 55 in Segment C. The postmortem tests carried out at each trench were those described in Table 2.4.

Table 2.4 Summary of postmortem testing schedule.

Pavement Component	Test	Location(s)
Asphalt Surface	Manual transverse profile	n/a
Basecourse Surface	Loadman NDM density Sand replacement density Water content Manual transverse profile	C/L, +/- 0.5 m & +/- 1.0 m C/L, +/- 0.5 m & +/- 1.0 m C/L Sand replacement test locations n/a
Subgrade Surface	Loadman Vane shear strength In situ CBR Dynamic cone penetrometer NDM density Sand replacement density Water content 30 mm diameter core Manual transverse profile	C/L, +/- 0.5 m & +/- 1.0 m C/L & +/- 0.5 m C/L C/L C/L, +/- 0.5 m & +/- 1.0 m C/L C/L C/L n/a

3. CAPTIF TEST PAVEMENT CONSTRUCTION

3.1 Subgrade

Three hundred tonnes of Tod Clay were transported from Hyde and stored for some months uncovered, at Isaacs Construction Company's quarry in Christchurch. Just before construction, the clay was transported to the CAPTIF site and stockpiled on a concrete pad adjacent to the test track. As the stockpile area was uncovered, care was taken during stockpiling to mix the material as much as possible to even out the water content. The water content was then measured at several points using digital scales and a microwave oven.

A pug mill could not be hired locally and the unknown workability of the material suggested that modifying the water content of the clay would be more easily achieved during construction than in the stockpile. A system of irrigation sprinklers and flow meters was developed to wet each lift of the material in the test pavement before compaction.

Lift 0 The existing layer of Waikari clay soil left from the previous test pavement was ripped to a depth of 150 mm by a bulldozer equipped with ripper blades. The bulldozer then levelled the surface and it was rolled with a 2.5 t combination steel drum/rubber tyre roller. A light sprinkling of water was added during the rolling. Spot levels and Loadman readings were taken on the centreline at the 10 m stations.

Lift 1 The first lift was 200 mm deep before compaction, in order to avoid contamination with the underlying Waikari soil while the water content of the Tod Clay was being modified. The lift was rotary hoed with a tractor-mounted hoe, to break up the larger lumps, and the water content was measured at 5 m stations in Segments A and C. The water content was found to be quite consistent at 23-24%. The volume of material in each segment, and the amount of water required to bring it up to the target water content, were calculated. A domestic irrigation system using 5 mini-sprinklers on a 20 m length of tubing with a flow meter at the tap was used to apply the water. The system performed adequately but there was some difficulty getting a uniform distribution of water, with the soil near the sprinkler heads getting more water than the remaining areas. Also, care had to be taken to ensure that the sprinkler nozzles did not become blocked with soil when they were moved in the very wet conditions. An initial misunderstanding caused the application of too much water and, when the water content was measured the following day, Segment A was found to have a water content of 28% while Segment C had a water content of 35%.

A rotary hoe was used to re-mix the soil. This was effective in Segment A but in Segment C the tractor tended to sink, displacing large amounts of the Tod Clay as it moved around the track. After mixing, a bulldozer was used to re-level the surface and this became the procedure that was used on all subsequent lifts. The soil was then left to stabilise overnight.

The following day a Ramex self-propelled trench roller with twin, cleated, vibrating drums and skid-steer was used to compact Lift 1. On the drier material it was very effective but became solidly stuck in Segment C. After one complete pass over Lift 1, density levels were measured with a Troxler nuclear density meter (NDM) operating in backscatter mode. The water content values obtained from the NDM were significantly higher than those obtained from oven drying tests. It was subsequently noted that the NDM is very sensitive to the mica content of soils and an adjustment needs to be made to correct for this. Additional roller passes were made while monitoring the density. Segments A and B came close to the dry density target of 1570 kg/m^3. Segment C became unworkable and testing was ignored for this segment. Loadman tests were performed at 5 m stations over the remainder of the track and shear vane tests were done at three transverse positions at these same stations.

The wet material (water content = 35%) in Segment C was removed and replaced with drier soil from the stockpile. This resulted in Segment A being the weaker section. It was decided to trial a heavier tandem steel wheel roller (Sakai SW500) which applied a higher static pressure for compacting the subgrade layers.

Segment C was backfilled again but the water content was not modified. The material in Segment C was rolled with the trench roller and the SW500 roller on static mode only. The SW500 roller caused significant flaking of the surface and after some investigation, it was found that only the top 75 mm of soil was being compacted. Density measurements were performed using both the NDM and sand replacement methods. Other tests included deflection measurements using the Loadman device and (microwave) oven-dried water contents.

Lifts 2 & 3 The nominal height of Lift 2 of the Tod Clay was set at 150 mm and this lift height was maintained through subsequent lifts. A routine of backfilling, rotary hoeing, water content testing, watering to the target, hoeing and re-testing followed for this lift and Lifts 3 and 4. The only variation was that the SW500 roller was abandoned after Lift 2 because the drums were bridging the uneven surface and causing irregular compaction. On Lift 3 a trench rammer and a rubber-tyred roller trailer were trialed without success. Only the trench roller was used on Lift 4. Vane shear testing was confined to Segment A, because the clay in the other segments was too firm and no shearing was occurring at the upper limit of the instrument.

Lift 4 Care was taken when finishing Lift 4 to achieve an even surface. The hoeing contractor was not available so another contractor with a much smaller tractor and hoe was engaged. As a result the work was much slower but the results were comparable. The water content decreased during the first attempt at levelling, so Segment A was hoed again and more water was added. A few soft spots were apparent where the water had concentrated around the sprinklers and it is clear that this method will have to be improved for future projects. The bulldozer had trouble with trimming the surface as the clay tended to ball. A Bobcat bucket was used as a backblade and the rubber roller trailer was used to achieve an even surface. The surface was covered with plastic to prevent the material drying out.

Extensive subgrade testing included surface profiles, nuclear density, sand replacement, Loadman, shear vane, core samples, dynamic cone penetrometer, and in situ CBR tests.

A beneficial aspect of the Tod Clay soil is that it remained very workable at water contents up to 32%. It did not clog the buckets and blades of construction equipment any more than other sandy clays used previously, though it was extremely slippery when wet.

3.2 Base / Sub-base

Lift 5 Lift 5 comprised a 167 mm-thick sub-base of AP40 aggregate. It was placed on top of the clay subgrade and lightly rolled with the rubber wheeled trailer. Loadman tests were then carried out at selected stations. A sieve analysis was also performed on the material.

Lift 6 comprised a 100 mm-thick layer of AP20 basecourse aggregate. The lift was placed and compacted with 5 passes of a Wacker 7060 plate compactor, with the density being monitored using the NDM on each pass. The dry density peaked at approximately 2100 kg/m^3 at 5% water content. Loadman tests were carried out as for Lift 5.

Lift 7 comprised the second lift of AP20 basecourse and that brought the pavement up to the design level. A bulldozer was used to carry out the initial levelling, then CAPTIF staff spent another four days backblading with a Bobcat, hand raking, compacting and dressing the surface. A combination of Wacker plate compactor, another small plate compactor, and the rubber wheeled trailer were used with applications of crusher dust and water to finish the surface. Wet weather delayed drying of the surface and it was several days before loose material could be swept from the surface and measurements taken.

Basecourse testing included surface profiles, NDM density, sand replacement density, and Loadman deflection measurements.

3.3 Asphalt Surface Course

The basecourse surface was heavily tack coated and 29 mm of Mix 7 asphalt was placed by hand over the entire track.

4. CONSTRUCTION TESTING

4.1 Subgrade

Section 2.1 of this report describes a number of tests carried out on the Tod Clay subgrade during its construction. The results of those tests are presented below. Segment A comprises Stations 0 to 20, Segment B comprises Stations 20 to 40, and Segment C comprises Stations 40 to 57.8.

Figure 4.1 shows a plot of water content versus station number for the four Tod Clay subgrade lifts. The plot shows that the water content achieved in Segment A was somewhat variable from lift to lift and from station to station. While the target water content was 30% the actual values ranged from approximately 24% to 37%. The water content in Segment C was much more uniform with an average value of approximately 23%.

CAPTIF staff reported difficulties in achieving uniform water contents and Figure 4.1 confirms that observation. As described in Section 3.1, the sprinkler system used to add water to the soil resulted in concentrations of water around the sprinkler heads. In the future, a more uniform water content may be achieved by adding water to the soil using a pug mill apparatus. The conditioned soil could then be stored in a covered facility for several days to promote equalisation of the water content.

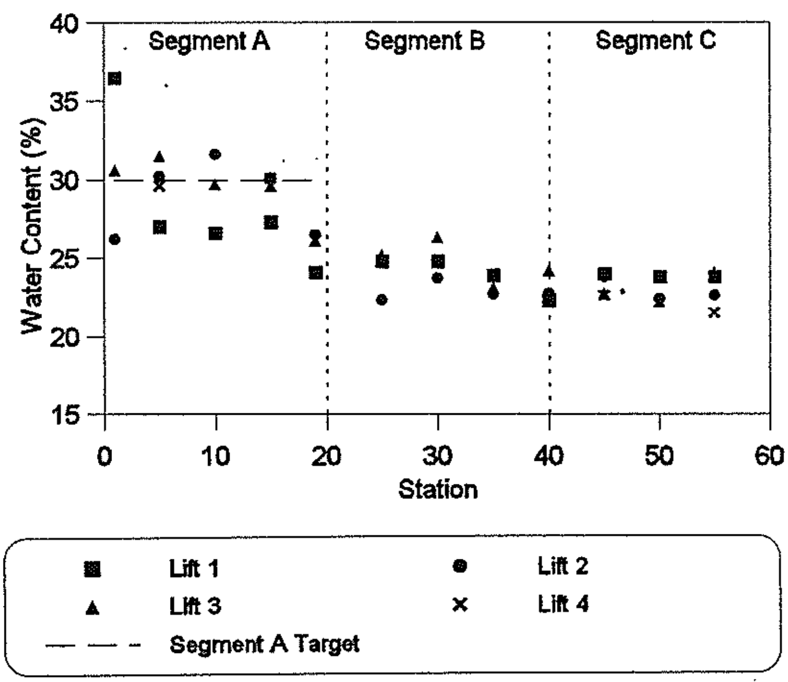


Figure 4.1 Subgrade water content (%) (microwave drying except Lift 4) versus station, for Segments A, B and C, of lifts 1 to 4 in Tod Clay subgrade construction.

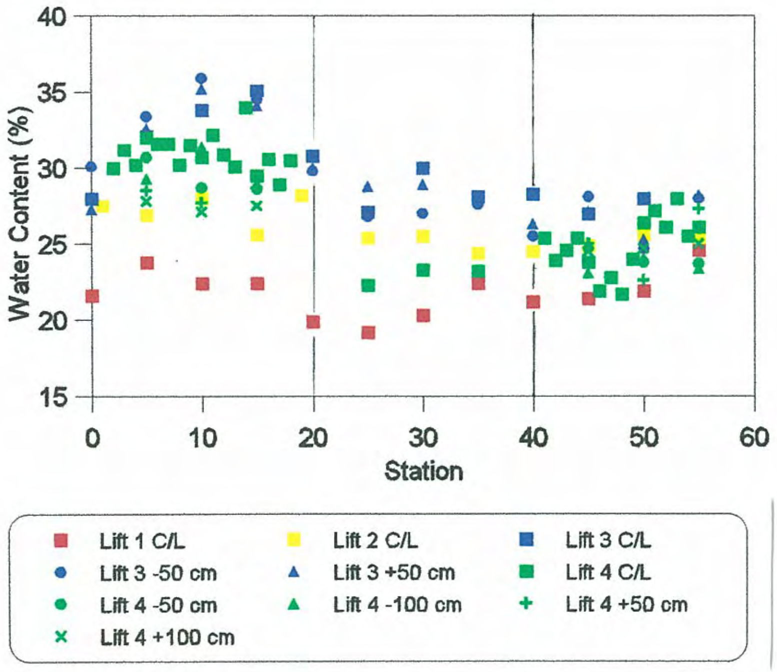


Figure 4.2 **Subgrade water content (NDM; Lift 4 oven-dried) for Lifts 1 to 4 in Tod Clay subgrade construction, at centreline, and offset 50 cm or 100 cm, versus stations for Segments A, B, C.**

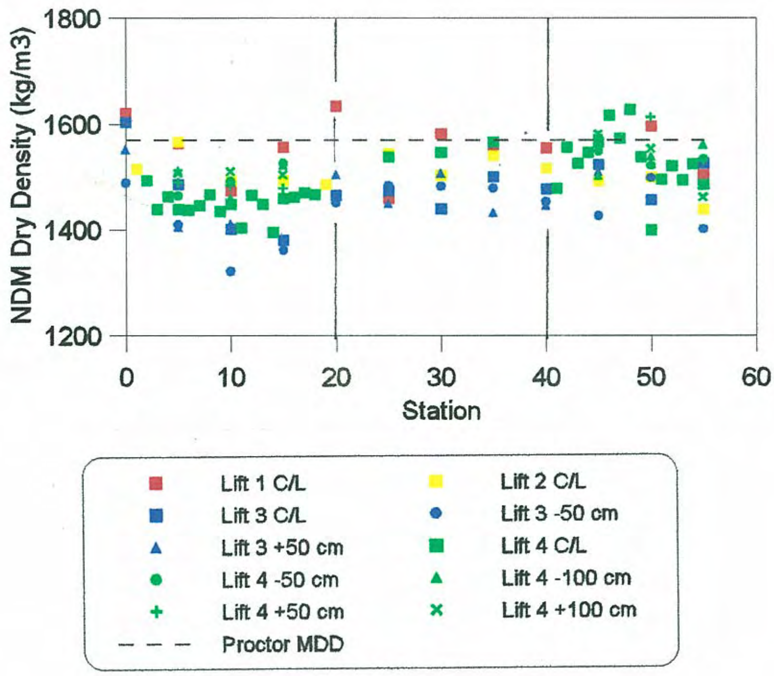


Figure 4.3 **Subgrade dry density (NDM) and Proctor maximum dry density (MDD) for Lifts 1 to 4 in Tod Clay subgrade construction, at centreline, and offset 50 cm and 100 cm, versus stations for Segments A, B, C.**

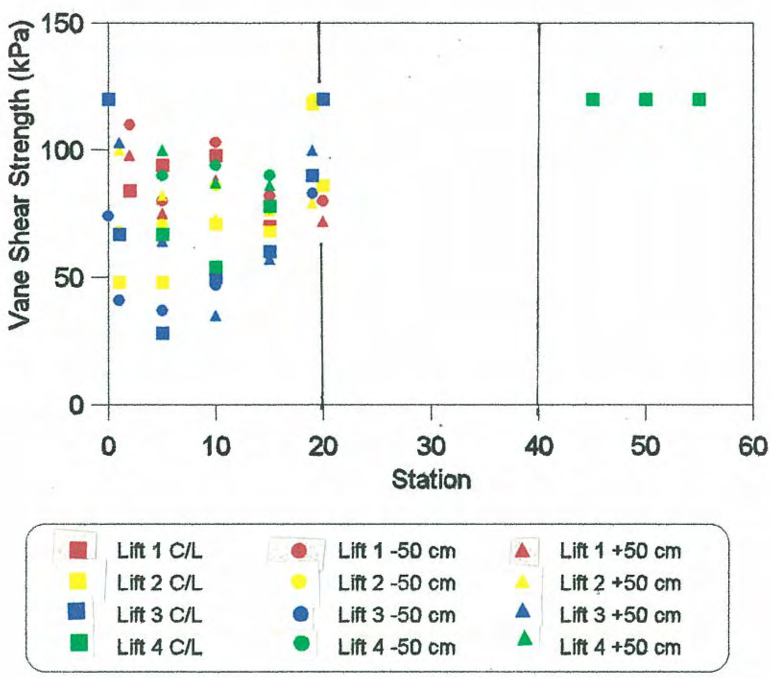


Figure 4.4 **Subgrade vane shear strength for Lifts 1 to 4 in Tod Clay subgrade construction, at centreline, and offset 50 cm, versus stations for Segments A, B, C.**

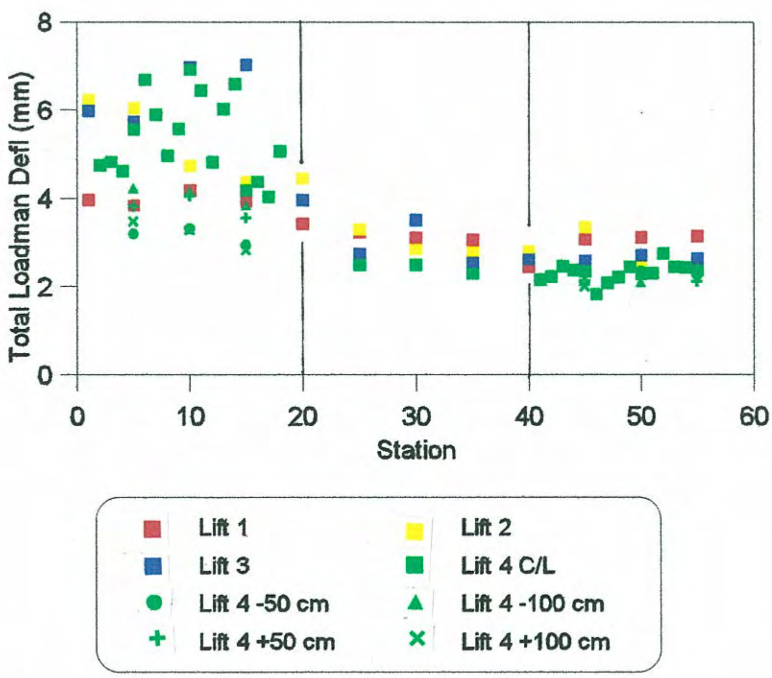


Figure 4.5 **Total Loadman deflections for Lifts 1 to 4 in Tod Clay subgrade construction, at centreline, and offset 50 cm and 100 cm, versus stations for Segments A, B, C.**

The soil placed in Segment C was finally placed at its natural water content which explains the superior consistency in that part of the subgrade. It is apparent that some refinement of the soil conditioning procedure is required to reliably achieve accurate and uniform water contents.

Figure 4.2 shows a similar plot of water content versus station number except the data were recorded using the NDM. The plot includes data from locations offset (by 50 cm and 100 cm) from the wheeltrack centreline for Lifts 3 and 4. Figure 4.2 shows that the water contents obtained using the NDM were generally higher than those obtained from oven drying techniques (see Figure 4.1).

Figure 4.3 shows a plot of dry density versus station number for the Tod Clay soil lifts. The plot shows that the dry densities achieved in Segment A were generally lower than those achieved in Segment C. In numerical terms the dry density values were generally close to the Proctor maximum dry density, except for the tests carried out in Lift 3 and in the Segment A area of Lift 4. These typically achieved about 90% of the maximum dry density value.

Figure 4.4 shows a plot of vane shear strength versus station number for the four Tod Clay subgrade lifts. Note that where the soil shear strength exceeded the capacity of the shear vane apparatus, a value of 120 kPa has been assigned. The plot shows that the soil shear strength in Segment A was somewhat variable, ranging from approximately 30 kPa to greater than 120 kPa. Conversely, the test results from Segment C were all in excess of 120 kPa.

Figure 4.5 shows a plot of total Loadman deflection versus station number for the Tod Clay subgrade. The data tend to mirror the vane shear strength plot, with variable deflections obtained in Segment A, and uniform and relatively low deflections obtained in Segment C.

The data in Figures 4.4 and 4.5 mirror the data in Figure 4.1. This confirms that the soil's (vane) shear strength and (Loadman) stiffness are closely related to the soil water content as suggested in the Stage 2 report of this project (Bartley Consultants Limited 1995).

4.2 Base / Sub-base

The pavement sub-base layer was placed and compacted in a single lift (Lift 5) while the basecourse was placed and compacted in two lifts (Lifts 6 and 7).

Figures 4.6 and 4.7 show plots of sub-base and basecourse water contents and dry densities versus station respectively. Figure 4.6 shows that the water content of the sub-base and basecourse layers ranged from approximately 4% to 6% with Segment C typically having slightly higher water contents than Segment A.

Figure 4.6 Water content (%) versus stations for Segments A-C, for sub-base (Lift 5) and basecourse (Lift 7) layers, at C/L, 50cm and 100cm offsets.

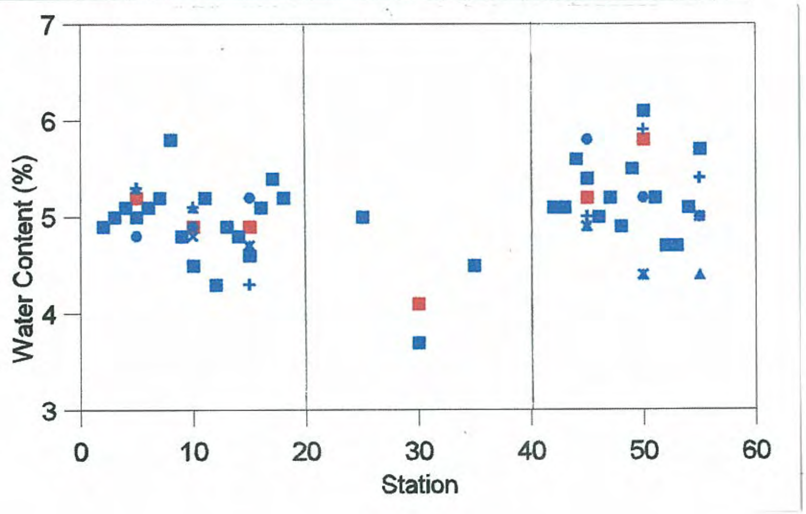


Figure 4.7 NDM dry density (kg/m³) versus stations for Segments A-C, for sub-base (Lift 5) and basecourse (Lift 7) layers.

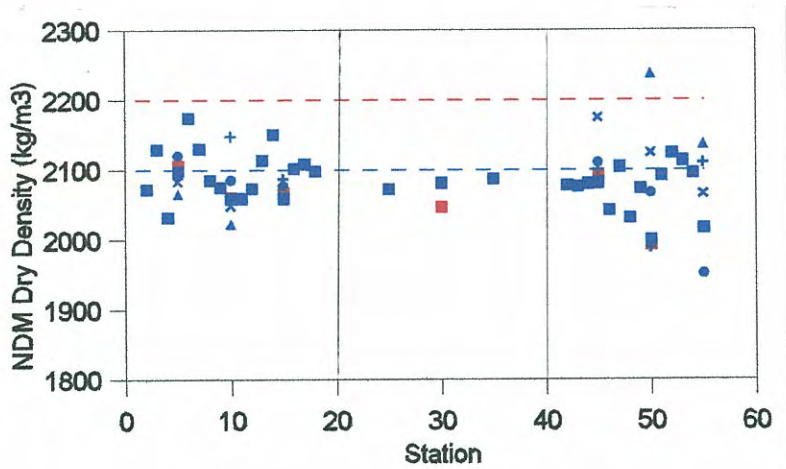


Figure 4.8 Total Loadman deflection (mm) versus stations for Segments A-C, for sub-base (Lift 5) and basecourse (Lifts 6 and 7) layers.

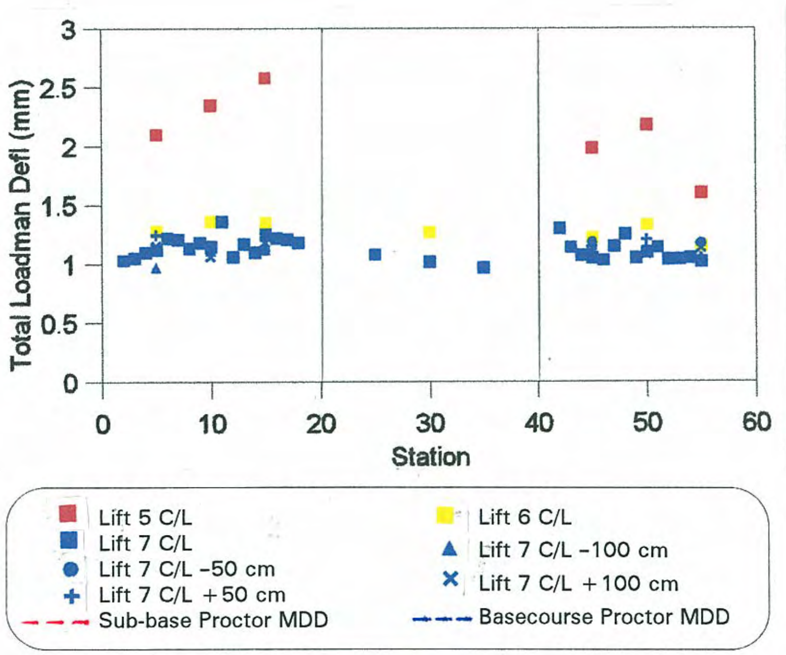


Figure 4.7 shows that the sub-base and basecourse layer dry densities ranged from approximately 1950 kg/m^3 to 2240 kg/m^3. Most of the basecourse dry density results were grouped around the Proctor maximum dry density although there was a slight tendency for the Segment A materials to have a higher dry density than those in Segment C. The sub-base layer achieved dry density values similar to those of the basecourse. However they were all about 5% below the maximum dry density for that material.

Figure 4.8 shows a plot of total Loadman deflection versus station for the sub-base and basecourse layers. The data show that the total deflection decreases from Lift 5 to Lift 6, and from Lift 6 to Lift 7. The final result is that the total Loadman deflection on the top of the basecourse layer is relatively uniform from station to station with a magnitude of approximately 1.0 mm.

4.3 Asphalt Surface Course

The only construction quality test carried out on the top of the asphalt surface course was a longitudinal profile which was established using laser equipment. Figure 4.9 shows a plot of the longitudinal pavement surface profile at the centre of the wheeltrack.

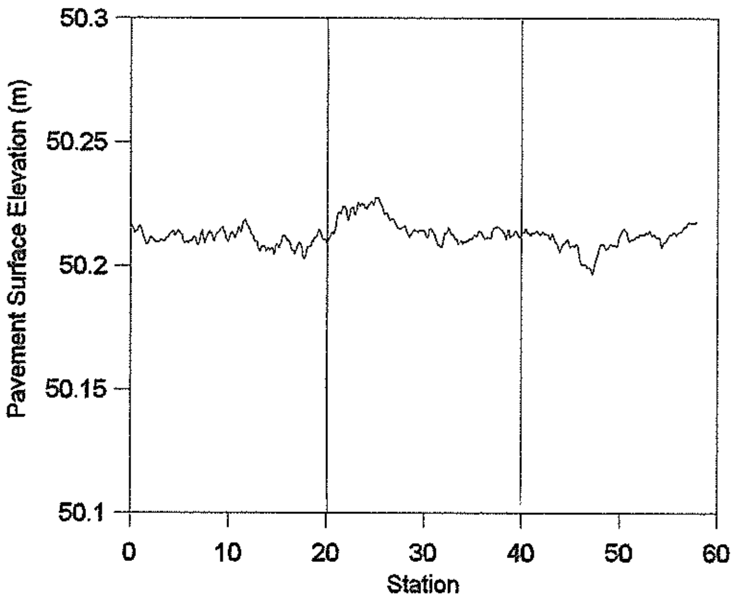


Figure 4.9 Longitudinal pavement surface profile at the wheeltrack centreline for asphalt surface course constructed for the Tod Clay pavement.

5. INTERMEDIATE TESTING

5.1 Rut Depths

Rut depth measurements were carried out at each station at regular intervals during the loading schedule. Figure 5.1 shows a plot of average rut depth versus number of loading cycles for Segments A, B and C of the test pavement.

The data shows that Segment A experienced rutting at a relatively high rate up to approximately 50,000 loading cycles. After that point, rutting continued to accumulate but at a reduced rate. As expected, Segments B and C suffered less rutting and the rate of rut accumulation was relatively small.

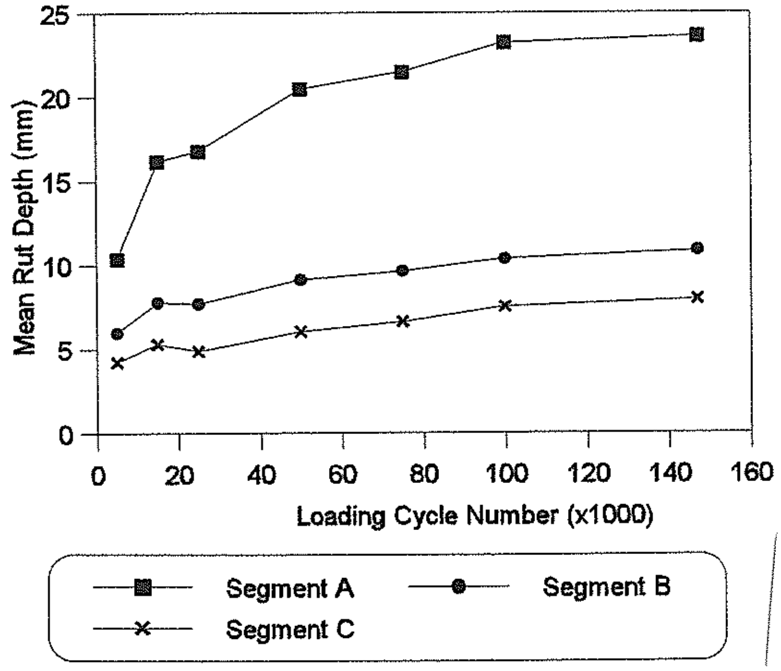


Figure 5.1 Mean rut depth (mm) versus loading cycle number for Segments A, B, C.

5.2 Longitudinal Profiles

Figure 5.2 shows a plot of change in pavement surface elevation versus station number after 147,000 loading cycles. The five traces represent profiles on the centreline and at offsets of +/- 40 cm and +/- 80 cm.

The data in Figure 5.2 mirror the data in Figure 5.1 to some extent. The longitudinal profile trace for the loading centreline shows significantly higher and more variable pavement deformation in Segment A compared with both Segments B and C. As expected, the deformation at 40 cm offsets to the centreline show generally smaller deformations.

Figure 5.2 shows that up to 6 mm of heave was observed in the 80 cm offset profiles. Heaving is caused by a lateral migration of material away from the heavily loaded wheeltrack location. This may have been influenced by the relatively high bitumen content in the asphalt layer and/or shearing within the base/sub-base layers. There is no clear relationship between the magnitude of the heaving and the condition of the subgrade.

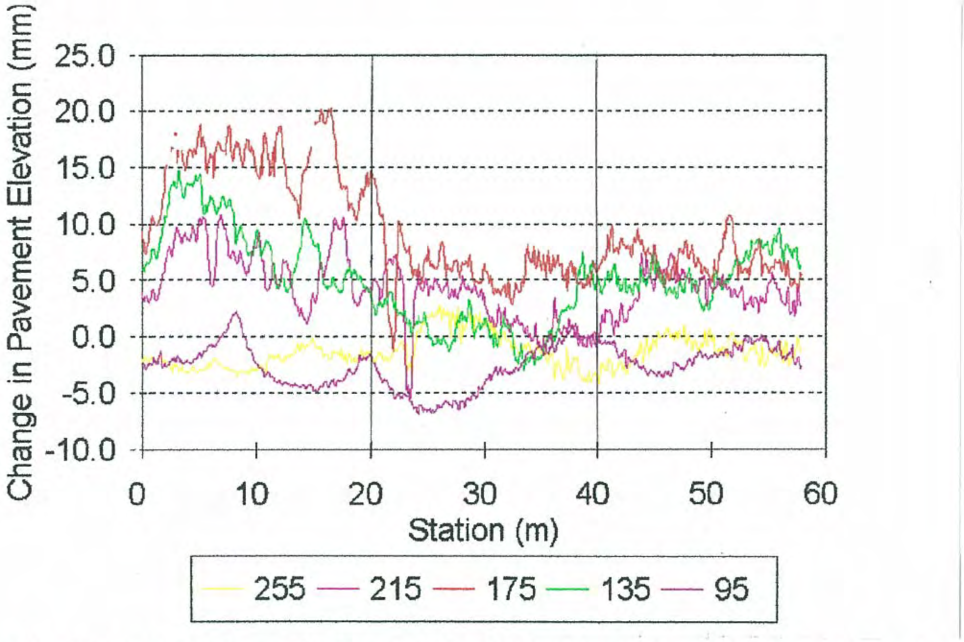


Figure 5.2 Longitudinal profile data at C/L (175 cm), +/- 40 cm (215 cm, 135 cm), and +/- 80 cm (255 cm, 95 cm) offsets versus stations for Segments A, B and C.

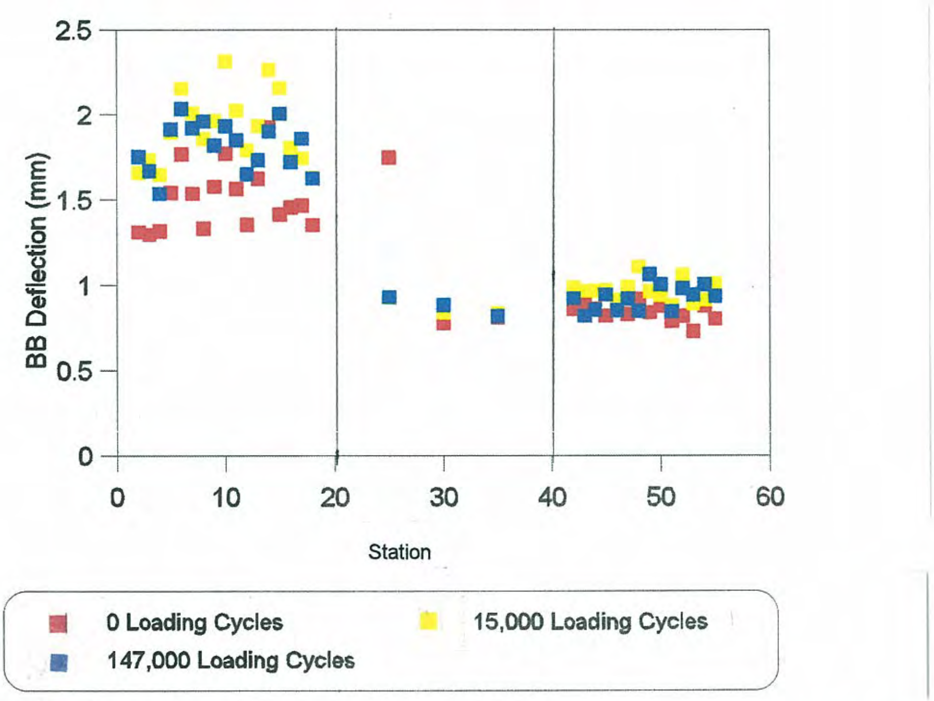


Figure 5.3 Central Benkelman Beam (BB) deflection versus loading cycle number for stations of Segments A, B and C.

5.3 Deflection Tests

Deflection tests were performed on the surface of the test pavement at the end of construction, after 15,000 load cycles, and at the conclusion of the loading, i.e. after 147,000 load cycles. The tests were carried out using CAPTIF's electronic version of the Benkelman Beam.

Figure 5.3 presents central deflection data for each station at 0, 15,000 and 147,000 loading cycles. The data indicate that the central deflection value generally increased between 0 and 15,000 loading cycles but then stabilised or decreased between 15,000 and 147,000 loading cycles. The variation in the deflection data was significantly greater in Segment A compared with Segment C.

The deflection bowl data was back-analysed to determine the elastic modulus of the various pavement layers. The analysis was performed using a computer program comprising a simple layered structure with isotropic elastic materials. The upper layers were modeled as being linear elastic while the subgrade was non-linear elastic. For simplicity, the influence of the asphalt surface course was ignored because of its insignificant thickness. The results of the deflection bowl back-analyses are summarised in Table 5.1.

Table 5.1 Summary of deflection bowl back-analysis results for subgrade.

Pavement Segment	Subgrade Deflection Bowl Back-Analysis Results					
	0 Load Cycles		15,000 Load Cycles		147,000 Load Cycles	
	Elastic Modulus (MPa)	Non-Linearity	Elastic Modulus (MPa)	Non-Linearity	Elastic Modulus (MPa)	Non-Linearity
A	30 - 40	moderate	15 - 25	high	20 - 40	high
C	70 - 80	low	45 - 55	moderate	40 - 75	low

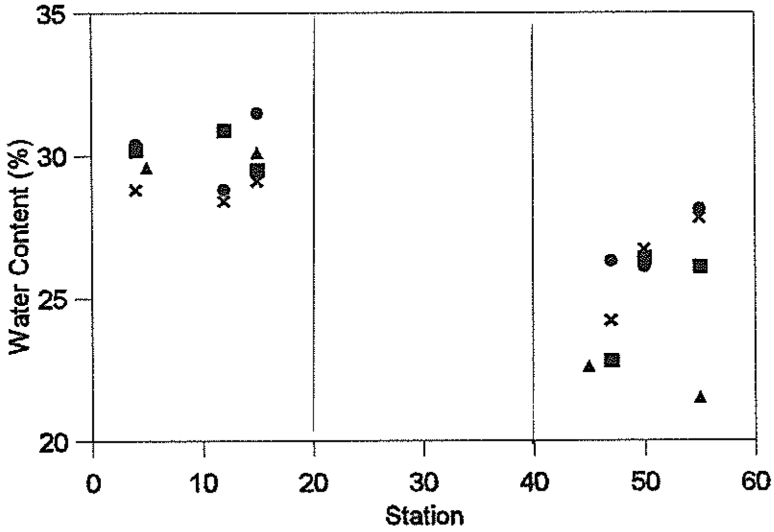
The reason for the apparent reduction and subsequent increase in the subgrade elastic moduli as the testing schedule progressed is not clear. One possible explanation is that pore water pressures were developed during the initial loading cycles. After a period of time the pore water pressures may have dissipated resulting in the tendency for the subgrade elastic moduli to return to their as-constructed values.

In most of the deflection bowl back-analyses, the elastic moduli of the sub-base and basecourse layers were of the order of 50 - 100 MPa and 80 - 250 MPa respectively. However, the influence of the upper layers on the analysis was relatively minor making it difficult to achieve a reliable outcome for these parameters.

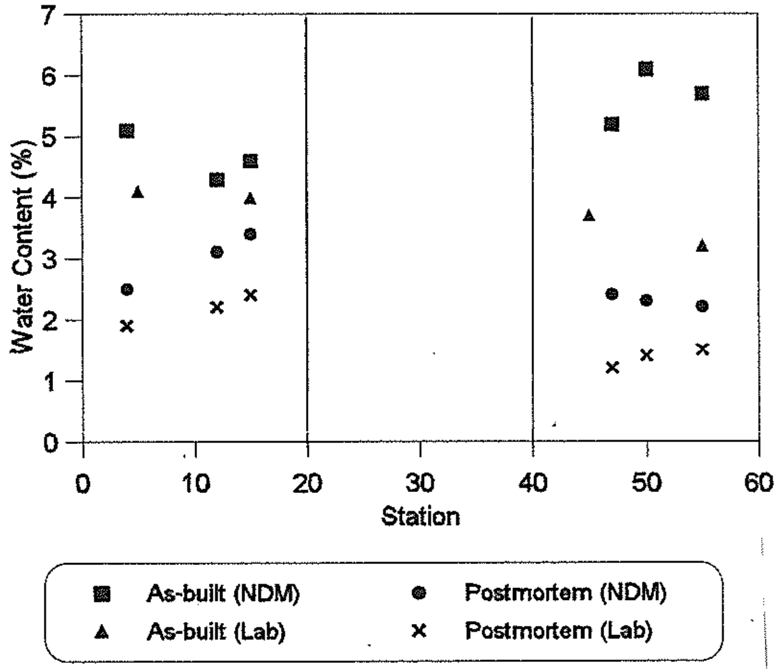
6. POSTMORTEM TESTING

6.1 Water Content

Water content tests were carried out at the completion of the loading schedule, i.e. after 147,000 loading cycles. These tests were performed using both oven drying and NDM techniques. Figures 6.1(a) and (b) show plots of water content (for as-built and postmortem values) versus station for the top of the Tod Clay subgrade and the top of the basecourse respectively.



(a) Top of subgrade



(b) Top of basecourse

Figure 6.1 As-built and postmortem water contents (%) for (a) top of Tod Clay subgrade and (b) top of basecourse versus stations, for Segments A, B and C.

Figure 6.1(a) shows a reasonable correlation between the NDM- and laboratory-derived water content results for the subgrade, except for the as-built result at Station 55. Comparing the as-built and postmortem data indicates that the passage of time and repeated loadings from the start of the test schedule to the end had little influence on the subgrade water content, at least for Segment A (Stations 0 to 20). The Segment C (Stations 40 to 58) subgrade showed a tendency to have an increased water content at the end of the test schedule. The source of the additional water is not clear although it is most likely to have come from the water used in the construction of the overlying sub-base and/ or basecourse layers. Alternatively, the increased water content may reflect normal scatter in the data associated with the sampling and/or the test method.

Figure 6.1(b) shows that the NDM-derived basecourse water content values were consistently higher than the results obtained from the corresponding laboratory-derived tests. Figure 6.1(b) also shows that the water content of the basecourse decreased quite significantly between the start of the loading schedule and the end. This may have been caused by drying during the construction of the asphalt surface course and/or the migration of water during the loading schedule. It is possible that the significant decrease in the basecourse water content in Segment C may be related to the slight increase in subgrade water content for the corresponding area (Figure 6.1(a)).

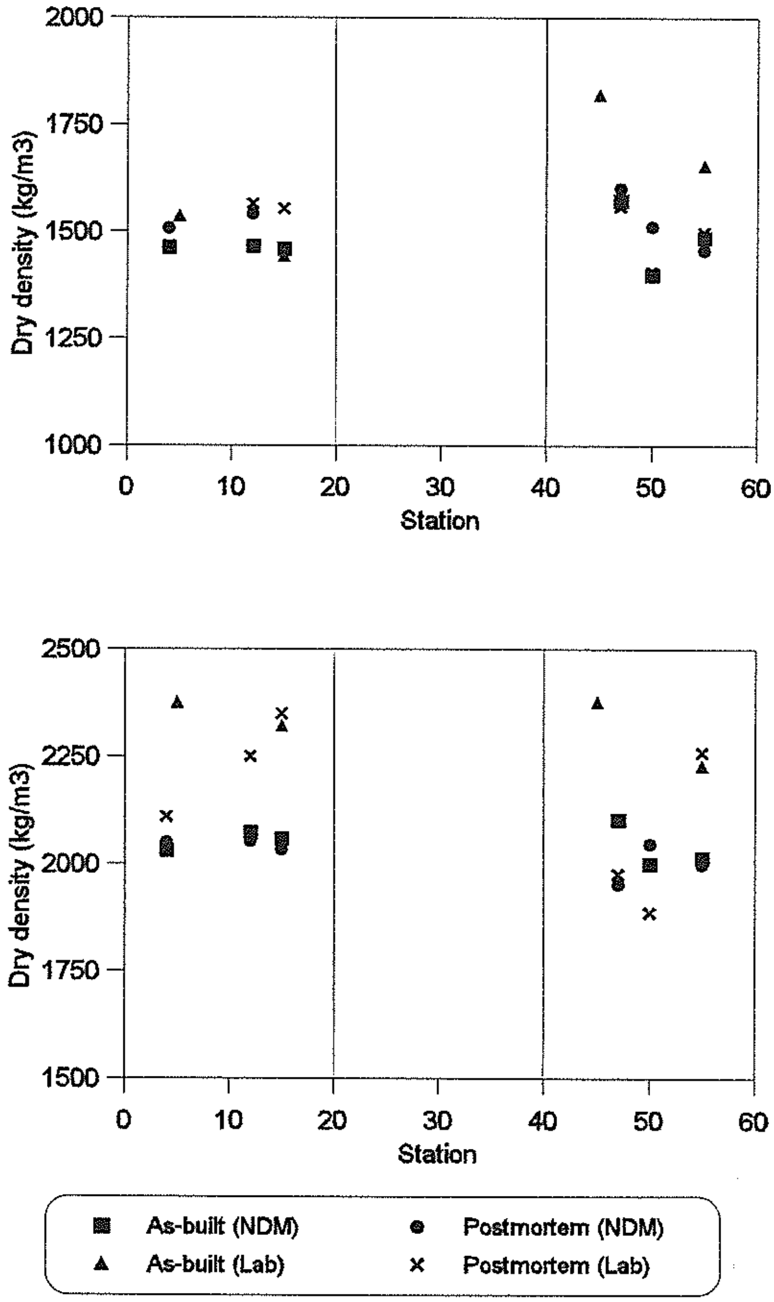
6.2 Dry Density

Density tests were carried out at the completion of the loading schedule, i.e. 147,000 loading cycles. The tests were performed using both sand replacement and NDM techniques. Figures 6.2(a) and (b) show plots of dry density versus station for the top of the Tod Clay subgrade and the top of the basecourse respectively.

Figure 6.2(a) shows a reasonable correlation between the NDM- and laboratory-derived dry density results for the subgrade, except for the as-built result at Station 55. Comparing the as-built and postmortem data indicates that the passage of time and repeated loadings from the start of the test schedule to the end had little influence on the subgrade dry density.

Figure 6.2(b) shows that the NDM-derived basecourse dry density results were consistently lower than the corresponding laboratory-derived tests. This is most likely caused by the inconsistency of the basecourse water content results described in Section 6.1.

Figure 6.2(b) also shows that no significant change occurred in basecourse dry density between the start of the repeated loading schedule and the end. This may be somewhat surprising as it is reasonable to expect some increase in density caused by trafficking compaction, even though the basecourse was well compacted at the time of construction.



(a) **Top of subgrade**

(b) **Top of basecourse**

Figure 6.2 As-built and postmortem dry density values for (a) top of Tod Clay subgrade and (b) top of basecourse versus stations, for Segments A, B and C.

At least two mechanisms could be occurring in the basecourse. The first mechanism is where the repeated loading imposes small shear strains on the basecourse layer which tend to re-arrange the aggregate particles into a closer packing regime, and therefore increase the layer density. An opposing mechanism associated with the observed pavement heaving may also be occurring, where somewhat larger shear strains cause the aggregate structure to dilate, and therefore decrease the layer density. It is suggested that the reasonably constant basecourse dry density is a result of a combination of these two mechanisms.

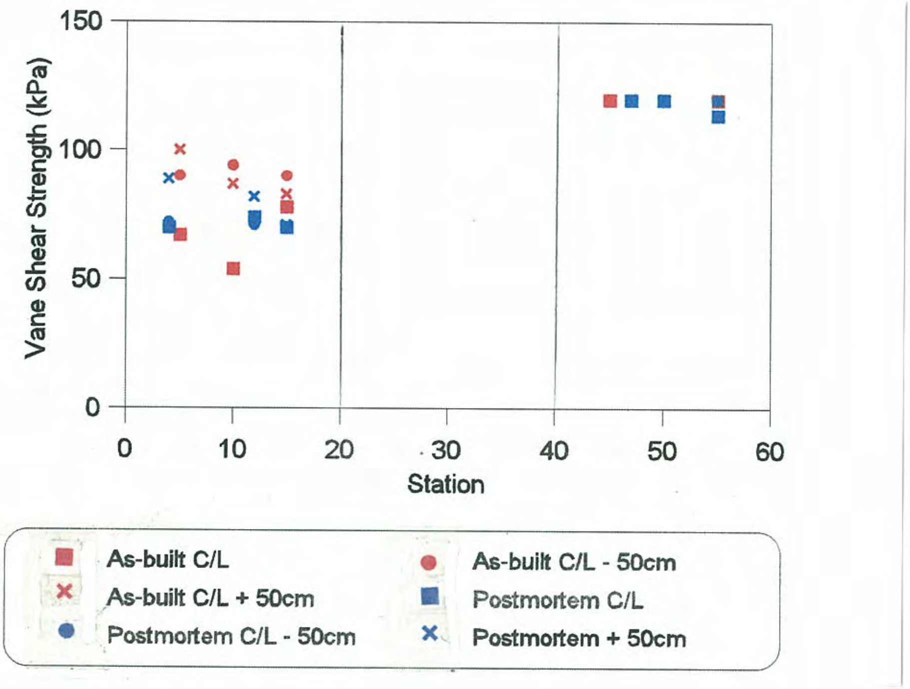
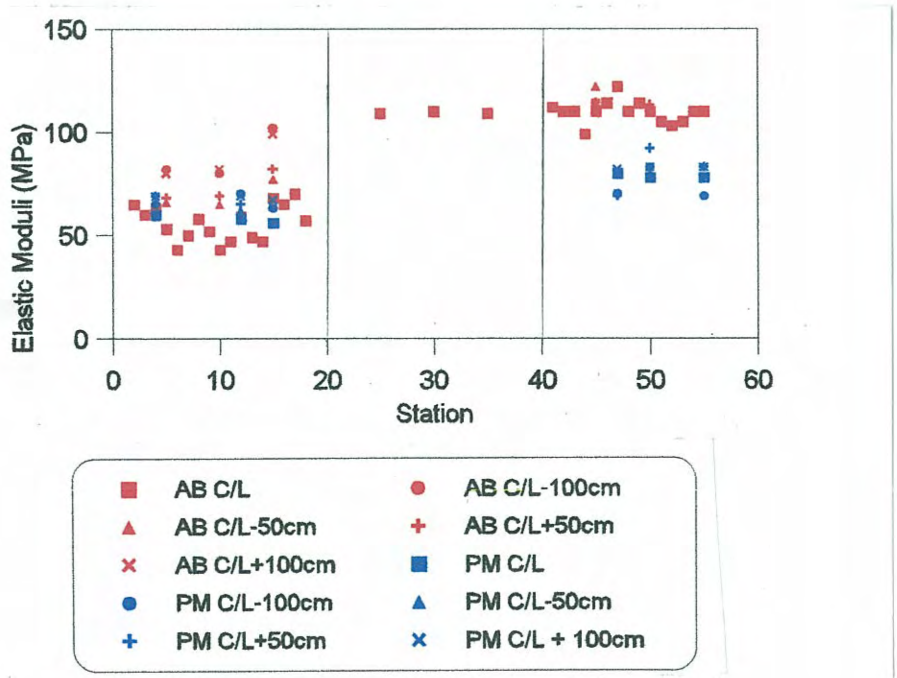


Figure 6.3 As-built and postmortem vane shear strengths of top of Tod Clay subgrade versus stations, for Segments A, B and C.



(AB as-built; PM postmortem)

Figure 6.4 Loadman back-calculated elastic moduli (MPa) of Tod Clay subgrade versus stations, for Segments A, B and C.

6.3 Subgrade Vane Shear Strength

Shear vane tests were carried out during construction of the Tod Clay subgrade and in the postmortem testing. Figure 6.3 shows a plot of vane shear strength versus station number for test locations on the centreline of the loaded area and at +/- 50 cm offsets. All tests were performed at the top of the Tod Clay subgrade.

Figure 6.3 shows that the vane shear strengths in Segment C were consistently high in the as-built and postmortem conditions. In Segment A the vane shear strengths were generally lower and less consistent. No significant trend was established between the as-built and the postmortem shear strengths.

6.4 Subgrade Loadman FWD Tests

Loadman tests were carried out on the top of the Tod Clay subgrade in the as-built and postmortem conditions. Figure 6.4 shows a plot of subgrade elastic moduli back-calculated from the Loadman deflection results.

Figure 6.4 shows that the as-built and postmortem subgrade elastic moduli coincided very well for Segment A. However, for Segment C the elastic moduli obtained from the as-built testing were consistently higher than the values obtained from the postmortem testing. This is most likely explained by the tendency for the subgrade water content in Segment C to increase in the period from the end of construction to the end of the loading schedule, as shown in Figure 6.1(a).

The data presented in Figure 6.4 also indicate that the Loadman is a useful device for subgrade testing as it successfully illustrated the effect of the increase in water content in Segment C. The numerical values of the inferred elastic moduli obtained by the Loadman are also similar to the expected values for the Tod Clay soil as determined from repeated load triaxial tests.

6.5 In Situ CBR Tests

In situ CBR tests were carried out on the top of the Tod Clay subgrade in the as-built and postmortem conditions. In both sets of tests an 8.0 kg surcharge was used. The results of the CBR tests are presented in Figure 6.5.

Figure 6.5 shows that the as-built in situ CBR test results were slightly lower than the corresponding postmortem results for Segment A. However, the as-built CBR results were somewhat higher than the postmortem results for Segment C. The latter observation is consistent with the Loadman test results and is likely to be a consequence of the slight increase in water content during the repeated loading schedule (see Figure 6.1(a)).

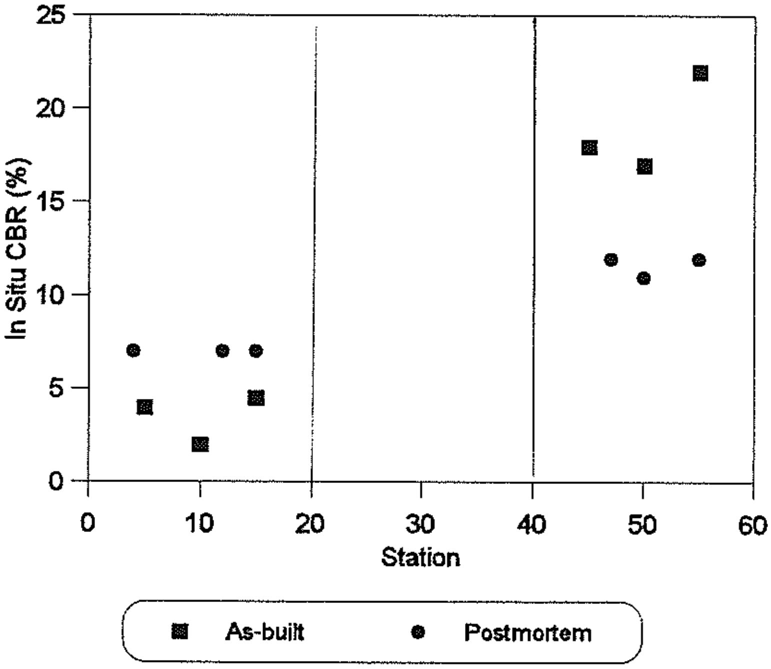


Figure 6.5 In situ CBR (%) for Tod Clay subgrade versus stations,for Segments A, B and C.

A point of conjecture that is often noted in the technical literature is the apparent relationship (or lack of) between the elastic modulus and CBR parameters. The AUSTROADS pavement design procedures (AUSTROADS 1992) suggest that the vertical elastic modulus of a material is approximately equal to ten times the CBR. This relationship is based on a material model with a degree of anisotropy of 2.0 (i.e. $E_{vertical} / E_{horizontal} = 2.0$) and a Poisson's Ratio of 0.45 for a cohesive subgrade soil.

The data in Figures 6.4 and 6.5 indicate that the AUSTROADS relationship appears to be reasonable for the Segment A test results but that the elastic modulus results for Segment C are somewhat less than ten times the CBR.

6.6 Dynamic Cone Penetrometer Tests

Dynamic Cone Penetrometer (DCP) tests were carried out on the Tod Clay subgrade in the as-built and postmortem conditions. The results of the DCP tests are shown in Figures 6.6 to 6.7. As the thickness of the Tod Clay subgrade was approximately 600 mm, i.e. the top 600 mm of each plot represents the properties of the Tod Clay subgrade, the remainder of the plot represents the properties of the underlying Waikari soil. Also the plots are presented with respect to mm/blow of the DCP. Therefore, the lower the mm/blow value, the greater the soil's resistance to penetration.

Four facts that can be deduced from the DCP results are as follows:

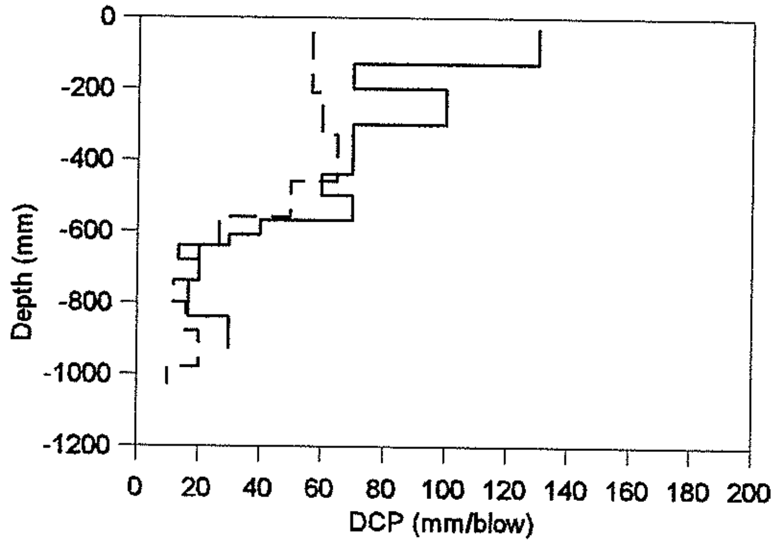
- In general there is no significant change in DCP blow count between the as-built and the postmortem tests. A possible exception is shown in Figure 6.6(a) where the Tod Clay has a greater resistance to the DCP at the postmortem stage compared to the as-built stage. In addition, the as-built soil resistance at the top of the subgrade (0 mm) shown in Figure 6.6(a) generally exceeds the postmortem soil resistance.

- DCP tests successfully delineate between the Tod Clay subgrade and the underlying Waikari soil (c.600 mm depth), particularly for Segment A where the properties of the two soils show greater contrast.

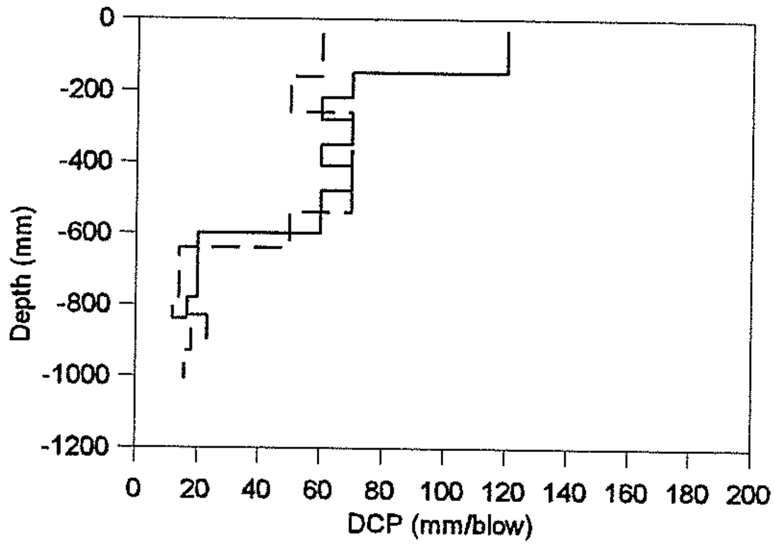
- DCP indicates that the typical result for the Tod Clay in Segment A is in the range of 50 to 70 mm/blow. Using the AUSTROADS (1992) relationship between DCP and CBR indicates that the inferred CBR for the Tod Clay subgrade in Segment A is approximately 2.5% to 3.5%. This is slightly lower than the results obtained from in the situ CBR tests.

- DCP indicates that the typical result for the Segment C soil is in the range of 30 to 40 mm/blow. Using the AUSTROADS relationship between DCP and CBR indicates that the inferred CBR for the Tod Clay subgrade in Segment C is approximately 4.5% to 6.5%. Again, this is lower than the results obtained from the in situ CBR tests. This discrepancy demonstrates the drawbacks associated with using published relationships between material parameters, and the need to establish separate relationships which are calibrated to the performance of local materials.

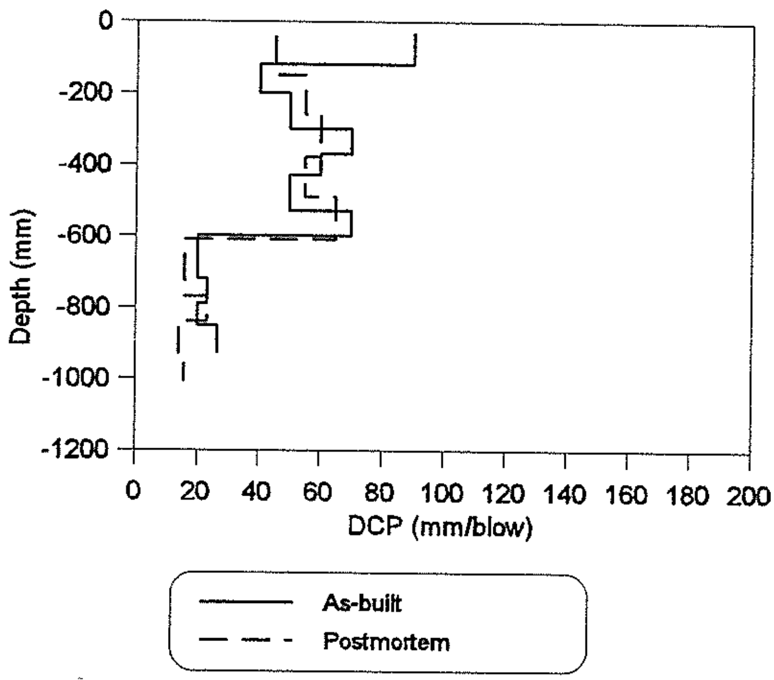
Figure 6.6 DCP (mm per blow) versus depth (mm) for Segment A Stations (as-built and postmortem) for Tod Clay subgrade (0-600 mm depth).



(a) Station 5 (as-built) and Station 4 (postmortem)



(b) Station 10 (as-built) and Station 12 (postmortem)



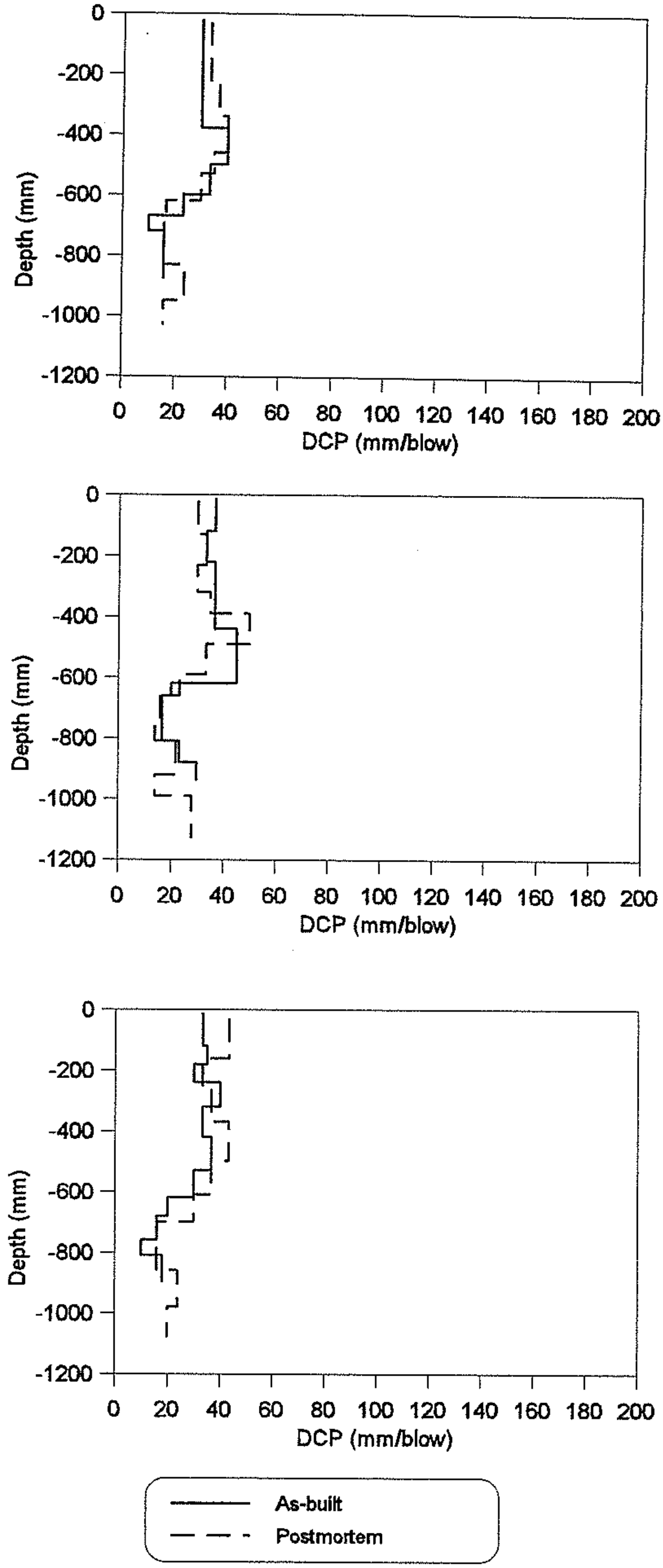
(c) Station 15 (as-built and postmortem)

Figure 6.7 DCP (mm per blow) versus depth (mm) for Segment C Stations (as-built and postmortem) for Tod Clay subgrade (0-600 mm depth).

(a) Station 45 (as-built) and Station 47 (postmortem)

(b) Station 50 (as-built and postmortem)

(c) Station 55 (as-built and postmortem)



6.7 Transverse Profiles

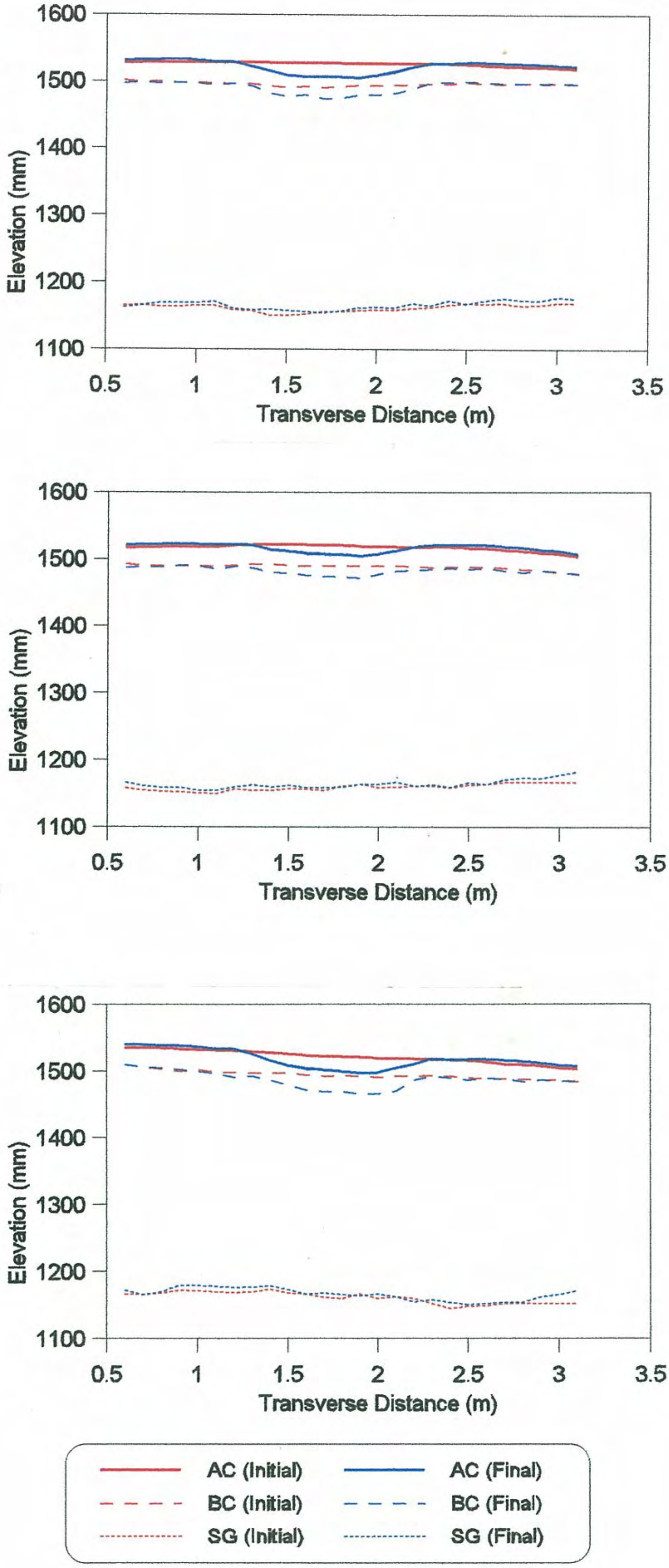
Transverse profiles were measured on the asphalt (AC), basecourse (BC) and Tod Clay subgrade (SG) surfaces at the time of construction and again in the postmortem testing. Figures 6.8 and 6.9 show initial and final transverse profiles at Stations 4, 12, 15 (Segment A), 47, 50 and 55 (Segment C) respectively.

The transverse profiles measured in the postmortem testing are subject to errors caused by penetration of the sub-base aggregate into the top of the subgrade, and by adherence of aggregate particles to the underside of the asphalt layer.

Figures 6.8a-c represent transverse profiles measured in Segment A and they show the presence of significant ruts at the pavement surface. Figures 6.9a-c represent transverse profiles measured in Segment C and they show significantly less rutting, as expected for this segment of pavement which had been constructed on a superior subgrade.

A feature of all of the transverse profiles is that very little, if any, deformation at the pavement surface can be attributed to permanent deformation at the top of the subgrade. Most of the deformation appears to be originating in the basecourse/sub-base layers. A closer examination of the layer elevations suggests that approximately 90% to 94% of the surface rutting stems from the basecourse. Almost all the other permanent deformation is occurring in the asphalt layer, and the subgrade has negligible influence.

Figure 6.8 Initial (as-built) and final (postmortem) transverse profiles for asphalt (AC), basecourse (BC) and Tod Clay subgrade (SG) layers, at Segment A stations.

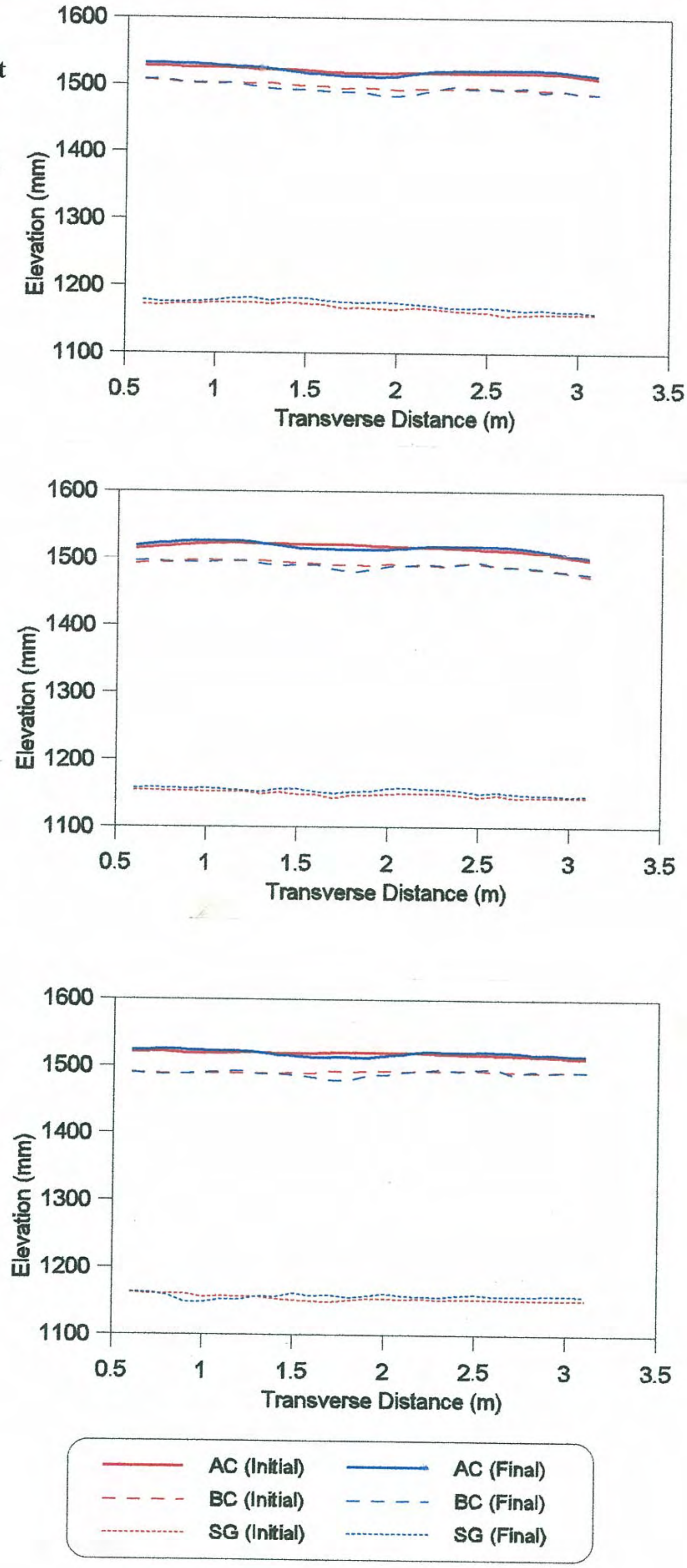


(a) Station 4

(b) Station 12

(c) Station 15

Figure 6.9 Initial (as-built) and final (postmortem) transverse profiles for asphalt (AC), basecourse (BC), and Tod Clay subgrade (SG) layers, at Segment C stations.



(a) Station 47

(b) Station 50

(c) Station 55

7. CONCLUSIONS & RECOMMENDATIONS

7.1 Conclusions

A large quantity of data was obtained from monitoring the construction and performance of materials for their suitability for use in the CAPTIF test pavement. Several test procedures were used to monitor properties of the materials. Most of the test data provide a consistent and credible characterisation of the pavement materials, with a particular emphasis on the Tod Clay subgrade.

The main conclusions that can be drawn for the project are the following:

- The Tod Clay soil is suitable for constructing test pavement subgrades at CAPTIF. It is workable over a reasonable range of water contents and did not suffer significant changes in water content or dry density after the application of a large number of repeated loads.

- Using the soil water content to control the strength/stiffness properties of the subgrade is an appropriate technique. However, the method of achieving the desired water content requires improvement as the sprinkler approach used in this project produced somewhat inaccurate and inconsistent results.

- The Loadman is a valuable tool for characterising pavement materials, requiring minimum of effort both during the testing and in postmortem analyses.

Secondary findings that support the main conclusions are as follows:

Construction Testing

- The water content of the Tod Clay in Segment A ranged from approximately 24% to 37%, compared with the target water content of 30%. The water content in Segment C was as found in the stockpile, i.e. approximately 23%.

- The main reason for the inconsistency in the water content in Segment A was that the sprinkler system used to add water to the soil resulted in some ponding and concentrations of water. Conversely, the consistency of the water contents in Segment C can be attributed to the fact that the soil was placed at its natural water content, and therefore it had achieved a state of equilibrium before it was placed.

- Care must be taken when using the NDM to determine water contents in the Tod Clay. The NDM-derived values were generally higher than the results obtained from oven-dried test specimens. Appropriate calibration of the NDM must be carried out for this material.

- At the end of construction, the dry density of the Tod Clay subgrade showed a slight trend of being higher in Segment C than it was in Segment A.

- Vane shear strength and Loadman (total) deflection data for the as-built subgrade showed very similar responses. In both tests the results obtained in Segment A were inconsistent and indicated a relatively low strength/stiffness condition, while in Segment C the results were very consistent and indicated a significantly higher strength/stiffness condition.

- The water content and dry density data for the base and sub-base layers at the end of construction showed little difference between the conditions of the aggregate layers in Segments A and C. If anything, there was a slight trend for the water content in Segment C to exceed that in Segment A, and for the dry density in Segment A to exceed that in Segment C.

- The Loadman (total) deflection data for the tests carried out on the as-built base and sub-base layers show that higher deflections were obtained in the lower lifts of Segment A. However very consistent deflections were obtained on the top of the basecourse for both Segments A and C.

- The Loadman device showed considerable potential for characterising pavement materials. It was effective in the current project and had many advantages over alternative test procedures such as the shear vane and the in situ CBR. The automated Benkelman Beam apparatus also provided valuable results but the data required significant post-processing. The DCP is a valuable tool because it provides a quick way of obtaining test data through a pavement profile, but it lacks the speed and precision of the Loadman.

Intermediate Testing

- Rut depths were greatest in Segment A. The rate of rut accumulation was relatively high for approximately the first 50,000 loading cycles. After that point the rate of rut accumulation decreased and approached a stable condition.

- Longitudinal profiles at the pavement surface showed that the permanent deformation at the centre of the wheel track was greatest and most inconsistent in Segment A. Areas of heave were identified at locations offset from the centreline, but there was no correlation between the location or severity of the heave and the condition of the subgrade.

- Analysis of deflection bowl tests showed that, at the end of construction, the subgrade in Segment A had an elastic modulus typically in the range 30 - 40 MPa. After 15,000 load cycles the elastic modulus decreased to 15 to 25 MPa. At the end of the loading schedule (147,000 cycles) the elastic moduli tended to regain the original range of values. A similar pattern of behaviour occurred in Segment C where the as-built elastic moduli were typically in the range 70 - 80 MPa before dropping to 45 - 55 MPa, and then tending to regain their original values.

Postmortem Testing

- The postmortem water content tests showed that the oven-dried and NDM test results were reasonably consistent for the Tod Clay but were very inconsistent for the basecourse. Segment C of the subgrade showed a slight tendency to increase in water content at the end of the test schedule. The water content in the basecourse showed significant drying between the start and end of the test schedule.

- The dry density of both the basecourse and the subgrade did not change significantly during the testing schedule. This was somewhat surprising considering that significant ruts had formed in Segment A and heaving was identified at several locations on the test pavement.

- The vane shear strength, Loadman and in situ CBR tests all showed similar material performance tendencies but with varying degrees of significance. Of these tests, the back-calculated elastic moduli obtained from the Loadman test results presented the clearest picture. The data showed that the as-built and postmortem elastic moduli in Segment A were reasonably consistent, whereas in Segment C the elastic moduli obtained in the postmortem testing were somewhat less than the as-built values. The reason for this drop-off is not clear. However it may have been caused by the slight increase in the water content of the subgrade during the testing schedule.

- Comparing the subgrade elastic moduli obtained from the Loadman test results with the CBR results showed that a reasonable correlation was found for the Segment A data. For Segment C there was a different correlation with elastic moduli and CBR.

- The DCP tests showed reasonably consistent results for the as-built and postmortem conditions. Applying the AUSTROADS relationship between DCP and CBR produced CBR results which were somewhat lower than the measured values.

- Transverse profiles showed that very little, if any, deformation at the pavement surface can be attributed to permanent deformation at the top of the subgrade. Most of the deformation (90% - 94%) appears to have originated in the basecourse / sub-base layers. Almost all the other permanent deformation is occurring in the asphalt layer, and the subgrade has negligible influence.

7.2 Recommendations

The experience gained during the construction, loading and monitoring of the test pavement has resulted in the identification of areas where the subgrade construction at CAPTIF can be improved, i.e:

- The manner in which the soil is conditioned needs improvement. The sprinkler system used during this trial did not provide the uniformity required. A pug mill or similar equipment should achieve a much more consistent water content and avoid the soft spots caused by the concentration of water around the sprinkler heads.

- The construction of a covered storage area should be an objective to strive for at CAPTIF. A covered storage facility would allow the equilibration of soil water content during CAPTIF's down-time and would protect the soil stockpile from erosion and loss. A covered facility would also provide a buffer area for soil that has already been conditioned in the pug mill, to be stockpiled before it is placed in the test pavement.

- A self-propelled trench roller is appropriate for the compaction of the CAPTIF subgrade soil. While alternative plant should not be excluded from consideration, the trench roller has proved to be effective in this project.

- The Loadman apparatus shows significant potential as a tool for characterising the elastic modulus of pavement materials, and it is recommended for future use in CAPTIF projects.

These issues have been addressed in the development of a revised CAPTIF subgrade construction specification. The specification is presented in Appendix B of this report.

8. REFERENCES

AUSTROADS. 1992. *Pavement Design: A Guide to the Structural Design of Road Pavements.* AUSTROADS Publication No. AP 17/92, Sydney, Australia.

Bartley Consultants Limited. 1995. *Investigation to determine materials and methods necessary to prepare appropriate subgrades for use at CAPTIF.* Stages 1 and 2 of Transit New Zealand Project PR3-0032. Transit New Zealand, Wellington, New Zealand.

Bartley Consultants Limited. 1998. Loadman portable falling weight deflectometer: determination of elastic moduli of pavement materials. *Transfund New Zealand Research Report No. 124.* 41pp.

Standards Association of New Zealand (SANZ). 1986. Methods of testing soils for civil engineering purposes. *New Zealand Standard NZS 4402:1986.*

APPENDIX A

ENGINEERING CHARACTERISTICS OF TOD CLAY

ENGINEERING CHARACTERISTICS OF TOD CLAY

A.1 General

The second stage of this project involved a thorough laboratory investigation into the characteristics of the Tod Clay soil, using standard procedures given in NZS4402:1986 (SANZ 1986). This appendix provides a summary of the test results reported in the Stage Two research report (Bartley Consultants Limited 1995).

A.2 Test Results

The samples of Tod Clay tested in the laboratory had the following characteristics:

Liquid Limit	= 69%
Plastic Limit	= 27%
Plasticity Index	= 42%
Natural Water Content	= 14%

NZ Standard Compaction (Test 4.1.1, New Zealand Standard 4402:1986)

Maximum Dry Density	= 1.62 t/m^3
Optimum Water Content (OWC)	= 20%

NZ Heavy Compaction (Test 4.1.2, New Zealand Standard 4402:1986)

Maximum Dry Density	= 1.74 t/m^3
OWC	= 14%
CBR at OWC	= 45%
CBR Soaked	= 1%

The dry density versus water content relationship for the samples of soil used in the resilient modulus tests is shown as Figure A1. The performance of samples in the resilient modulus test is illustrated in Figures A3 to A10 inclusive.

A plot of the results of the resilient modulus test versus water content for Tod Clay and for Calben (calcium bentonite) Clay is shown in Figure A2. This graph also indicates the range of subgrade strengths adopted for this project. Calben Clay was another candidate for a *CAPTIF* subgrade but it was rejected because it was too difficult to work with.

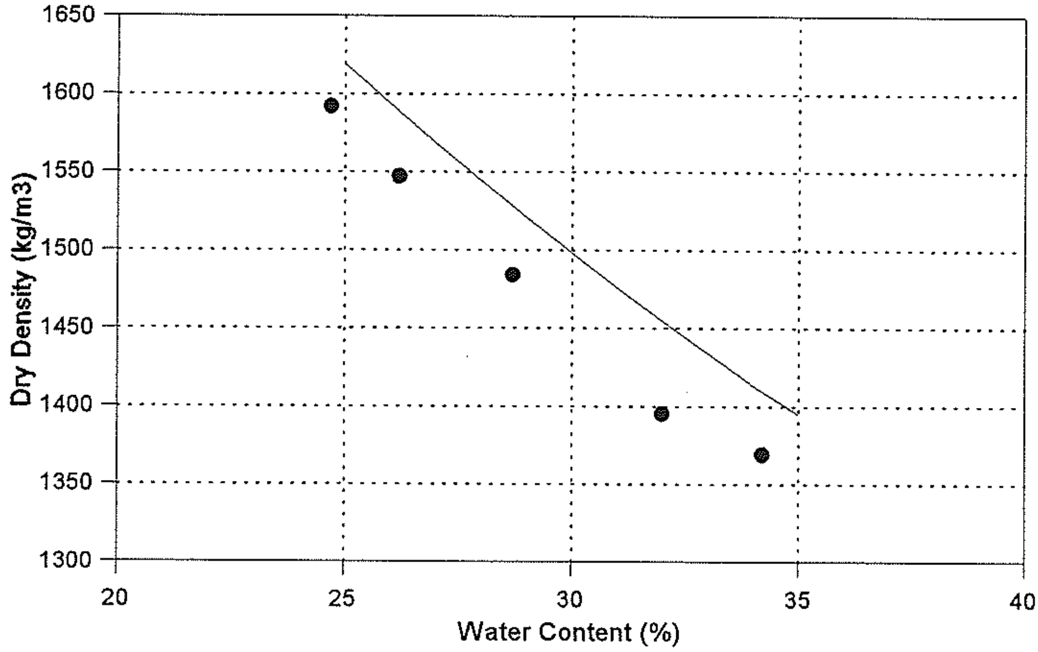


Figure A1 Plot of dry density versus water content (%) for the Tod Clay specimens.

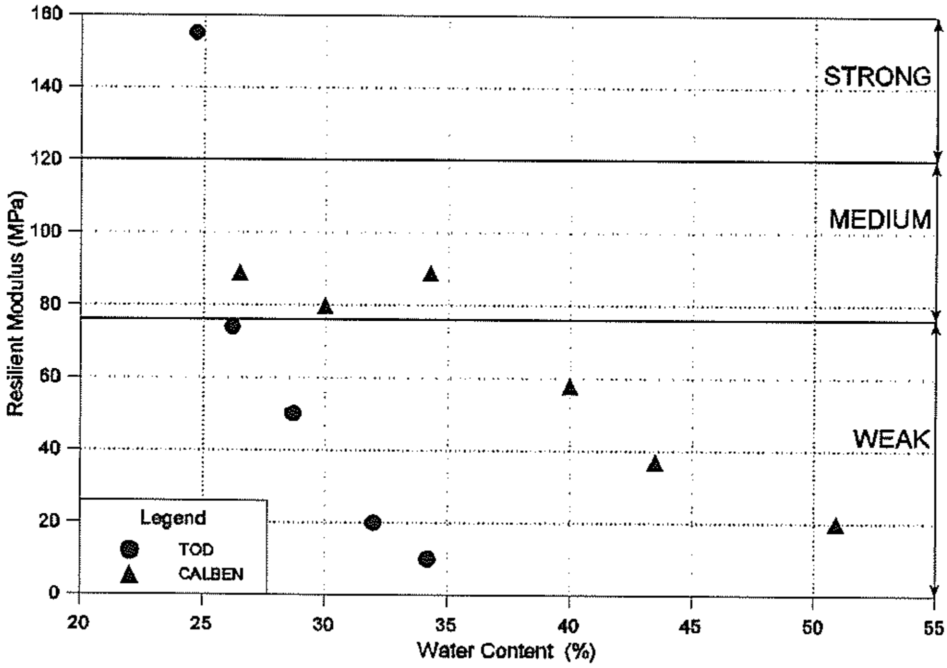


Figure A2 Plot of resilient modulus versus water content (%) for the Tod Clay and Calben Clay specimens.

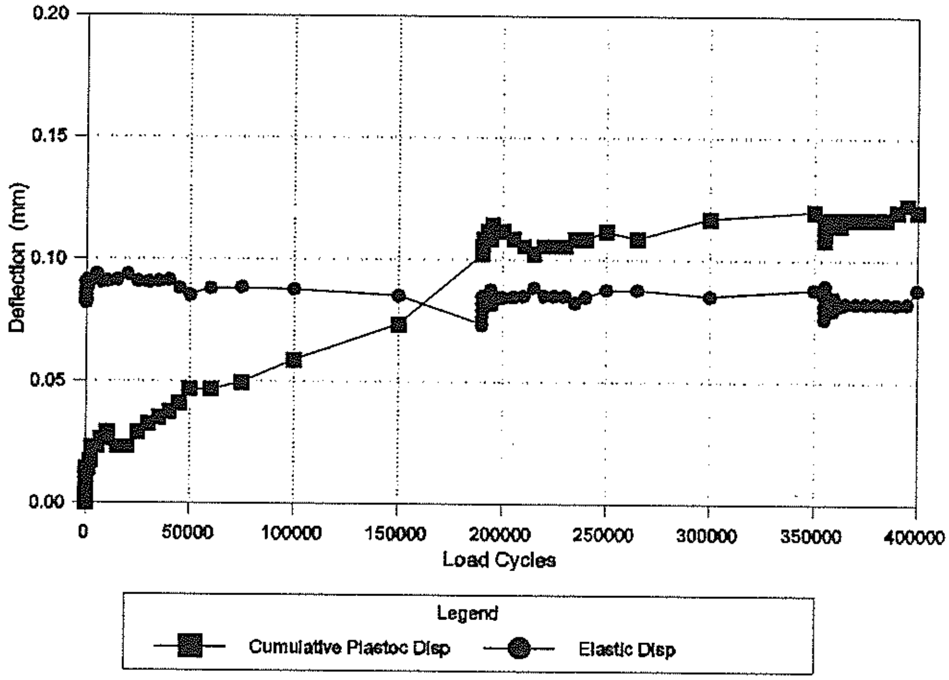


Figure A3 Plot of elastic and plastic deflection versus loading cycles for Tod Clay Specimen 1.

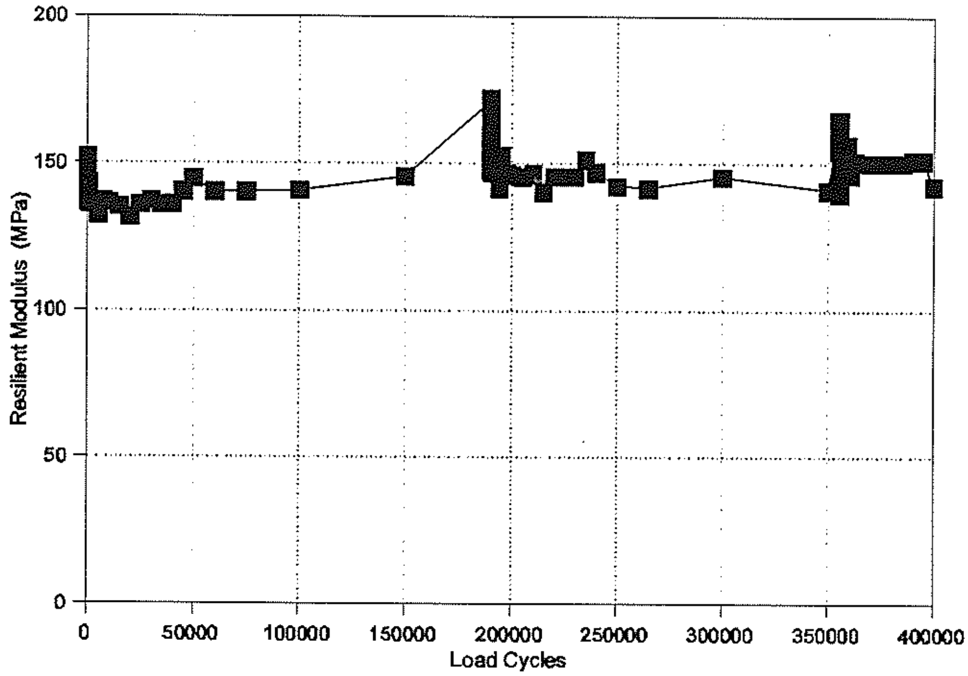


Figure A4 Plot of resilient modulus versus loading cycles for Tod Clay Specimen 1.

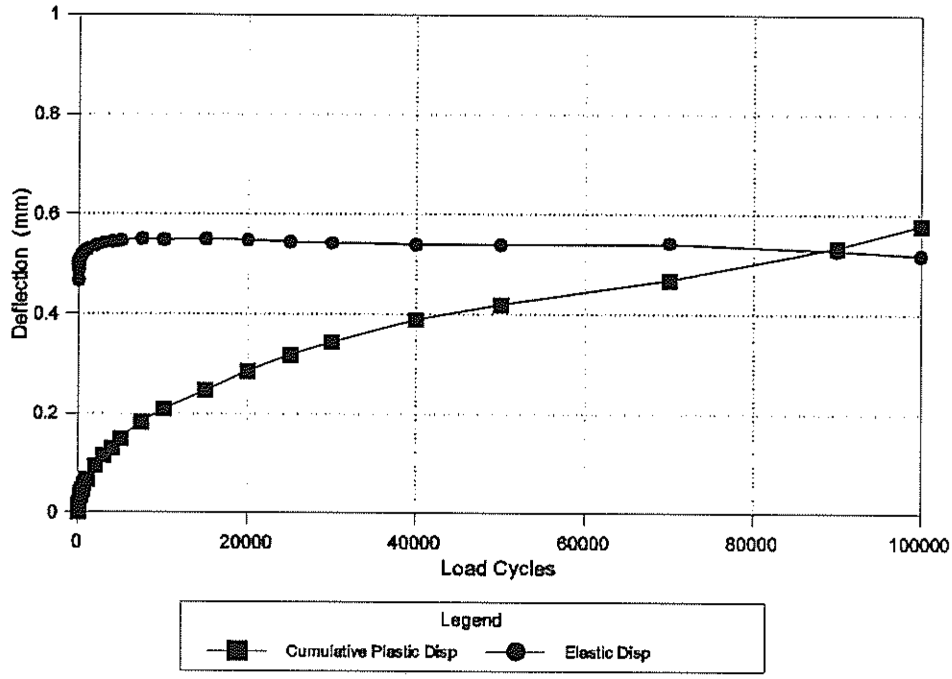


Figure A5 Plot of elastic and plastic deflection versus loading cycles for Tod Clay Specimen 2.

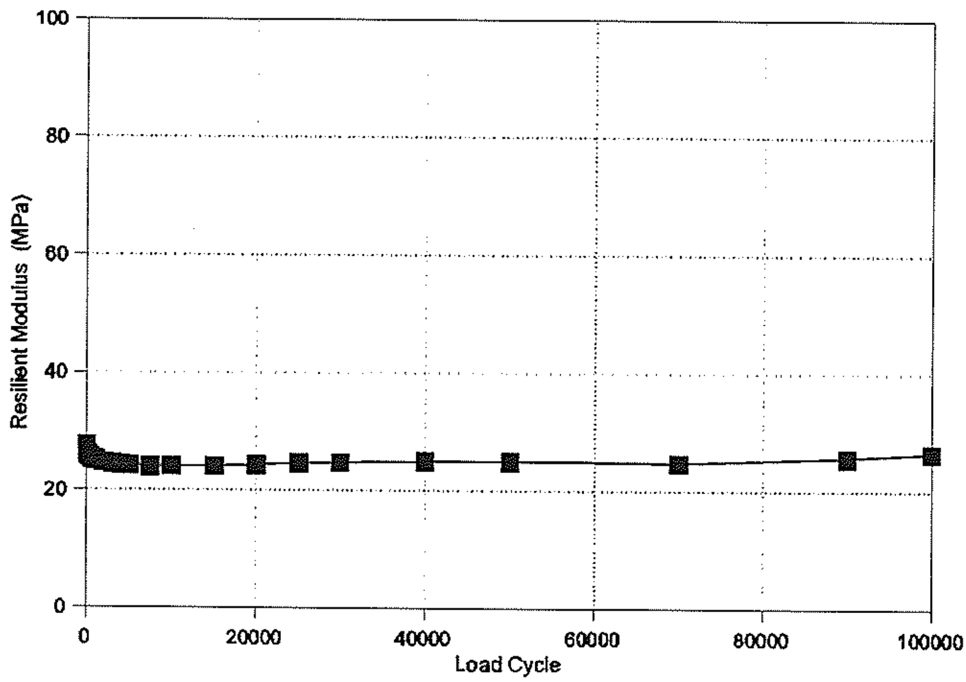


Figure A6 Plot of resilient modulus versus loading cycles for Tod Clay Specimen 2.

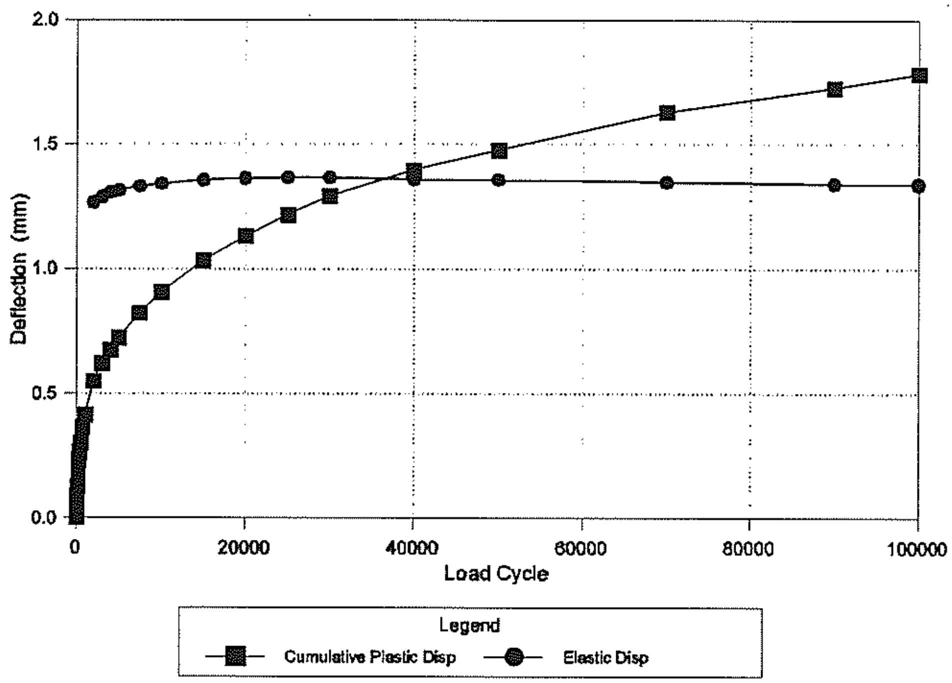


Figure A7 Plot of elastic and plastic deflection versus loading cycles for Tod Clay Specimen 3.

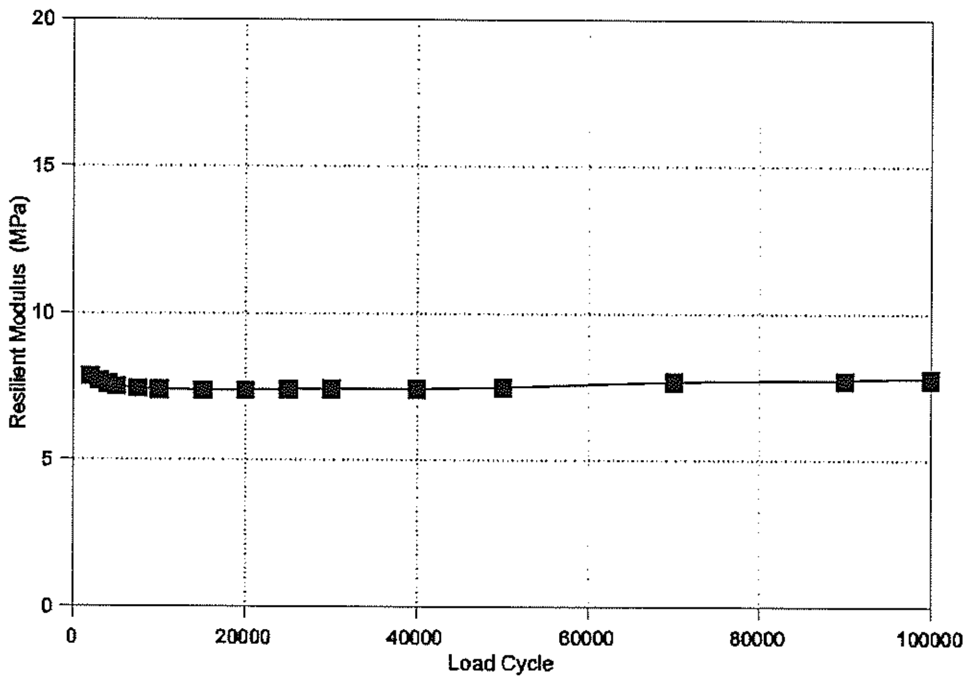


Figure A8 Plot of resilient modulus versus loading cycles for Tod Clay Specimen 3.

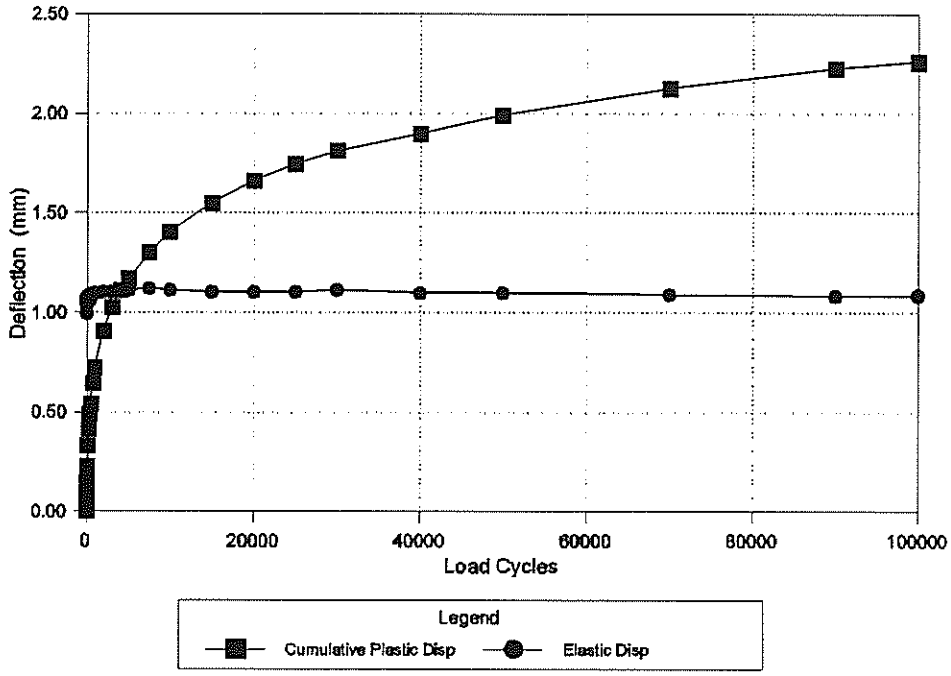


Figure A9 Plot of elastic and plastic deflection versus loading cycles for Tod Clay Specimen 4.

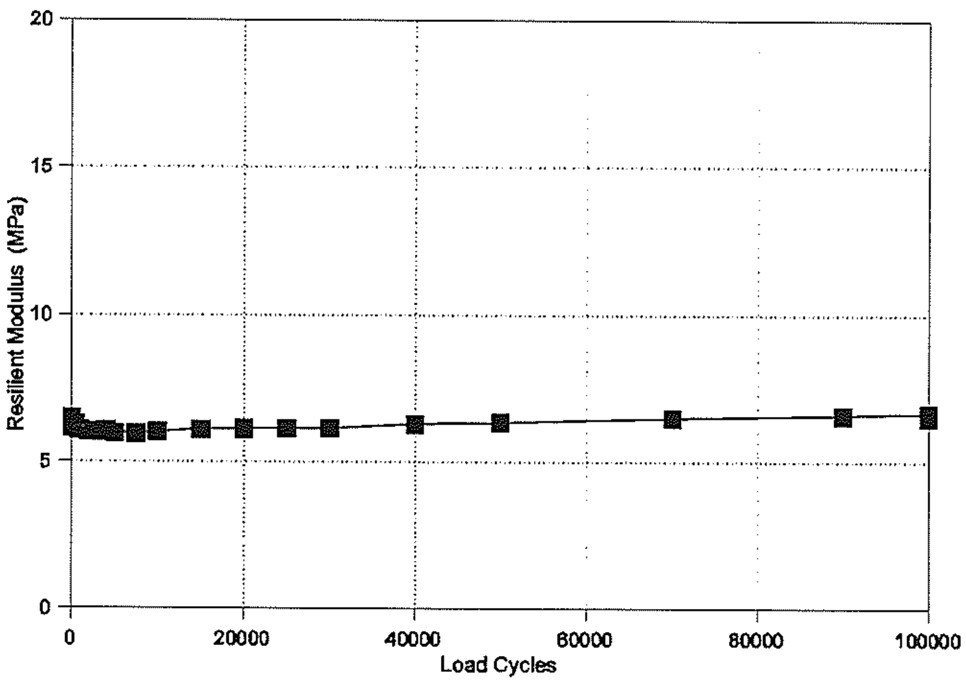


Figure A10 Plot of resilient modulus versus loading cycles for Tod Clay Specimen 4.

APPENDIX B

CAPTIF SUBGRADE CONSTRUCTION SPECIFICATION

CAPTIF SUBGRADE CONSTRUCTION SPECIFICATION

B.1 Introduction

The results of this project have confirmed the suitability of Todd Clay as a subgrade material suitable for use at CAPTIF. However there has been a need to improve the control on conditioning the soil before placing it in the concrete tank. This specification provides for that as part of the overall construction of the subgrade.

The subgrade layer should be at least 750 mm thick and needs to be of a uniform consistency, i.e. with regard to soil type, density, water content and resilient modulus.

Past experience with the construction of test pavements has indicated that subgrade soils tend to "work harden" under traffic (i.e. the stiffness of the soil, particularly weak soils, increases during the test period). To minimise this effect the soil should be compacted to maximum density at a moisture content wet of optimum.

Construction of a subgrade in the weak range involving a very wet soil may be particularly difficult to achieve. The water content must be uniform throughout the layer, otherwise problems will be experienced with soil weaving or sticking to the compaction equipment.

B.2 Procurement and Storage

The mass of soil necessary to form the test subgrade is estimated to be 300 tonnes (at 14% water content). This is calculated from a solid volume of 180 m^3 (median circumference of annular track 58.2 m, by 4 m track width, by 0.75 m depth of clay) at a dry density of 1.57 t/m^3 compacted within the test tank, plus a contingency of 5%.

The clay shall be stored on a concrete pad or other impermeable layer, and covered with plastic sheets or under a covered area.

B.3 Construction Procedures

In its natural state Tod Clay is lumpy and will need to be broken down to a relatively small size to ensure even distribution of the water that needs to be added to bring the soil to the target water content. A process should be established to ensure that the water content lies within the range plus or minus half of one percent of the target water content. A small pug mill could be useful to break the soil down and to facilitate the uniform increase in the water content.

Another option would be to break the soil down and mix water in using a rotary hoe. The soil should be stored under cover for a period to allow the water distribution to become uniform and mixed again before laying.

Normal fill construction procedures should be adopted wherever possible, with the objective of achieving the minimum variation in soil properties. A suitable statistically based test programme will confirm that the variation achieved is appropriate to the objectives of the project.

Before construction of the subgrade, the existing material in the track should be excavated to a depth of 1 m below the elevation of the track tank walls.

Lift layer thickness shall be between 150 - 175 mm. The clay shall be compacted with a self-propelled trench roller. Compaction should continue until the air voids content is less than 3%.

The top of the fill must be covered whenever necessary to reduce evaporation, and the water content and density should be continuously monitored. At the end of construction the density and water content values should be recorded over the complete area of the subgrade. Construction of the unbound granular cover layer should follow immediately or the surface should be covered or lightly watered to stop evaporation.

B.4 Testing

All tests must be in accordance with the relevant section of New Zealand Standard Specification 4402:1986 (SANZ 1986). Where no standard test exists, the test shall be carried out in accordance with the most relevant procedure available.

A 38 mm-diameter undisturbed core sample is to be taken from the subgrade near the centreline.

Tests shall be taken at 5 m intervals around the inside circumference of the track.

Tests to be undertaken include:

1. Loadman tests at the track centreline starting opposite station 0.00, and at 0.5 either side of the centreline. The electronics on the Loadman are to be reset for each drop and the four values on the display are to be recorded for each drop (modulus, deflection, deflection time and elasticity index). The test shall be repeated at each test site until the last three results are consistent (within a range of 6 MPa).

2. Four Shear Vane tests around the perimeter of, and immediately adjacent to, the three sites of the Loadman tests (provided the soil is not too hard).

3. In situ CBR tests on the track centreline, starting opposite position 1.00 at the surface only.

4. Dynamic Cone Penetrometer (DCP) tests to a depth of 0.9 m, on the track centreline immediately adjacent to each CBR test site .

5. Water replacement density tests at the track centreline starting at station 2.00 and at 0.5 m either side of the centreline.

6. Four Nuclear Density (NDM) tests by probe around the perimeter of, and immediately adjacent to, the three sites of the water replacement density tests.