Total Voids in Unbound Granular Pavements

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Total Voids in Unbound Granular Pavements

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Works Infrastructure provided data from its trials on the Albany Interchange section of the Upper Harbour Corridor and gave permission for the use of data from a report prepared for it on the Kamo Bypass.

Bloxam, Burnett and Olliver not only gave permission for the use of data from a report prepared for them on the Maungaiti Hill realignment project but also supported the research with additional information and comment.

Pavement Management Services carried out the highspeed data surveys at minimal cost even though the timing of the surveys was not always convenient to them.

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Executive summary

Excessive wheel path rutting due to post construction densification is a common problem that has occurred in a number of new pavements constructed in recent years. It is believed that much of the deformation could be avoided if the aggregate layers were compacted to a high level of density prior to the road being opened to traffic. This research project was designed to explore the factors that control the density of unbound granular aggregate and the influence that particle size distribution has on the density, on the compaction process and on the subsequent performance of a pavement.

In the initial phase of this project samples of basecourse from each of three different sources were prepared to match grading curves with "*n*" values varying between 0.25 and 0. 55 where "*n*" is the integer in:

$$p = 100\left(\frac{d}{D}\right)^n$$

Where *p* = percent passing sieve size *d* and
D = maximum particle size.

The optimum grading for maximum dry density was found to lie between 0.33 and 0.39.

Examination of the results of various tests on two existing pavements that showed excessive deformation (ruts) in the wheel paths soon after construction, indicated that densification of the basecourse was a major contributor but that the subbase could also densify under traffic. Densification was likely to occur when the Total Voids content in the basecourse at the end of construction was greater than 15%. Data from one other project where additional compaction using fully laden water carts was applied showed that while the additional work had a small effect on the density of the basecourse it caused a significant increase in the stiffness of the layer.

The results of two full scale pavement trials using more densely graded aggregate are described. One project was designed to evaluate the properties of an aggregate for use on a Motorway project while the other specifically studied the properties of three aggregates manufactured to different gradings.

In the first trial the normal product from the quarry was found to be difficult to lay and compact. A more densely graded product produced by additional crushing was laid on a section of Motorway on-ramp. The mean Total Voids achieved prior to sealing were 15% in the basecourse and 20% in the subbase, which was a substantial improvement over what was achieved with the normal products. The mean deformation that occurred in the first 20 months of trafficking was small, generally less than 10 mm.

In the second full scale trial the properties of three different gradings were compared. One was the normal quarry basecourse product which complied with the Transit New Zealand M/4 Specification. The other two gradings were produced in the quarry by

blending sand with the normal product. Mean rut depth values for all test sections after approximately 12 months trafficking were less than 5 mm. The most dense grading had the lowest Total Voids and smallest rut depth.

As a consequence of the research carried out in this project it is recommended that:

a) The limits for particle size distribution set out in TNZ M/4 be changed.
b) Quarry operators be permitted to add clean quarry fines or a suitable sand to ensure that their product fits well within the PSD limits and is as well graded as possible.
c) Use of the laboratory density test for the control of compaction should be discontinued.
d) Compaction of both subbase and basecourse should be controlled in terms of Total Voids.
e) Total Voids should be calculated using the Apparent Specific Gravity determined using ASTM C127:1980 and C128: 1980 as a reference.
f) Apparent Specific Gravity for each aggregate source should be available from the Quarry Operator.

Abstract

In New Zealand excessive deformation in the wheel paths due to post construction densification has occurred in a number of new pavements constructed in recent years. It is believed that much of the deformation could be avoided if the aggregate layers were to be compacted to a high level of density prior to the road being opened to traffic.

This research project was designed to explore the factors that control the density of unbound granular aggregate and the influence that particle size distribution has on the density, on the compaction process and on the subsequent performance of a pavement. It was carried out in the period between 2004 and 2007.

The results show that it is practical to produce basecourse with a dense grading by blending the normal quarry product with crusher dust or sand. Full scale trials showed that dense graded basecourse could be compacted to a Total Voids content of less than 15% and that the mean depth of ruts after twelve months trafficking was less than 5 mm.

1 Research Background

1.1 Introduction

The pavement for the majority of roads constructed in New Zealand comprise unbound granular aggregate covered with a bitumen and chip seal coat. In time these pavements commonly show depressions along each wheel path due to the compressive loads imposed by heavy vehicular traffic. These depressions are referred to as ruts.

The pavement commonly contains two layers. The "subbase" is the layer placed immediately on top of the soil subgrade. The layer on top of the subbase is called "basecourse". The terms "subbase" and "basecourse" may also be used to describe the quality of the aggregate.

Aggregate is manufactured from fresh rock or from river gravel by a process of crushing and screening designed to yield a product that fits within a particular grading envelope. For example, basecourse for State Highways in New Zealand must meet the requirements described by Transit New Zealand Specification M/4 *Basecourse Aggregate* (TNZ M/4).

The subbase aggregate is usually spread on the prepared subgrade to the specified thickness, watered to bring it to the required water content and then rolled with steel wheeled vibrating rollers. Once an initial density has been achieved the compaction is continued with the rollers operated in static mode. The objective being to compact the layer until the density of the aggregate meets the required level.Commonly the density requirement is specified as the proportion of that achieved in the laboratory. For example TNZ Specification B/2 *Construction of Unbound Granular Layers* (TNZ B/2) requires basecourse to be compacted to not less than 98% of the Maximum Dry Density determined using NZS 4402 1986:Test 4.1.3.

1.2 Purpose of this research

Excessive wheel path rutting due to post construction densification is a common problem that has occurred in a number of new pavements constructed in recent years. In most cases the surface roughness has increased, the waterproofness of the surface has been compromised and in some cases the safety of users has been threatened. Examples that the researcher is aware of include SH1 Maungaiti Hill (Waikato), SH 1 Pokeno By-pass (Waikato), PJK Expressway (Bay of Plenty), SH 1 Rangiriri to Ohinewai (Waikato) and SH1N Kamo Bypass (Northland).

Insufficient compaction also means that the resilient modulus of the aggregate layer is less than assumed in the design and could mean that the subgrade is overstressed at least during the initial stage of trafficking.

The cost consequences of premature damage to the pavement vary from site to site and have not been quantified. However, the cost to the constructor, the roading authority and the road users must be substantial.

It is believed that much of the deformation could be avoided if the aggregate layers were to be compacted to the highest possible level of density prior to the road being opened to traffic. Further, it is apparent that the standard laboratory test referred to in TNZ B/2 does not necessarily provide a measure of the highest possible density. The reasons for this are:

a) The NZS laboratory test used as a reference is subject to wide variation because of operator and other influences;
b) results depend on the grading of the sample used;
c) constructors are permitted to control compaction on the basis of the results of only one laboratory density test;
d) compaction method used in the laboratory test is not related to the type of plant used in the field.

An alternative is to use the Total Voids (TV) content in the compacted layer. Voids are the spaces between individual particles of rock that make up the layer and in some cases the vesicles within a rock particle (e.g. in volcanic rock). Total Voids is the sum of these spaces.

Total Voids is defined as:

$$TV = \left(1 - \frac{\gamma_d}{\gamma_s}\right)100$$

where γ_d = dry density in t/m^3

γ_s = Apparent Specific Gravity in t/m^3

The Apparent Specific Gravity (ASTM Des C 127 & 128) is used because it takes account of the vesicles and provides the most appropriate measure of the overall dry mass of rock in the system. It should be noted that the value for a rock type from a particular area is constant and will only vary depending on the amount of weathering that has occurred in the rock. For example, the value for Auckland Basalt is 3.03 while that for Drury Greywacke is 2.73. Jennings DN, Cunningham JK and Thrush M (1998) provide a more detailed discussion of this topic.

There are a number of aspects that influence compaction of unbound granular materials including:

1. stiffness of the underlying layers;
2. particle size distribution of the aggregate;
3. surface roughness/friction of the rock particles;
4. crushing resistance of the aggregate;
5. shape of the particles;
6. number of crushed faces;
7. fines content;
8. water content of the aggregate;
9. type of compaction equipment used;

10. energy input during compaction.

The underlying layers must be as stiff as possible but all the other aspects must be at optimum levels to ensure efficient compaction to maximum density.

The development of the standard Basecourse Aggregate and Construction specifications provided by Transit New Zealand have been based on the philosophy that the aggregate layer should be free draining to avoid pot holing or the development of positive pore water pressures. There has been a concern that a dense basecourse layer does not facilitate drainage. However, in practice a new layer of basecourse that is not at a maximum level of density when opened to traffic will densify under the wheel loads applied by modern heavy transport vehicles.

The risk of early rutting in the aggregate layers can therefore be minimised by compacting the layers to a low level of TV prior to sealing. This would require current construction procedures to be modified so that:

i) the Particle Size Distribution (PSD) of the aggregate is as close as possible to the dense side of the TNZ M/4 envelope;
ii) an appropriate density requirement in terms of TV is provided in the construction specification; and
iii) the aggregate is maintained as close as possible to optimum water content during compaction.

1.3 Project Objectives

The objectives of this project as described in the project brief are:

a) To determine the optimum particle size distribution for maximum density and optimum water content for a selection of aggregates.
b) To identify the mechanism of rut development in new pavements by studying the changes in the characteristics of selected pavements.
c) To review available data to determine practical TV limits for construction.
d) To incorporate the available results from this project into a full scale construction project as far as that is possible and to monitor the results.
e) To analyse the results and prepare a draft report.

The procedures to be followed are discussed in the following sections.

1.3.1 Optimum particle size distribution.

The PSD of a well graded aggregate can de described as:

$$p = 100\left(\frac{d}{D}\right)^n$$

Where *p* = percent passing sieve size *d*
D = maximum particle size
n = an integer which commonly has a range between 0.3 and 0.6.

The current TNZ M/4 envelope was originally based on the range of product available in New Zealand. Most suppliers can produce basecourse that fits or nearly fits within the prescribed limits but the material is seldom well graded. The harder rock types e.g. greywacke, do not readily produce adequate quantity of fine particles. The PSD of such material tends to fall outside the 0.4 limit for large particles and to generally lie close to the coarse limit (n = 0.6) for the remainder. TNZ M/4 needs to be reconsidered in order to provide an envelope more suitable for pavement construction and to give the supplier guidance on how the product can be improved.

The requirement that the aggregate be "well graded" needs some explanation. Such aggregates have a PSD (also referred to as "grading curve") that plots as a straight line on a log-log graph. Theoretically the most dense arrangement of spherical particles follows a grading for which n = 0.5. However, the crushing and screening procedures currently used in the quarrying industry seldom results in a "well graded" product. Improvement to the shape of the grading curve to more closely follow a straight line, would generally involve further crushing or blending.

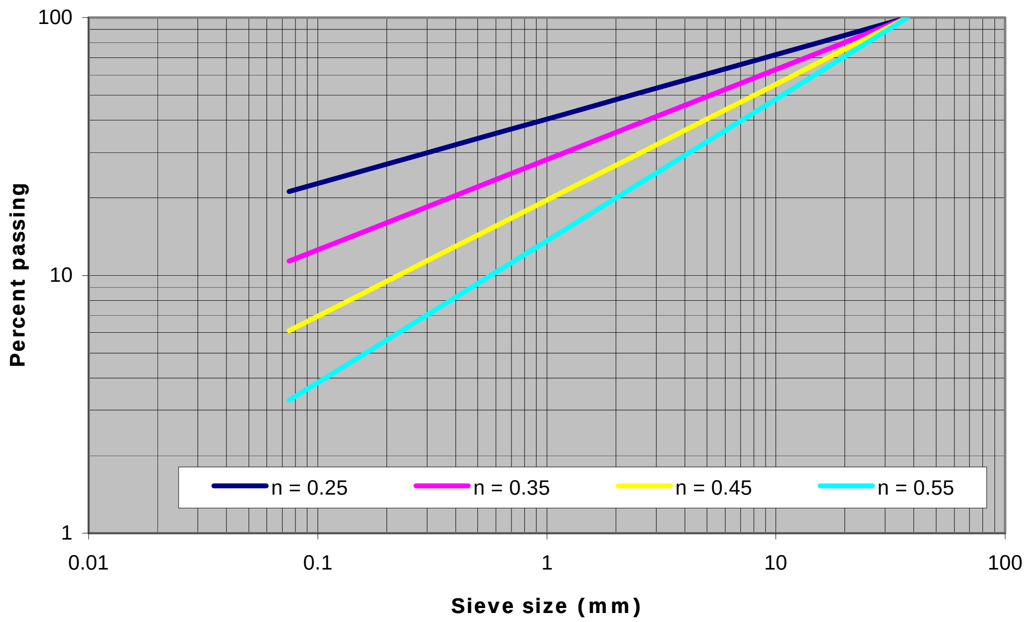

Figure 1.1 Ideal grading curves.

1.3.2 Rut development in new pavements

Rutting is believed to be related to densification and shear movements within a particular layer. Premature rutting occurs within the early life of the pavement typically within the first year.

The first part of this investigation will involve a review of existing data that describes the initial condition of three pavements that suffered excessive rutting once they were put into service. Existing QA and other test data will be analysed to determine start PSD, TV and water content. In some cases measurements of rut depth and the density and water content of the basecourse in the outer wheel path and between wheel paths after trafficking also exists and will be reviewed. The data from these projects will be compared to find common factors associated with post construction densification.

1.3.3 Practical limits

Any proposed changes to specified requirements that result from this research must be practical and relatively easy to implement. All the available data will be reviewed to determine the extent of change necessary.

Compaction data will be analysed to determine the significance of:

i) particle size distribution of the aggregate;
ii) type of compactor;
iii) trafficking (where available).

The maximum density achieved with respect to the PSD of the aggregate used will be determined. Practical limits for the acceptable PSD, TV and water content at the end of compaction will be established and these will be discussed with contractor organisations.

1.3.4 Trial pavement

Staff at Works Infrastructure (Auckland) were interested in the research and provided data on similar work that they had carried out as part of one of their projects. Fulton Hogan (Waikato) offered to include the construction of test sections in a new pavement subject to time and cost limitations.

2 Particle Size Distribution

2.1 Introduction

Three laboratory studies to determine the optimum grading were carried out, two in Christchurch and one in Hamilton. Both laboratories were owned by Fulton Hogan Ltd. The Christchurch laboratory tested greywacke aggregate from Canterbury and from Auckland. An andesite from the Hauraki Plains area was tested at the Hamilton laboratory.

The standard M/4 aggregate produced in each quarry was dried and sieved into specific particle size ranges. A minimum of six samples with a specific grading integer (n) were produced by recombining the sieved materials and adding quarry dust or other fine grained material derived from the same source. Sufficient water was added to each of four samples to ensure an appropriate range of water contents was tested. Each sample was then placed in a mould and compacted in accordance with NZS 4402 Test 4.1.3 (Vibrating Hammer). Once the compaction curve had been defined the water content of the remaining two samples was adjusted to the optimum value. These samples were compacted using the vibrating hammer compaction method. The average of the three values of dry density was taken to be the maximum dry density for that grading integer.

The three samples used in the density test at OWC were then combined and used to determine the particle size distribution using NZS 4407 Test 3.8.1 to confirm the grading integer value. The mean of the values of the grading integer at each sieve size was taken to be the actual "*n*" value for that grading.

The results are described in the following sections.

2.2 Canterbury Greywacke Aggregate

The results of the compaction tests on a Canterbury Greywacke aggregate are summarised in Table 2.1 and Figure 2.1. The Solid Density of the aggregate was assumed to be 2.76.

Table 2.1 Maximum dry density versus Grading Integer (n).

n	Maximum Dry Density (t/m^3)	Total Voids (%)	Optimum Water Content (%)
0.25	2.26	18	6.0
0.35	2.35	15	4.5
0.44	2.34	15	4.8
0.54	2.21	20	4.5

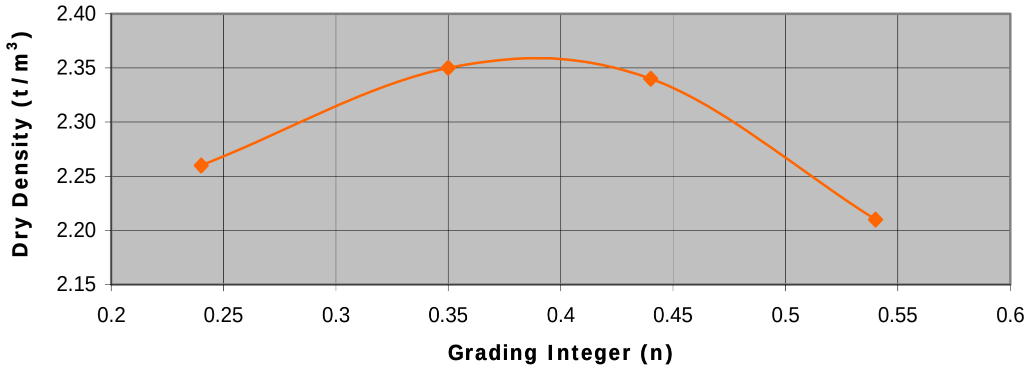

Figure 2.1 Canterbury Greywacke.

2.3 Auckland Greywacke Aggregate

The results of the compaction tests on an Auckland Greywacke aggregate are summarised in Table 2.2 and Figure 2.2. The Solid Density of the aggregate was assumed to be 2.76.

Table 2.2 Compaction characteristics.

n	Maximum Dry Density (t/m^3)	Total Voids (%)	Optimum Water Content (%)
0.24	2.33	16	5.5
0.35	2.44	12	4.3
0.45	2.37	14	4.5
0.55	2.28	17	4.5

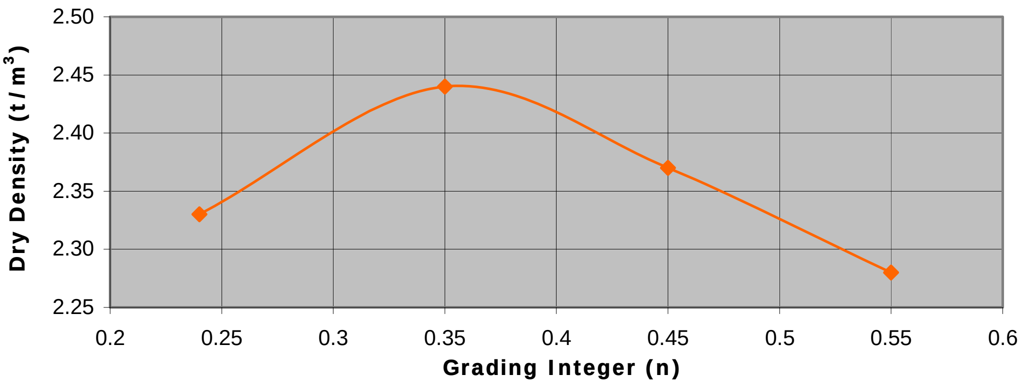

Figure 2.2 Auckland Greywacke.

Hauraki Andesite

The results of the compaction tests on the Hauraki Andesite aggregate are summarised in Table 2.3 and Figure 2.3. The Solid Density of the aggregate was measured using NZS 4407: Tests 3.7.1 and 3.7.2 and reported to be 2.76.

Table 2.3 Maximum dry density versus grading integer (n).

n	Maximum Dry Density (t/m^3)	Total Voids (%)	Optimum Water Content (%)
0.24	2.25	18	6.9
0.33	2.31	16	5.8
0.43	2.25	18	6.6
0.52	2.13	23	3.5

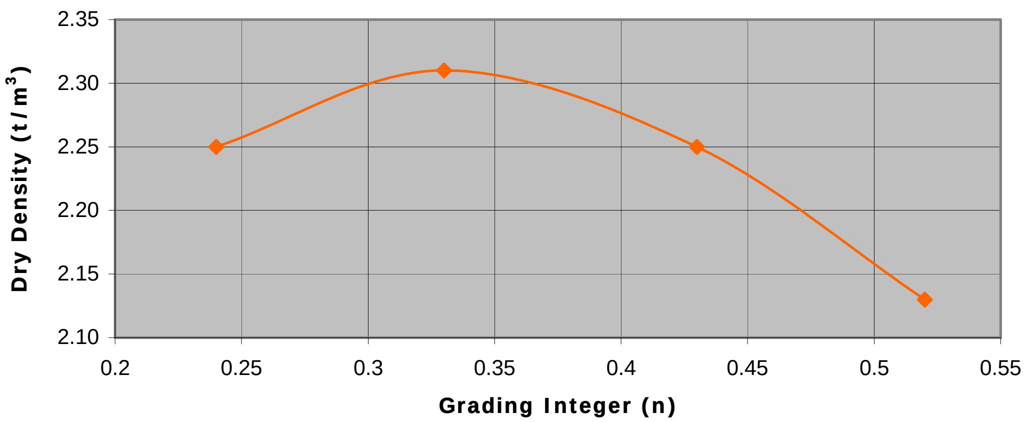

Figure 2.3 Hauraki Andesite.

2.4 Summary

The results of the laboratory compaction test using the vibrating hammer compactor are summarized in Figure 2.4. This shows that the optimum grading for maximum density lies between 0.33 and 0.39 and this infers that the aggregate should have a high proportion of fines (12% passing 75 microns for *n* = 0.35). This is significantly greater than the maximum of 5% permitted by the current Transit NZ M/4 Basecourse Specification.

The Total Voids content of these aggregates when compacted to maximum dry density varied between 12 and 15 percent. But note that in each case the laboratory calculated the voids using a measured or assumed value for the Solid Density determined using NZS 4407:1991, Test 3.7 as specified in TNZ B/2/:2005.

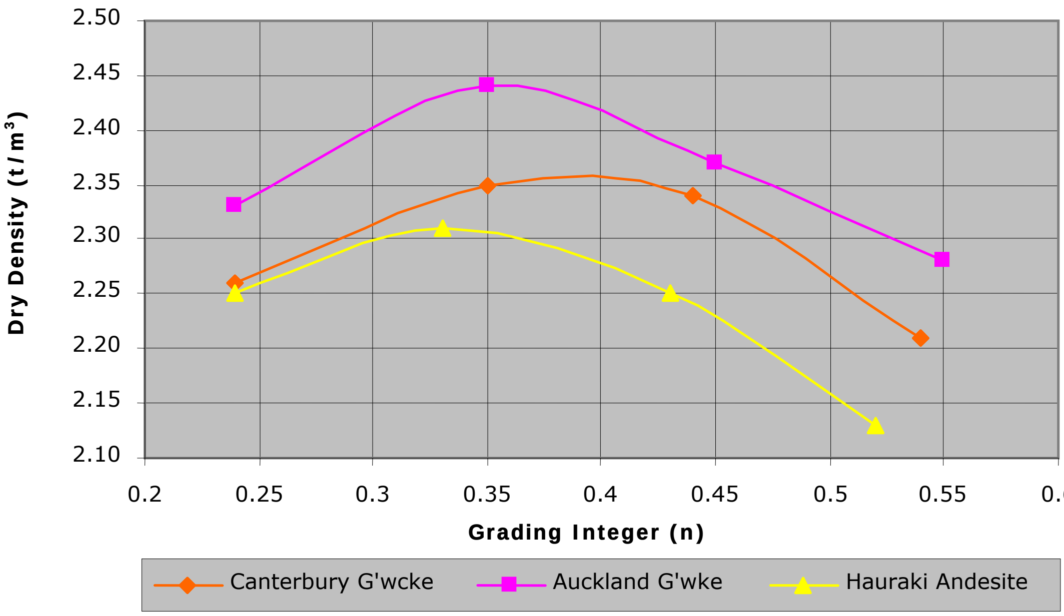

Figure 2.4 Summary of results.

3 Rut Development in New Pavements

3.1 Introduction

The objective of this section of the research was to explore the information gathered from site investigations carried out to determine the cause of excessive permanent deformation in the wheel paths of two newly constructed pavements and the outcome of special compaction applied to another.

3.2 SH1 Maungaiti Hill

3.2.1 Introduction

Construction of a 6 km re-aligned section of SH1 RP 548/13.30 -19.30 at Maungaiti Hill was completed in May 1996. The road pavement comprised 230 mm basecourse, 170 mm subbase, 400 mm subgrade improvement layer and a subgrade with a CBR 5%. Soon after the road was put into service deep ruts (25 – 35 mm) formed in the outer lanes and to a lesser depth in the northbound passing lane.

An investigation was carried out by the Researcher in 1996 to determine the cause of the deformation.

3.2.2 Construction requirements

The documents for the Maungaiti Hill Contract required that the aggregate layers be constructed in accordance with the then current Transit NZ Specification B/2. This required that the "plateau density" be established for each type of aggregate used. The subbase and basecourse layers were then to be compacted to a specific proportion of the plateau density with the selected rollers. Secondary compaction was to be provided by normal vehicular traffic under controlled conditions.

The Documents went on to modify TNZ B/2 to improve the quality of the work. For example:

- all subbase and basecourse aggregate was to be paver laid;
- primary compaction was reached when the dry density was not less than 98% of the maximum dry density achieved in the "plateau density" test;
- pneumatic tyred rollers were to be used for secondary compaction;
- Total Voids content of the compacted subbase was to be not less than 10% nor more than 25%;
- prior to sealing the Total Voids content of the basecourse was to be not less than 8% nor more than 20%;
- bulk specific gravity determined using ASTM C127: 1980 and C128: 1979 was to be used to calculate the Total Voids;
- uniformity of compaction was to be checked using a Clegg Hammer or a Nuclear Densimeter. The minimum value was to be not less than 90% of the mean and the CIV prior to sealing was to be not less than 75.

The aggregate chosen for this project was a vesicular Basalt, ex Watts Quarry. This material was considered to be preferable to the local Rhyolite which was a much softer rock.

3.2.3 Density measurements

The result of the plateau density test on the second layer of subbase showed that a maximum dry density of 2190 kg/m^3 was achieved after 8 passes with a vibrating roller followed by 6 passes with a static steel-wheeled roller.

The contractor proceeded to construct the aggregate layers in accordance with the requirements of the specification. Secondary compaction was applied until the Total Voids values were within the required range indicating that the basecourse met the contract requirements. However the Contractor used a specific gravity (2.68) determined using NZS 4407:1991 Test 3.7.1 to calculate the Total Voids, instead of the ASTM tests specified. One other discrepancy occurred at the same time. The values achieved when the basecourse was tested with the Clegg Hammer were predominantly in the range 50 – 60; well below the minimum value of 75 that was specified.

The results of a specific gravity test carried out in January 2005 on a sample of rock from the quarry are summarised in Table 3.1. The test complied with ASTM C127 and C128.

Table 3.1 Results of specific gravity test on rock ex Watts Quarry.

Type of test	Result
Bulk specific gravity oven dry	2.78
Bulk specific gravity saturated surface dry	2.85
Apparent specific gravity	2.97
Absorption	2.2%

The Total Voids content of the basecourse based on the results of density measurements taken prior to sealing are plotted in Figure 3.1. The Total Voids were calculated using the Apparent Specific Gravity (AppSG) from Table 3.1.

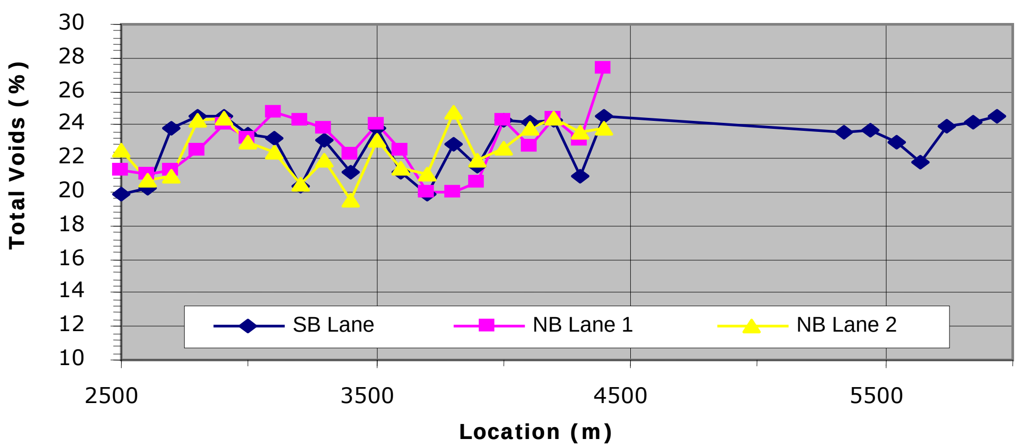

Figure 3.1 Maungaiti Hill basecourse.

These results indicate that the Total Voids in the basecourse were generally above the maximum specified value of 20% prior to sealing.

3.2.4 Investigation

Three trenches were excavated across the southbound lane in December 1996. The aggregate layers were removed over a width of approximately 300mm to uncover the top of the pumice subgrade improvement layer. A nuclear densimeter operated in direct transmission mode was used to measure the density in the basecourse layer after the seal was removed. Clegg CIV measurements were also taken in the same area.

The dry density, water content, rut depth and Clegg CIV value measured are summarized in Table 3.2.

Table 3.2 Measurements in test pits.

Test Pit	Transverse position	Dry Density (kg/m^3)	Water content	Rut depth (mm)	Clegg CIV
1	BWP	2336	3.6		66
	OWP	2384	3.7	23	64
2	BWP	2323	3.7		74
	OWP	2357	4.0	18	58
3	BWP	2266	4.1		50
	OWP	2343	3.8	30-35	62

BWP = between wheel path
OWP = outer wheel path

Note that the water content values recorded are within the range and close to the mean value of 4.0% measured immediately prior to sealing.

Samples of the basecourse aggregate were sent to the laboratory for PSD, Sand Equivalent (SE) and Crushing Resistance tests.

A stringline was used to check the shape of the interface between the subgrade improvement layer and the subbase and also to measure the depth of the surface rut. There was little or no unevenness in the surface of the subgrade improvement layer and it was concluded that all the densification was confined to the aggregate layers. However it was not possible to ascertain which aggregate layer had densified the most because the aggregate used in the subbase and basecourse was from the same quarry.

The density in the OWP at the test pits was compared with the density prior to sealing at the test pit location or at an immediately adjacent section. These results are summarized in Table 3.3. The calculated rut depth is based on an aggregate layer thickness of 400 mm and an average rut width of 500 mm.

Table 3.3 Estimated depth of rut based on the change in density.

Location	Dry density (kg/m³)		Ratio of densities	Rut depth (mm)	
	Initial	After trafficking		Calculated	Measured
3300	2280	2355	0.968	26	20
3400	2340	2355	0.994	5	19
3900	2280	2385	0.956	35	33
5740	2260	2345	0.964	29	30 – 35*
5840	2250	2345	0.959	32	30 – 35*

* = estimated rut depth

3.2.5 Aggregate properties

The particle size distribution of the basecourse aggregate was monitored on a regular basis between early November 1995 and mid March 1996. The results of Particle Size Distribution (PSD) tests on samples taken from stockpiles and from the road prior to compaction are shown on Figure 3.2 in comparison with the TNZ M/4 limits. The samples generally met the TNZ M/4 requirements apart from having slightly more passing 19 mm. The PSD results in the mid range in particular plotted towards the "coarse" side of the envelope.

The aggregate used had a crushing resistance value in the range of 9 – 10 % at 130 kN indicating that it was relatively "soft" and easily crushed. The particle size distribution of the samples recovered from the investigatory trenches (Tr 3320 1, Tr 3320 2 & 5900) shown in Figure 3.3 contain more fines than the stockpile samples indicating that some breakdown occurred during compaction or as a result of trafficking.

The results of Sand Equivalent tests during production generally lay in the range 47 - 77 with a mean value of 60. The SE values for samples recovered from the investigatory trenches were in the range 39 - 72 with a mean of 54 indicating a small increase in clay size fines.

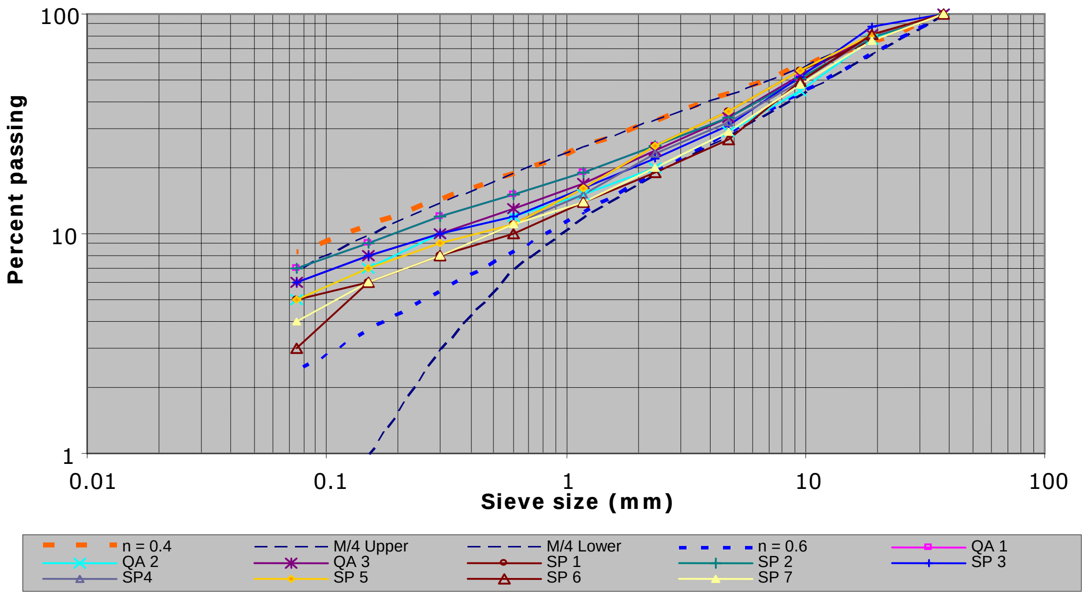

Figure 3.2 Maungaiti Hill AP 40 ex stockpile and road.

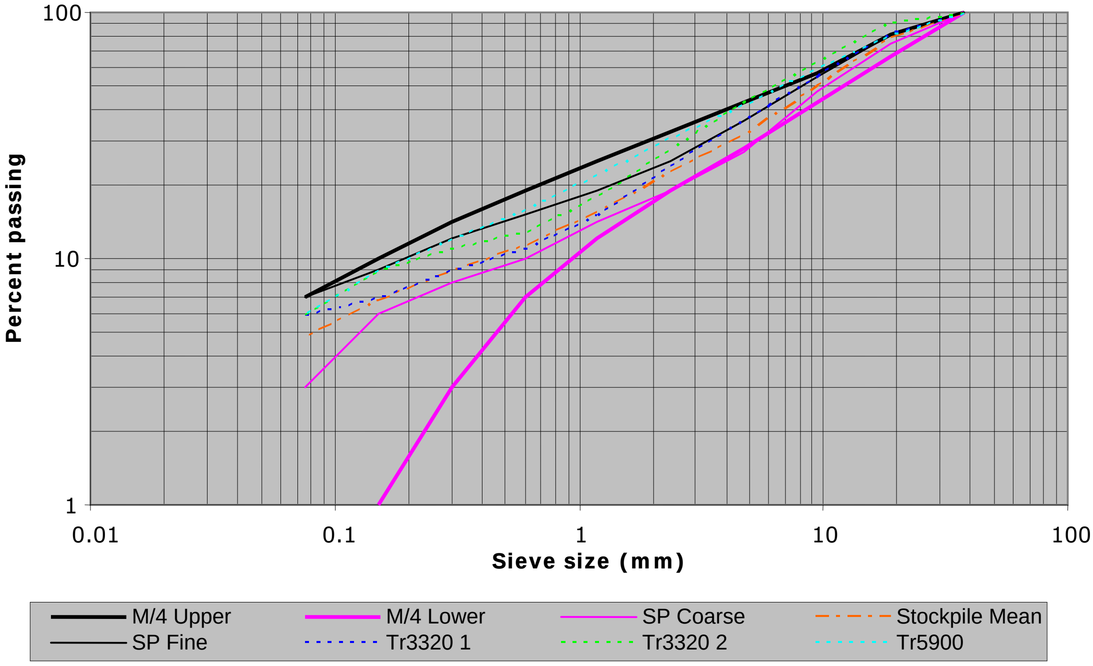

Figure 3.3 Particle size analysis of basecourse samples.

3.2.6 Cause of deformation

The ruts that developed in the wheel paths soon after the road was put into service were primarily due to densification within the aggregate layers.

Program CIRCLY was used to ascertain where most of the densification was likely to have occurred on the assumption that it would be related primarily to the normal vertical. The validity of this approach may be queried but the analysis indicated that the mean normal vertical stress in the basecourse layer was approximately 69% of the applied stress and that in the subbase was approximately 23%. It was concluded therefore, that most of the densification occurred in the basecourse layer. This is supported by the difference between the density of the basecourse in the outer wheel path and between the wheel paths shown in Table 3.2.

3.2.7 Discussion

The specification for the construction of the pavement included a number of requirements in addition to those in the then current edition of TNZ B/2. In particular it:

- provided for paver laying to reduce segregation,
- required a pneumatic tyred roller to be used for secondary compaction,
- set limits for the Total Voids prior to sealing and
- required uniformity of compaction.

In spite of that significant rutting occurred soon after the pavement was put into service.

While the quality assurance data indicated that the pavement met all the specified requirements except for the Clegg CIV, it has since been determined that the Total Voids calculations were based on an incorrect Specific Gravity test. The rut depth calculated from the change in density that occurred appears to provide a good estimate of the actual rut depth.

The water content of the basecourse did not change from that recorded during construction.

This project showed that "soft", coarse graded basecourse aggregate with a Total Voids content in excess of 20% prior to sealing could quickly develop deep ruts.

3.3 Kamo Bypass

3.3.1 Introduction

The Kamo Bypass which is located just north of Whangarei, was a greenfields project opened to traffic in late 2000. Soon after opening wheel track ruts developed predominantly in the southern third of the new 3.4 km long pavement. A site investigation that included the excavation of five transverse trenches was carried out in early 2002.

3.3.2 Rut depth measurement

The results of a high speed rut depth survey carried out in April 2002 and repeated in December 2002 are set out in Table 3.4 and illustrated in Figure 3.4.

Table 3.4 Rut depth measurements from high speed surveys.

Trench	Southbound				Northbound			
	LWP or OWP		RWP or IWP		RWP or IWP		LWP or OWP	
	April	Dec	April	Dec	April	Dec	April	Dec
2380	2	4	7	10	3	4	10	14
2480	3	10	18	17	4	4	14	17
3000	6	9	17	19	2	3	14	16
3100	2	6	10	10	4	4	4	8
3125	5	3	5	17	4	10	3	8

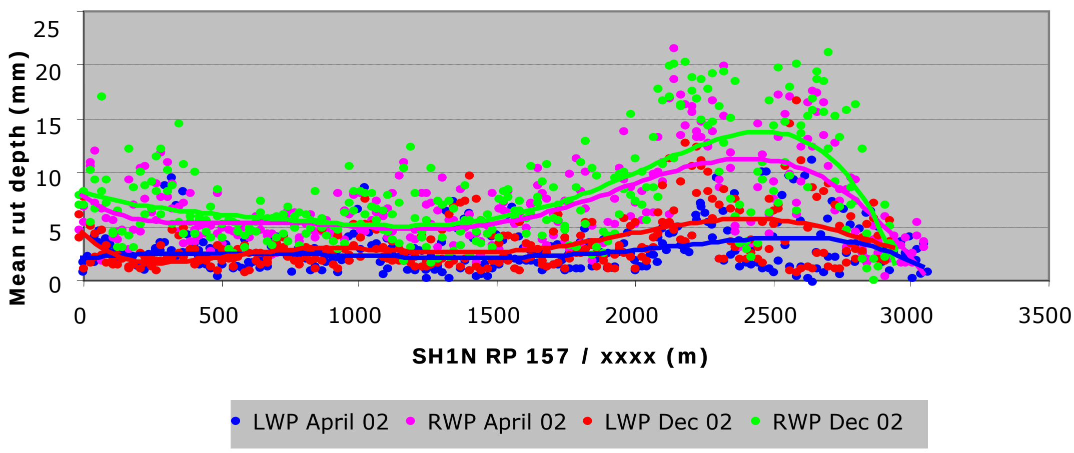

Figure 3.4 Rut depth data.

This analysis shows that a significant increase in rut depth occurred between the two sets of readings and that the most seriously affected area was between 1,800 and 2,800 m.

3.3.3 Site Investigation

Five trenches were excavated across the whole width of the pavement in April 2002. They were at 2380, 2480, 3000, 3100 and 3125. The thickness of each of the structural layers, the density and water content of the basecourse and the subgrade improvement layer were recorded and samples of basecourse and subbase were collected. The density of the subbase was not measured. Figure 3.5 shows the variation in the thickness of the layers and the Total Voids in the basecourse as measured in the trench at 3000 m. The plots for the other trenches. are similar except that the variation in the Total Voids is less pronounced.

The mean Total Voids in the basecourse aggregate in the lightly loaded sections of the pavement (i.e. shoulders, between wheel paths and on the centerline) for the five trenches, varies between 21 and 23%. The mean water content in the basecourse was 4.8%. The Total Voids in the wheel paths when the trenches were excavated typically varied between 15 and 19%.

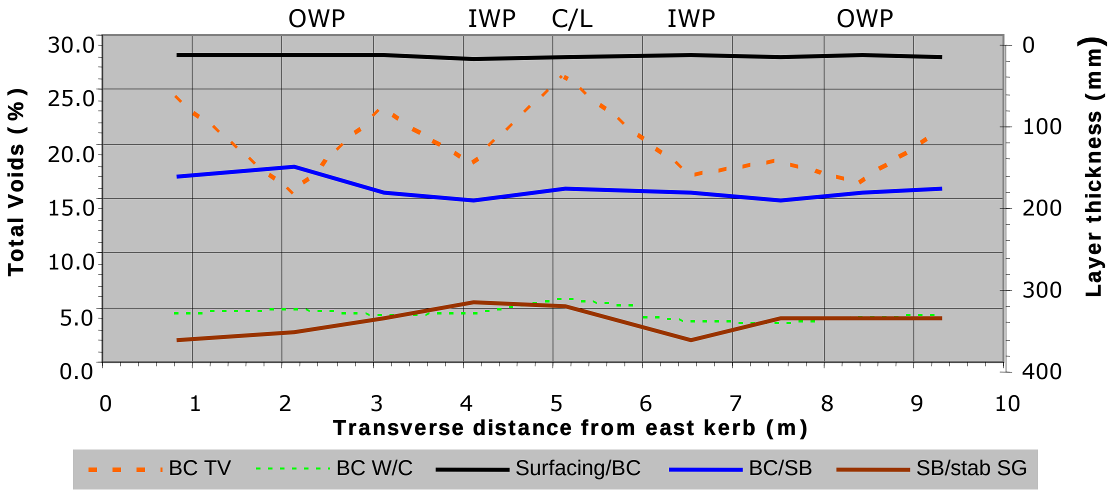

Figure 3.5 Trench 3000.

The measured rut depth deduced from the trenches has been compared with the estimated densification of the basecourse and subbase layers in Figure 3.5. For this figure the rut depth was estimated from the measurements taken in the trench (i.e. the average thickness of the surfacing either side of the wheel path) plus the deformation in the surface recorded during the high speed rut depth survey. The total rut depth was considered to be the sum of the densification in the basecourse and that in the subbase. However, no density measurements were made in the subbase so the deformation was assumed to be the difference between the thickness of the subbase layer outside the wheel path and that within the wheel path.

The trend lines indicate that a major part of the rut depth is due to densification in the subbase and that the best fit involves densification within both the basecourse and the subbase. Unfortunately there is no record of the Total Voids content in the subbase layer.

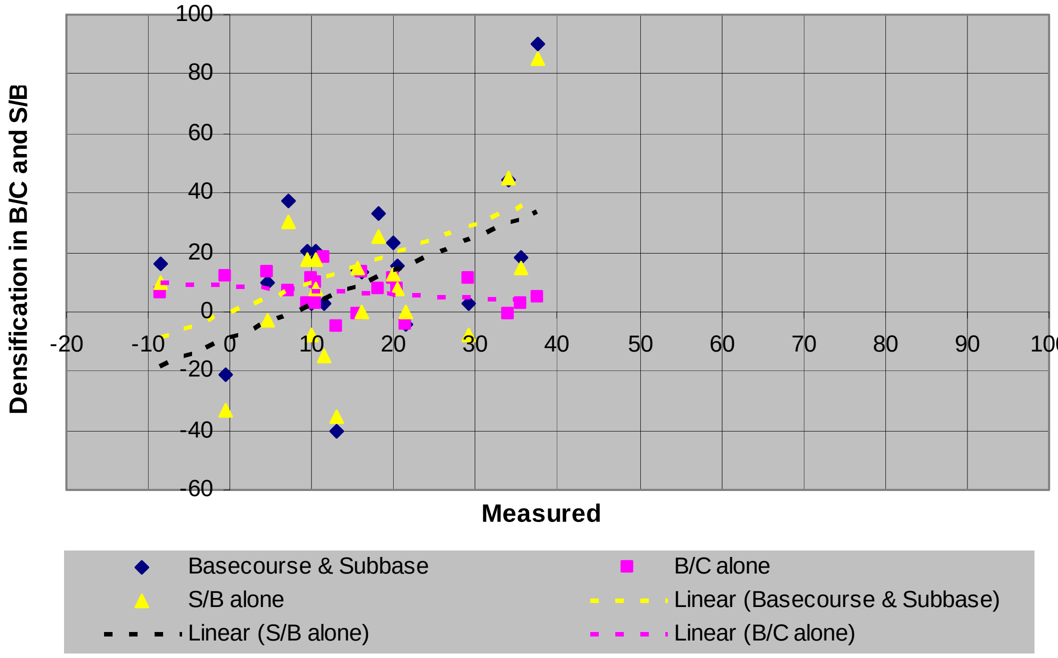

Figure 3.6 Estimated and measured rut depth.

3.3.4 Aggregate

The aggregate was a TNZ AP40 manufactured from Greywacke rock. Samples recovered from the trenches were tested for compliance with TNZ M/4 Specification. The samples had a mean Sand Equivalent of 25, a Clay Index of 2.6 and a Solid Density of 2.65. The results of PSD analysis are shown in Figure 3.7.

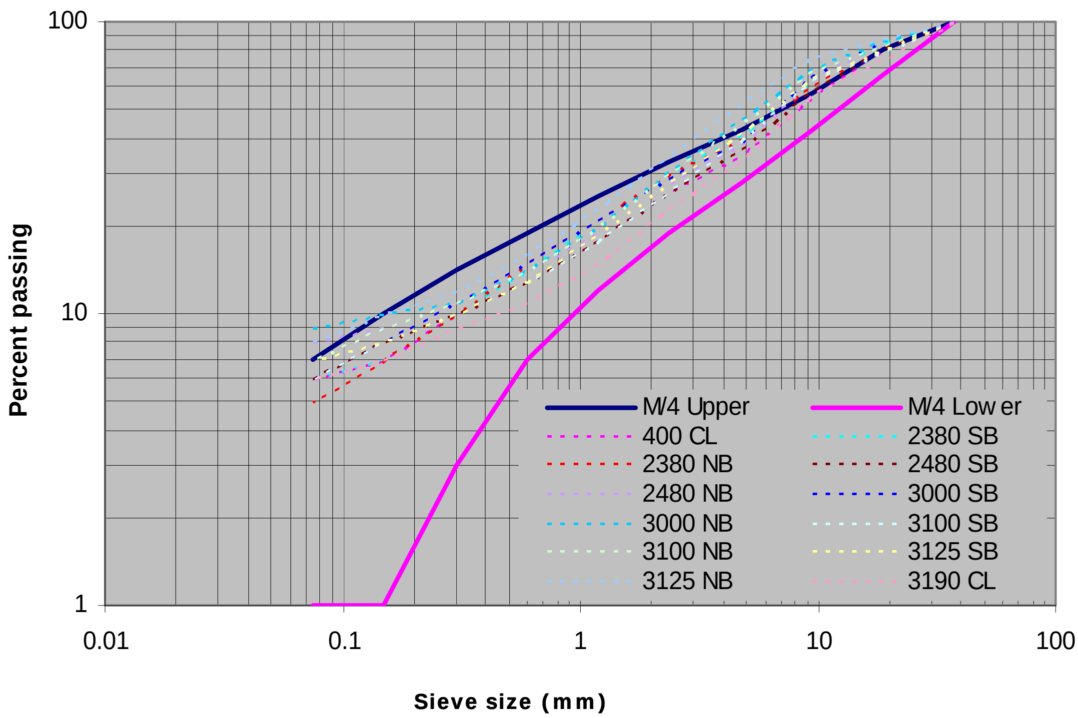

Figure 3.7 AP 40 curves.

The grading integer "*n*" has been calculated for each sieve size and is illustrated in Figure 3.8. This shows that:

a) the aggregate could not be considered to be "well graded" because the integer varies with each the sieve size (instead of being constant);

b) coarse fractions (4.75 to 19 mm) have integer values in the range 0.27 to 0.43;

c) fine fractions (< 2.36) generally have an "*n*" value equal to or greater than 0.45.

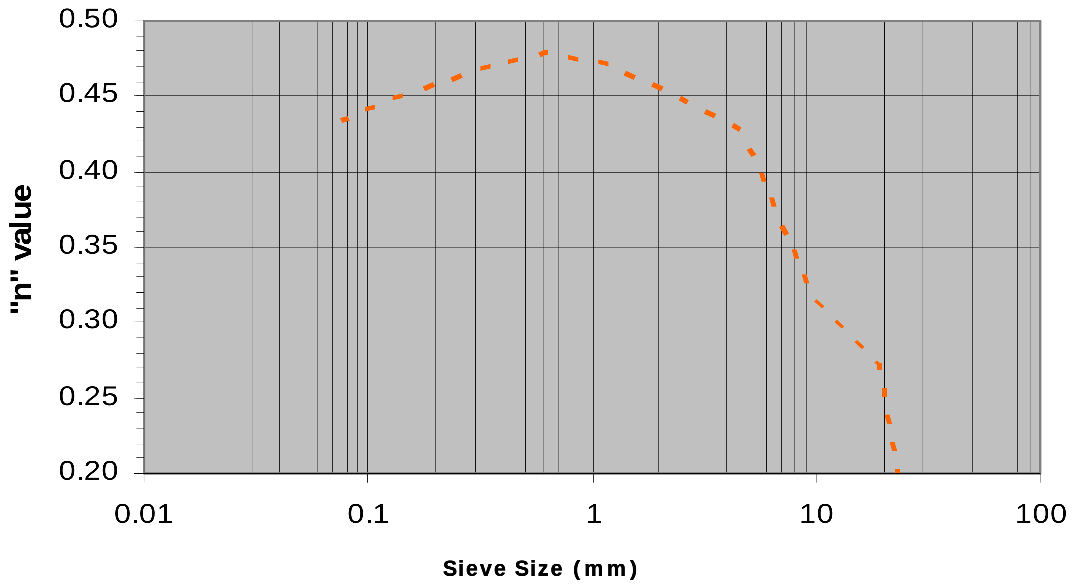

Figure 3.8 Variation in "*n*" value.

3.3.5 Conclusion

The main cause of the permanent deformation in the surface of the Kamo Bypass pavement was densification within the subbase and the basecourse layers after the road opened to traffic.

The Total Voids content of the basecourse as measured between the wheel paths in the trenches excavated across the pavement, was between 21 and 23%. It is likely that the pavement prior to sealing had a similar voids content. The Total Voids in the wheel paths when the trenches were excavated typically varied between 15 and 19%.

3.4 SH1 Hampton Downs Interchange

3.4.1 Introduction

The four laning of SH1 between Mercer and Rangariri involved the construction of an interchange at Hampton Downs. Part of the pavement was constructed under "greenfields" condition while part of the Northbound lane was built over the existing two lane road.

The Specification for the work required that the normal steel wheel roller compaction be followed by 30 passes of a 10 tonne axle load. This was designed to reduce the risk of severe deformation developing in the wheel paths. The Contractor elected to apply the additional compaction using a watercart with the tank elevated so that most of the load was on the rear wheels.

The Contractor provided a set of the construction QA data for review as part of this research. Extensive use has been made of this information. Unfortunately this data did not include the results of any aggregate tests.

A highspeed roughness and rut depth survey was carried out in December 2003, approximately 11 months after the Southbound lanes were put into service. This information was supplied by the Network Manager.

3.4.2 Southbound Lanes

Figures 3.9, 3.10 and 3.11 provide a summary of the density in terms of Total Voids, the modulus values recorded for the basecourse and the maximum rut depth measured eleven months after the Southbound pavement was put into service.

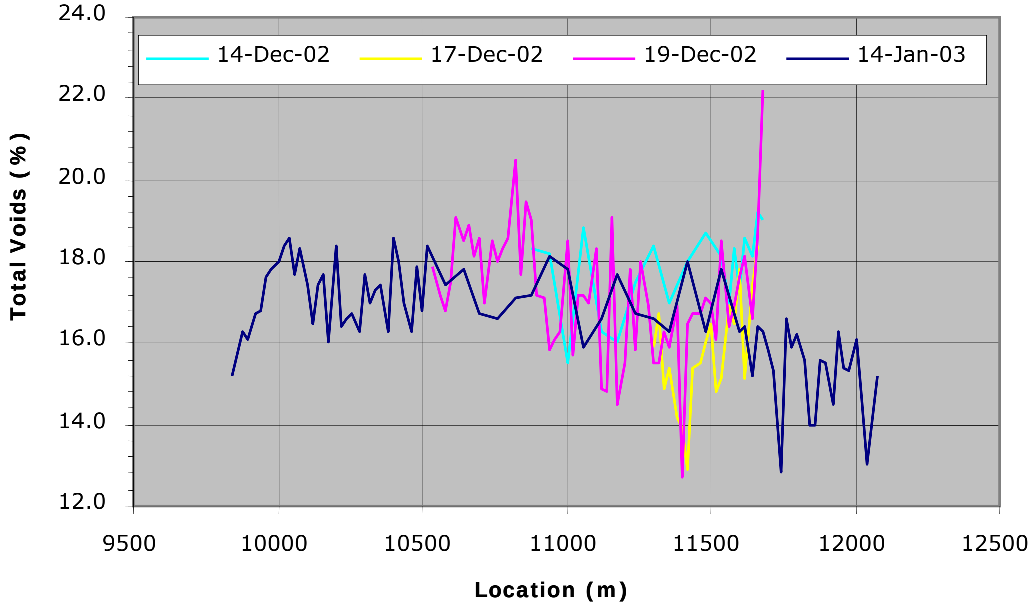

Fig 3.9 Total Voids in basecourse – Southbound.

Figure 3.9 illustrates the variation in the Total Voids of the basecourse during the compaction period. The final measurements generally lay between 16 and 18% dropping to between 14 & 16% over the last 400 metres.

Note that the density measurements were made with a nuclear densimeter and that the location of each series of tests varied over the full width of the pavement. The overall results can therefore only be judged on a statistical basis as summarised in Table 3.5.

Four sets of measurements were taken during the final period, three in December 2002 and one in January 2003. The length where the four sets of measurements can be compared lies between 11,300 and 11,660 m. The mean and standard deviation for these measurements are presented in Table 3.5.

Table 3.5 Total Voids in basecourse.

Date	14 Dec 02	17 Dec 02	19 Dec 02	13 Jan 03
Mean (mm)	18.1	15.8	16.6	16.6
Std Dev (mm)	0.71	1.38	1.30	0.84

These results confirm that the Total Voids and the standard deviation were reduced as a result of the additional compaction.

Figure 3.10 shows that between Dec 02 and Jan 03 the modulus of the basecourse laid from 10490 to 10880 increased significantly due to the additional compaction. The mean Total Voids calculated from density measurements taken in this section decreased from 18.3 % to 17.3 % and the standard deviation reduced from 0.92 to 0.61.

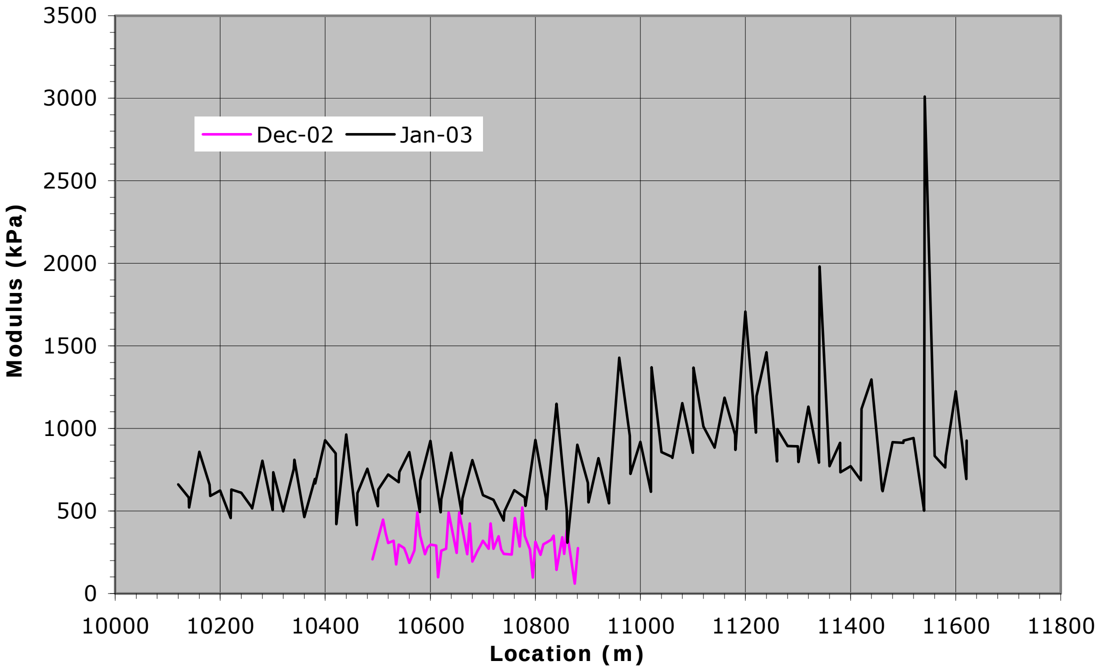

Figure 3.10 Basecourse Modulus - Southbound.

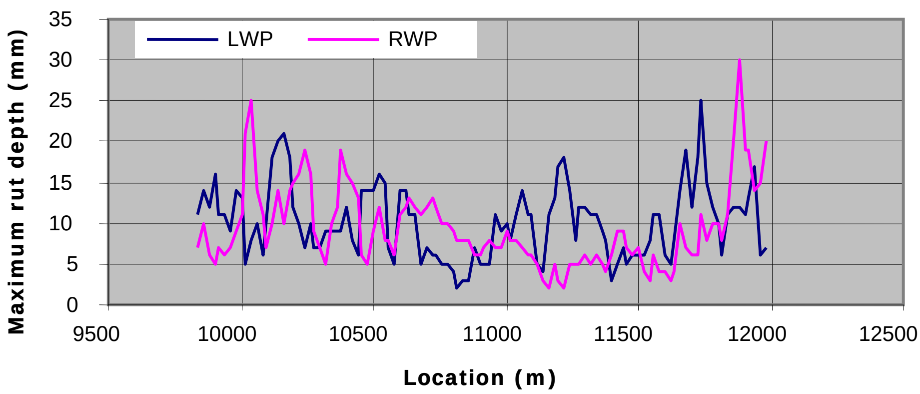

Figure 3.11 Maximum rut depth - Southbound.

Figure 3.11 shows the maximum rut depth in the wheel paths in the South bound lane in December 2003. The measurements are not particularly large but do not appear to relate to the Total Voids or the Modulus values.

3.4.3 Northbound Lanes

Approximately one kilometre of the Northbound Lane (10,920 to 12,000) was constructed as an overlay and widening of the existing pavement. The remainder of the existing pavement, (approximately 1100 m north of 10,920) was removed to form the interchange at Hampton Downs Rd. A small length at the north end was also overlaid.

The Total Voids, Modulus and rut depth measurements for the Northbound lanes are summarized in Figures 3.12, 3.13 and 3.14.

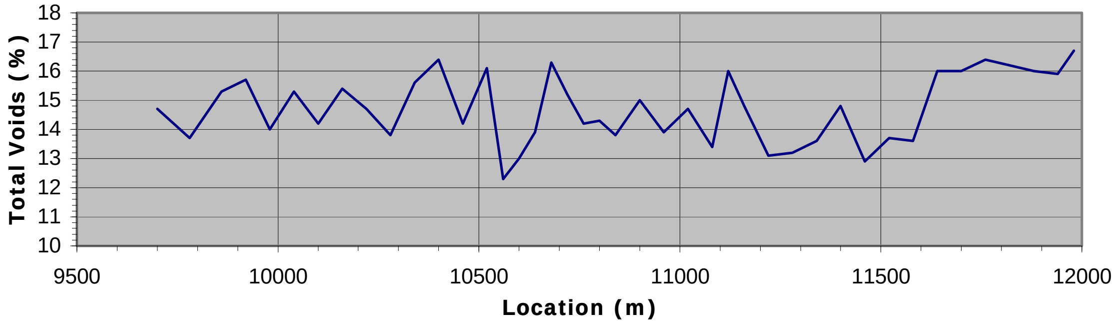

Figure 3.12 Total Voids in basecourse - Northbound.

The mean Total Voids for the Northbound lanes is 14.7% and the standard deviation is 1.2%. This is approximately 2 percentage points below the TV achieved in the Southbound lanes. The uniformity of the values is comparable.

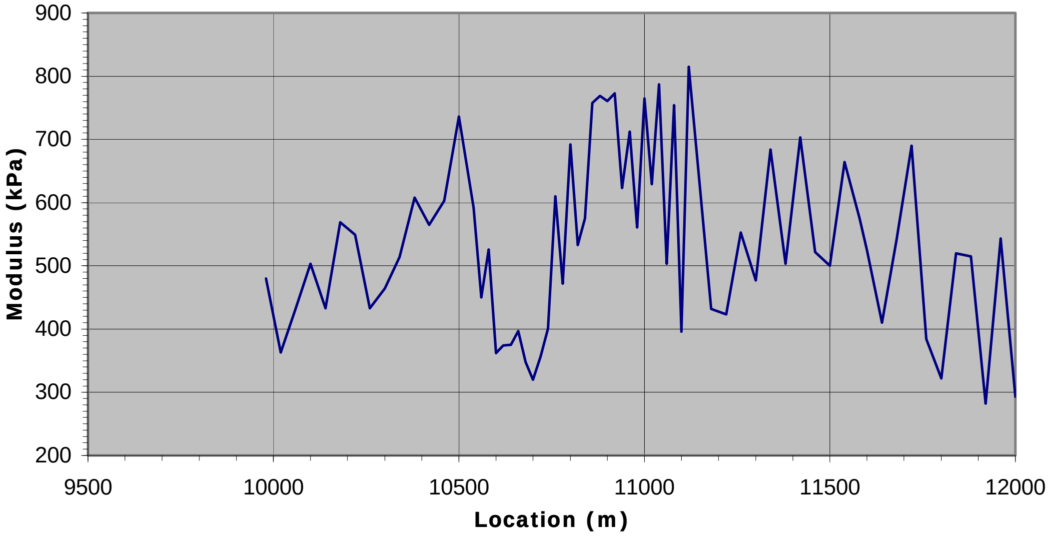

Figure 3.13 Modulus of basecourse - Northbound.

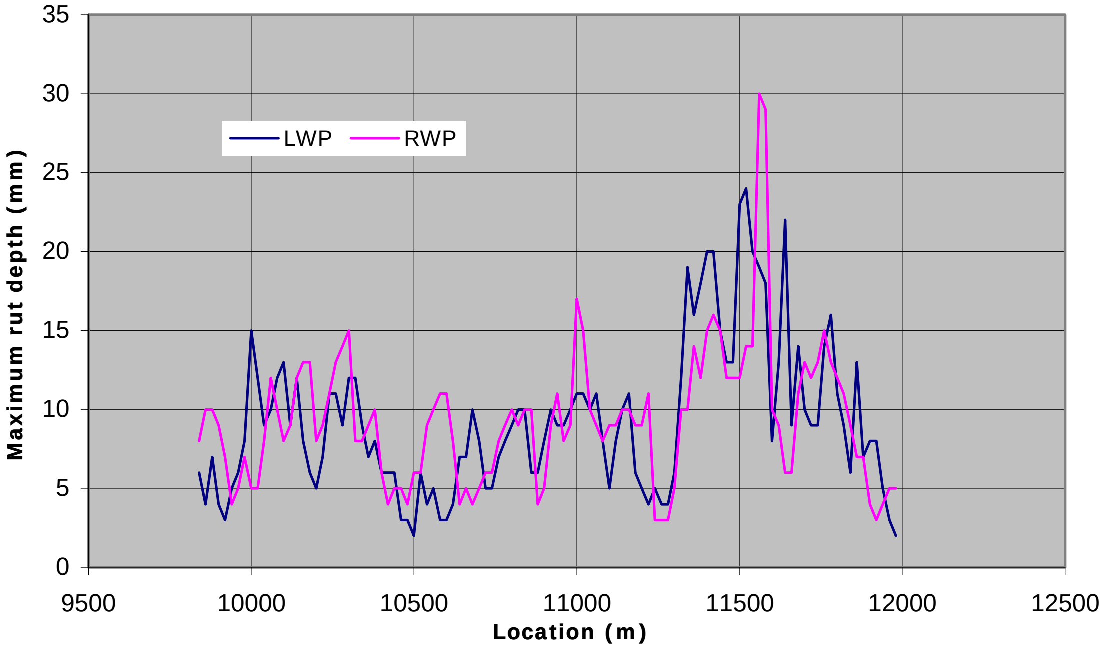

Figure 3.14 Maximum rut depth - Northbound.

The mean and standard deviation of the maximum rut depth measurements in the wheel paths are presented in Table 3.6.

Table 3.6 Mean values of the maximum rut depth measured in each wheel path.

	Southbound		Northbound	
	LWP	RWP	LWP	RWP
Mean (mm)	9.3	9.3	10.1	9.4
Std Dev (mm)	4.8	4.4	4.4	5.0

3.4.4 Rut depth data

The analysis of the rut depth information for both lanes indicates that both pavements have performed in a similar manner even though there are large variations in the values that defy explanation in terms of basecourse density. It would appear that the properties of the other pavement layers may have contributed to the development of the ruts in a manner similar to that described in Section 4.2 Kamo By-pass.

3.4.5 Comparison of Northbound and Southbound pavements

The Northbound lanes from the Interchange south were mainly located on the old SH1 pavement. The Southbound lanes were predominantly constructed on new fill and up to a third of the fills were on swamp deposits that had been either undercut or had wick drain and high strength geotextile placed. The subgrade on the fills achieved design CBR but it is believed that years of trafficking increased the stiffness of the Northbound (existing road) subgrade. As a consequence the stiffer well compacted subgrade of the northbound lanes provided a harder anvil to compact against.

3.4.6 Conclusion

The additional compaction applied using heavily loaded truck axles increased the density and improved the uniformity of the basecourse layer. It also caused a significant increase in the modulus of the basecourse.

The Total Voids values for the basecourse layer in the Northbound Lanes are lower than the equivalent values in the Southbound lanes. It is believed that existing, well trafficked subgrade under the Northbound lanes provided a better foundation on which to compact aggregate.

The results of a rut depth survey carried out in December 2003 indicate that the performance of the pavements has been satisfactory. The mean maximum depth values recorded are generally less than 10 mm.

This project has shown that mean Total Voids values as low as 15% are achievable provided additional compaction using heavily loaded pneumatic tyred vehicles is used.

The depth of the ruts that developed in the Northbound and Souhbound pavements were similar even though the basecourse in Northbound lanes was slightly denser.

3.5 Summary

It is known that surface deformation, mainly in the form of depressions in the wheel path, is a common problem found in newly constructed pavements. It is believed that the depth of ruts can be minimized by constructing the basecourse to the lowest Total Voids content possible.

Laboratory compaction of three different aggregates all well graded to different "*n*" values has shown that maximum density is achieved when the "*n*" value is between 0.33 and 0.39. This requires approximately 12% passing 75 microns which is much larger than most basecourse currently produced in New Zealand.

With regard to practical limits the analysis of data relating to three major pavement construction projects reviewed in this Section indicates:

a) Severe deformation occurred in pavements at Maungaiti Hill where the Total Voids content prior to trafficking was in excess of 20%. There appeared to be little or no deformation in the subbase layer.

b) Post-construction densification that occurred in both the basecourse and subbase layers at the Kamo By-pass caused significant deformation of the surface. The mean Total Voids in the basecourse between wheel paths which reflects the likely density prior to trafficking varied between 21 and 23 percent compared with a range of 15 to 19 percent measured in the wheel paths.

c) Additional rolling using heavily loaded trucks at Hampton Downs increased the density and the uniformity of compaction. An increase in the basecourse modulus also occurred.

d) A mean Total Voids value of 15% with a standard deviation close to 1% was achieved in the Northbound lanes at Hampton Downs but the subsequent rutting that developed was similar to that in the Southbound lanes where the mean value was 16.5%. The mean maximum rut depth varied between 9.3 and 10.1 mm.

4 Construction using a Dense Graded Basecourse

4.1 Introduction

This section describes the results of two construction projects where dense graded basecourse aggregate was used in a heavily loded pavement. One was a trial carried out by Works Infrastructure on a ramp linking Albany Highway to Constellation Drive. This was part of an investigation designed to identify a basecourse aggregate for the Upper Harbour Corridor Contract. The other involved the construction of full width test sections on SH5 near the intersection with SH28. This work was carried out by Fulton Hogan as part of their re-alignment of the highway at Tapapa.

4.2 Upper Harbour Corridor

4.2.1 Data collection and its interpretation

The information contained in this section was primarily extracted from internal reports that were generously made available to the Researcher. One additional set of surface data was subsequently obtained by the Researcher.

The Researcher was responsible for interpretation of the data and for opinions that are expressed about the results of the trial.

4.2.2 General

During 2003 Works Infrastructure investigated the use of greywacke aggregate from the Whitford Quarry for the Greenhithe Section of the Upper Harbour Corridor (UHC) Contract. The study resulted from a concern expressed by the Consultants employed by Transit NZ about the suitability of that particular aggregate.

Part of the investigation involved construction of a full scale pavement trial at the Whitford Quarry followed later by the construction of a larger trial section on the eastern on-ramp of the Albany Interchange.

The construction of a trial section at the quarry was intended to:

- i) evaluate the laying and compaction characteristics of the aggregate;
- ii) determine the plateau density for basecourse and subbase; and
- iii) monitor the development of ruts within the pavement caused by quarry traffic.

The trial pavements at the quarry were approximately 20 m long and occupied both the inward and outward lanes to the main crushing and stockpile areas. The pavements were constructed as normal road pavements using plant similar to that expected to be used on the UHC contract.

The particle size distribution of both subbase and basecourse aggregate plotted towards the lower limit of the respective grading envelopes. As a result problems were experienced during the compaction phase particularly with segregation of the aggregate and it was necessary to blind the surface with fines before the seal could be applied. The mean Total Voids achieved in the basecourse was approximately 20%.

The surface of the trial sections was monitored using a precise level. The results showed that:

i) Densification in the wheel path occurred rapidly and reached 5 mm after two weeks of use. The maximum difference for markers outside the wheel path to those within the wheel path was between 3 and 7 mm.
ii) The depth of the ruts in the Exit Lane from the quarry was greater than for those in the Entry Lane.
iii) Some minor elevation of the pavement occurred (a maximum 3 mm between and on either side of each wheel path).
iv) Maximum rut depth in the Exit Lane was approximately 8 mm.
v) The largest ruts occurred where the basecourse density (at time of sealing) was lowest.
vi) It was estimated that the rut depth could reach 10 mm long term (10^7 ESA).

The results of this trial showed that the particular aggregates used could be difficult to compact and were prone to segregation during compaction.

The trial was intended to determine a plateau density for each of the aggregates, but the outcome was unsatisfactory because of variations that occurred during compaction and also uncertainty about the results of the laboratory density tests.

One of the outcomes of the trial was a decision to attempt to manufacture a more densely graded aggregate.

4.2.3 Test section on Albany Interchange Eastbound On-ramp

4.2.3.1 Introduction

The objective of the construction of the Eastbound On-ramp was to further explore the suitability of the Whitford aggregate for use on the Upper Harbour Corridor Contract.

Part of the on-ramp was constructed to provide a link to Constellation Drive while a bridge was constructed to carry Albany Highway over the new Motorway. The section selected for the trial was on a permanent section of the ramp and lay between chainage 50 m and 150 m. It was constructed during March and early April 2004.

The aggregate used in the trial were AP 40 and AP 65 products produced to denser grading limits than those used for the Quarry Trial discussed in the previous section.

The pavement comprised:

- 150 mm TNZ AP40 basecourse
- 150 mm AP 65 subbase

- 250 stabilised subgrade
- subgrade CBR ≥ 4%

Approximately half the length of the trial section was in cut while the remainder was on fill.

4.2.3.2 Aggregate

A more densely graded aggregate was produced by decreasing the production rate from the Barmac Crusher from 140 t/hr to 90 t/hr. This effectively lifted the mid range of the particle size distribution from near the bottom of the grading envelope and reduced the Sand Equivalent of the basecourse to 32. The quantity of fine particles (– 150 micron) was not altered and the Clay Index remained low (2.2).

A comparison between the denser product and the Quarry Trial basecourse is illustrated in Figure 4.1.

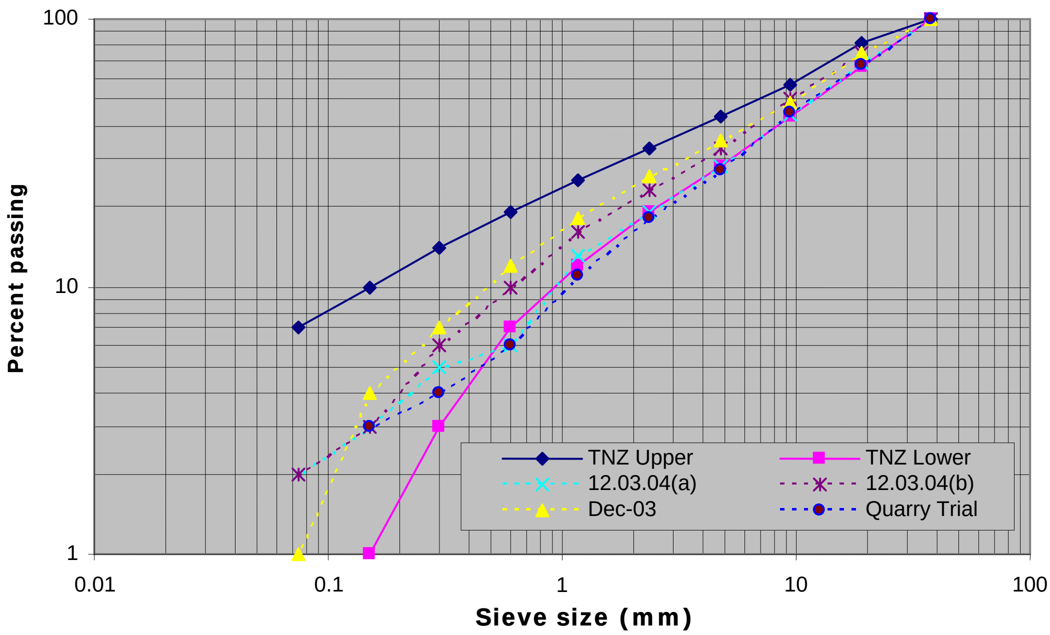

Figure 4.1 AP 40 curves.

4.2.3.3 Construction

The subgrade was stabilised with quicklime and compacted. Placing the subbase was commenced 7 days after stabilisation of the subgrade was completed and a combination of static and vibrating rollers was used to compact the layer. The mean density achieved was 98.6% of MDD giving a Total Voids content of 20.1% with a maximum value of 22.5%.

The basecourse was laid 20 days after the subgrade was stabilised. Compaction was carried out using similar plant to that used to compact the subbase but a 12 t pneumatic tyred roller was used to tighten the surface. The mean density achieved was 98.4% of MDD. The mean Total Voids was 15.4% with a maximum value of 17.4%.

The pavement was sealed with a chip seal before being opened to traffic on 13 April 2004.

4.2.3.4 Performance

The results of a number of high speed data surveys carried out over the first 20 months are summarized in Table 4.1 and Figure 4.2. The initial average rut depth within each 10 m length ranged between 3 – 7 mm and remained virtually constant over the first three months. A set of measurements eight months later showed a significant decrease suggesting that the pavement may have been resealed. The change in the Sand Circle results suggests that a coarser chip was used. Over the next eleven months the mean depth of the ruts increased again but it was not unusually large at the time that the last set of measurements were taken.

Table 4.1 Change in surface characteristics.

Date	Mean Sand Circle Dia (mm)	Increasing distance			Reducing distance		
		NAASRA Counts/km	Rut Depth (mm)		NAASRA Counts/km	Rut Depth (mm)	
			LWP	RWP		LWP	RWP
01 May 04	162	106 (20.6)	5.7	4.2	120 (37.2)	4.0	5.3
15 May 04	149	107	5.3	4.3	114	3.1	4.5
08 June 04	177	111	5.4	5.6	109	4.4	6.9
04 Jan 05	106	95	1.8	1.7	106	1.5	1.8
05 Dec 05	181	99 (22.8)	9.0	8.5	108 (32.8)	11.8	7.7

(The figures in brackets are the standard deviation for each set of measurements.)

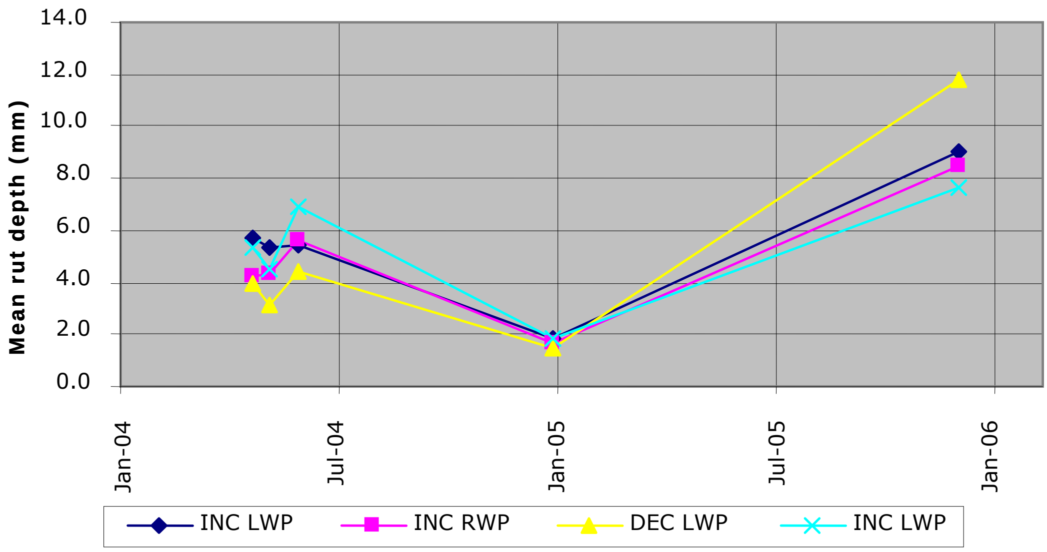

Figure 4.2 Change in mean rut depth.

The initial Roughness measurements show that the new pavement was rougher (mean NAASRA 106 & 120 counts/km) than the specified TNZ B/2 limits (mean 60 counts per km with a maximum of 70). The excess roughness was attributed to lack of experience of the roller drivers used on this section. Some improvement occurred during the monitoring period with the mean values decreasing to approximately 100 counts.

4.2.3.5 Discussion

4.2.3.5.1 Project objectives

The investigation undertaken by Works Infrastructure was intended to establish that the aggregate from the Whitford Quarry was suitable for use in the construction of pavements for the Greenhithe Section of the Upper Harbour Corridor. The full scale trials formed part of the evaluation process. The internal reports that were reviewed as part of this research suggest that the objectives were achieved.

4.2.3.5.2 Quarry test sections

The initial trial sections located at the quarry were intended to determine the laying and compaction characteristics of the aggregate.

Problems that were experienced during construction were primarily related to the lack of fines in the aggregate. Analysis of changes in the particle size distribution showed that the quantity of fines had decreased during compaction. This suggested that the fine material had dropped to the bottom of the layer and was not recovered as part of the test sample. It is doubtful whether this trial was adequate to define the compaction characteristics because of the segregation problems encountered.

The PSD of the AP 40 aggregate used as basecourse plotted towards the lower (coarse) side of the grading envelope and a running course was added before the surface was sealed. The harder rock types such as greywacke do not readily crush to a dense grading and it is not unusual for fines to be added to fill hungry areas and produce a surface suitable for sealing.

The Sand Equivalent of the aggregate used in the trial was below the minimum limit of 40 specified in TNZ M/4. This is a common problem with Auckland greywacke because of the fine-grained nature of the rock. The aggregate commonly lacks sand sized particles and clay size fines tend to be produced as rock particles break down during crushing and later during compaction. However, the quarry owners have argued in the past that pavements constructed using their basecourse perform well even though they do not meet the specified SE limit. This argument was eventually accepted by Transit New Zealand to the extent that the M/4 Specification was modified so that an aggregate with a SE < 40 is acceptable if either the PI < 5% or the Clay Index < 3.

Compaction for the subbase and basecourse layers was continued until each met the requirements set out in the TNZ B/2 specification. However, it was subsequently found that the laboratory MDD value used was low. This highlights a common problem with the compaction criteria based on the laboratory compaction test. This is because:

e) the NZS laboratory test used as a reference is subject to wide variation because of operator and other influences;
f) results depend on the grading of the sample used;
g) constructors are permitted to control compaction on the basis of the results of only one laboratory density test; and

h) compaction method used in the laboratory test is not related to the type of plant used in the field.

The results of this trial showed that while the products from the Whitford quarry met TNZ M/4 requirements the aggregates were difficult to compact and were prone to segregation during compaction. It also highlighted the problem with controlling compaction in accordance with TNZ B/2 specification even though the specified requirements were achieved.

4.2.3.5.3 Albany Interchange Eastbound Onramp

The test section on the Eastbound Onramp was intended to further explore the suitability of the Whitford aggregate for use on the Upper Harbour Corridor Project.

The aggregates used in the trial were AP 40 and AP 65 products produced to denser grading limits than those used for the Quarry Trial discussed in the previous sections. The basecourse produced by virtually halving the feed rate of aggregate to the Barmac crusher, had a grading curve closer to the mid-range of the M/4 envelope.

The denser graded aggregate along with an improved method of laying and compaction ensured that the Total Voids prior to sealing were low. The mean values achieved (subbase 20.1% and basecourse 15.4%) are remarkable and demonstrate the benefit of the more densely graded aggregate.

The surface of the test section was monitored closely for the first two months using high speed laser equipment. Another measurement was taken after eight months and the final run occurred 20 months after the pavement was put into service. The average rut depth increased slightly over the two first two months and then virtually doubled again over the next eighteen months. However, at the end of that time the ruts were not unusually deep.

The new pavement had a high level of roughness that decreased slightly with time.

4.2.4 Tapapa Curves realignment project

4.2.4.1 Introduction

Fulton Hogan Ltd (Hamilton) won a contract to realign SH5 at the intersection with SH28 at Tapapa not long after they completed the contract at Hampton Downs discussed in Section 3.4. As a result of their experience at Hampton Downs they agreed to assist the research into dense graded aggregates by accommodating full scale pavement trials on part of the Tapapa Curves contract. This work was approved by Transit New Zealand provided the work was under the observation of the Researcher.

A three lane, 680 m long section of the highway located approximately 150 m south of the SH28 intersection was selected. Within this length it was possible to construct three 200 m long test sections with a 40 m long transition between each test section. The transition sections were provided to accommodate a change from one aggregate type to the next.

The subgrade throughout the length was volcanic ash stabilized with cement.

4.2.4.2 Aggregate selection

Fulton Hogan provided particle size analysis for AP 40 aggregate from the Matamata Metal Supplies quarry at Matamata. This material is similar to that used in the laboratory compaction tests discussed in Section 2.4. Results of historical particle size analysis are shown in Figure 4.3.

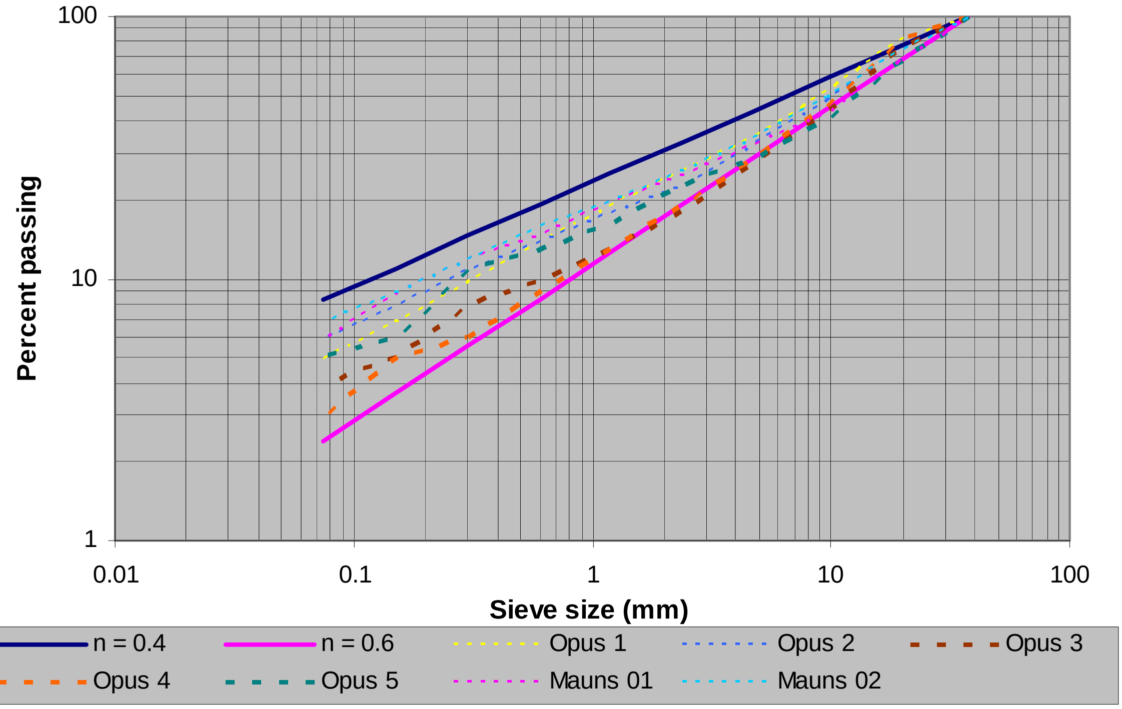

Figure 4.3 Matamata TNZ AP 40.

Particle size analyses for samples of AP 40 and quarry dust from the quarry and for samples of a local sand were used in a spreadsheet to evaluate suitable blends. This exercise was generally successful to the extent that the most suitable grading was found to be a blend of AP 40 and sand. While it was not possible to generate a "well graded" material by this method the addition of 15% sand provided a grading curve that was close to the *n* = 0.4 line. As a consequence it was decided that the aggregates to be trialed would be as follows:

- i) normal AP 40 produced by Matamata Metal Supplies Ltd (AP 40)
- ii) blend of 85% AP 40 and 15 % sand (15% blend), and
- iii) blend of 92.5% AP 40 and 7.5% sand (7.5% blend).

The blend of basecourse and sand was manufactured in the quarry by mixing the required quantities of AP 40 and sand in the stockpile area. This was a relatively cheap and simple exercise compared with the longer crushing process as used at Whitford Quarry, refer Section 4.1. Delivery of the aggregate commenced midway through October 2005.

The particle size distribution of samples of the aggregate obtained from the quarry prior to laying, are plotted in Figure 4.4. The samples had a mean Sand Equivalent value of 66.

It should be noted that the Contractor elected to construct the pavement on either side of the test sections using TNZ AP40 basecourse from the Waotu Quarry located near Putaruru.

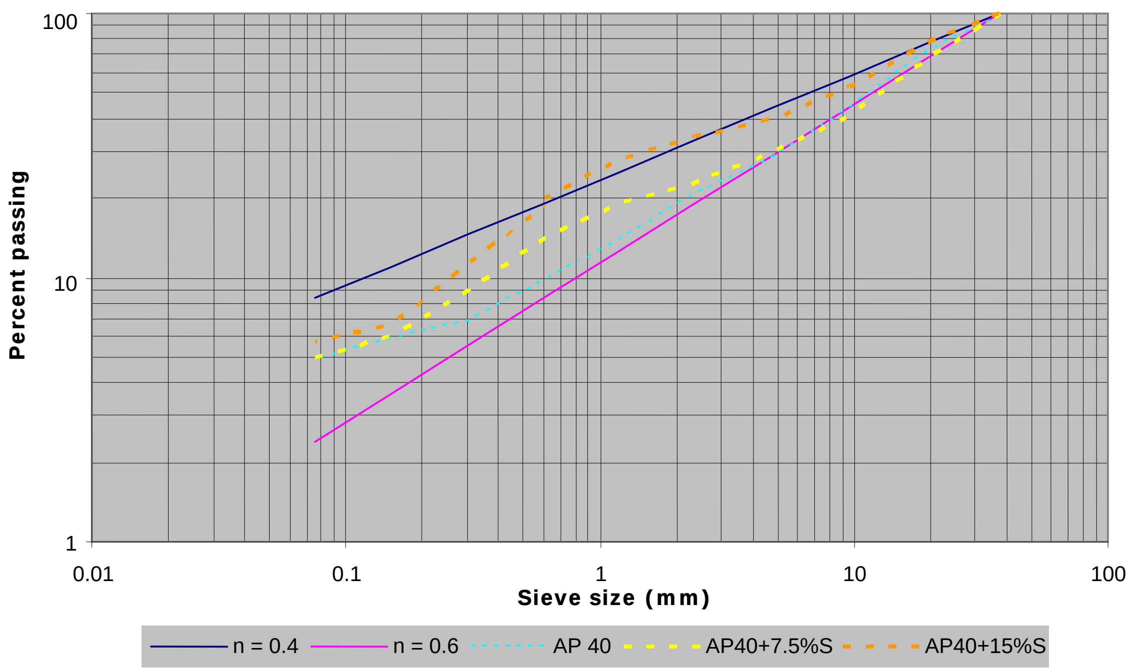

Figure 4.4 Aggregates used in test sections.

4.2.4.3 Construction

The layout of the test sections was as follows:

Chainage	Aggregate
7800 - 7820	Transition from Waotu AP 40 to Matamata AP 40
7820 – 8020	Matamata AP 40
8020 – 8060	Transition from Matamata AP 40 to AP 40 + 7.5% sand
8060 – 8280	Matamata AP 40 + 7.5% sand
8280 – 8320	Transition from AP 40 + 7.5% sand to AP 40 + 15% sand
8320 – 8520	AP 40 + 15% sand
8520 – 8560	Transition from AP 40 +15% sand to Waotu AP 40
8560 – end	Waotu AP 40

A plateau density test was carried out on the aggregate as soon as a sufficiently large area of each type was laid to the required depth (150 mm). The layer was watered before a vibrating steel wheel roller was used to compact it with 8 to 12 passes. During this time the change in density of the layer was monitored using a nuclear densimeter operated in backscatter mode. The same roller with the vibrator switch off was then used to complete the process to establish the plateau density. The nuclear gauge was positioned over the same point for each measurement.

The results of the plateau density tests are shown in Figure 4.5. Note that one "pass" involves the roller moving forward and then back over the same point.

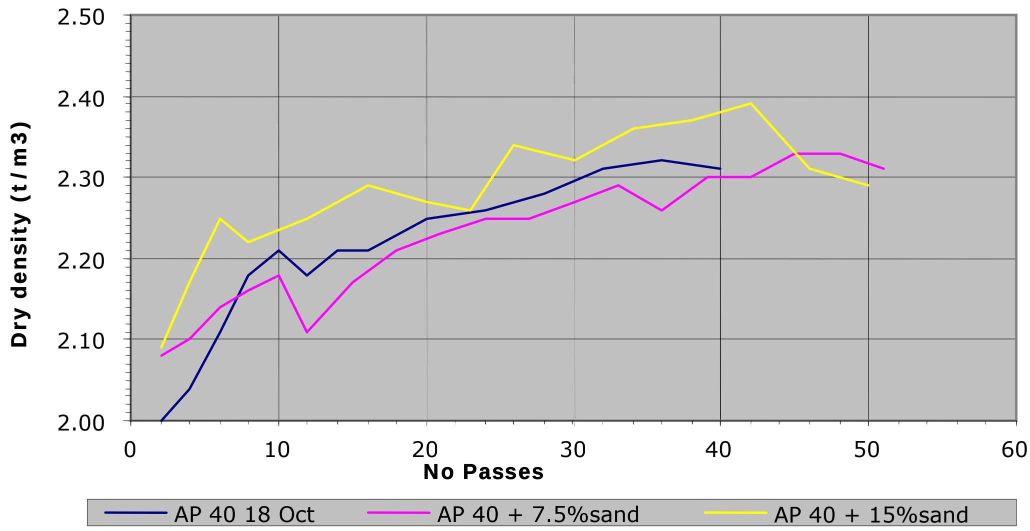

Figure 4.5 Plateau density tests.

The mean water content during the test on each aggregate was:

Aggregate	Mean Water Content (%)	Optimum Water Content (%)
Matamata AP 40	5.2	4.8
Matamata AP 40 + 7.5% sand	4.2	Not known
Matamata AP 40 + 15% sand	6.3	8.2

The mean PSD for three samples of each type of aggregate recovered from the compacted basecourse after the completion of the plateau density test are shown in Figure 4.6.

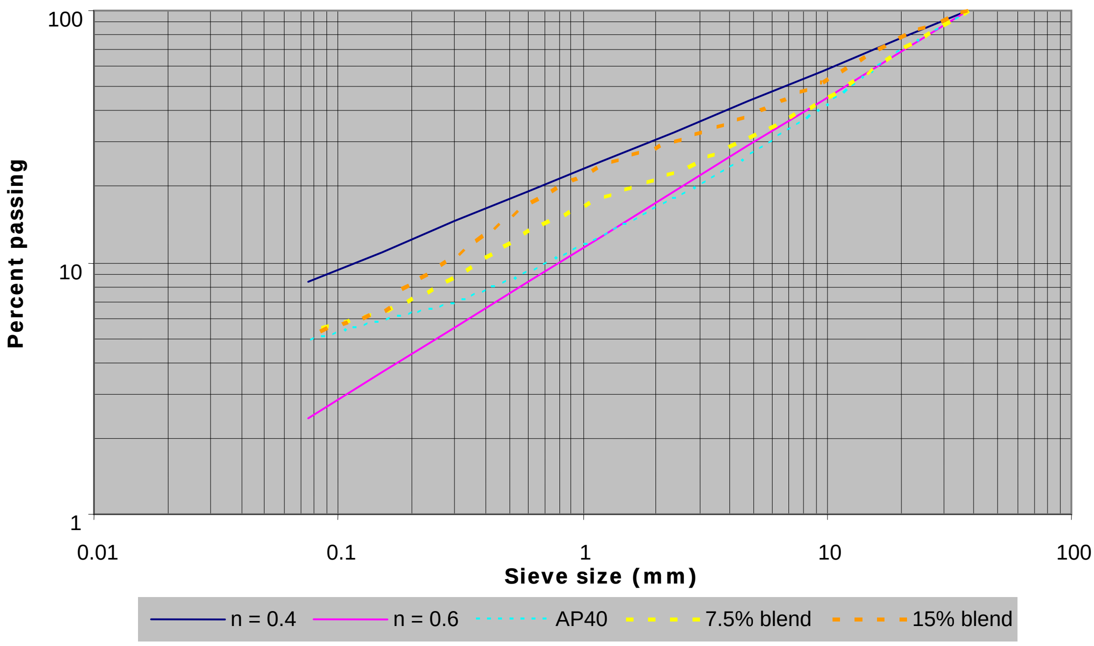

Figure 4.6 Aggregates after plateau density test.

The construction of the test sections was incorporated into the normal procedures being used throughout the Contract. Once all the basecourse was in place the watering, trimming and rolling using vibrating rollers continued for some weeks. During that time an AP 20 running course was applied over the whole road to tighten the surface.

In early December, the unsealed pavement was opened to traffic for approximately two weeks under controlled conditions. During that time it was also watered, trimmed and rolled with static steel rollers. The road was finally sealed in January and opened to traffic during the first week in February 2006.

The results of density tests carried out on the test sections prior to sealing are summarised in Figure 4.7 and the mean Total Voids for each test section are summarised in Table 4.2. The Dry Density increased (Total Voids decreased) as the proportion of sand increased (i.e. *n* value tended towards 0.4).

Table 4.2 Mean Total Voids

Aggregate Type	Mean Total Voids	Standard Deviation	Mean W/C (%)
AP 40	16.3	1.4	3.6
AP 40 + 7.5% sand	14.2	1.0	2.8
AP 40 + 15% sand	12.6	1.1	3.1

The water content was well below the Optimum Water Content (OWC) for these aggregates. For example the 15% sand blend has an OWC of 8.2%.

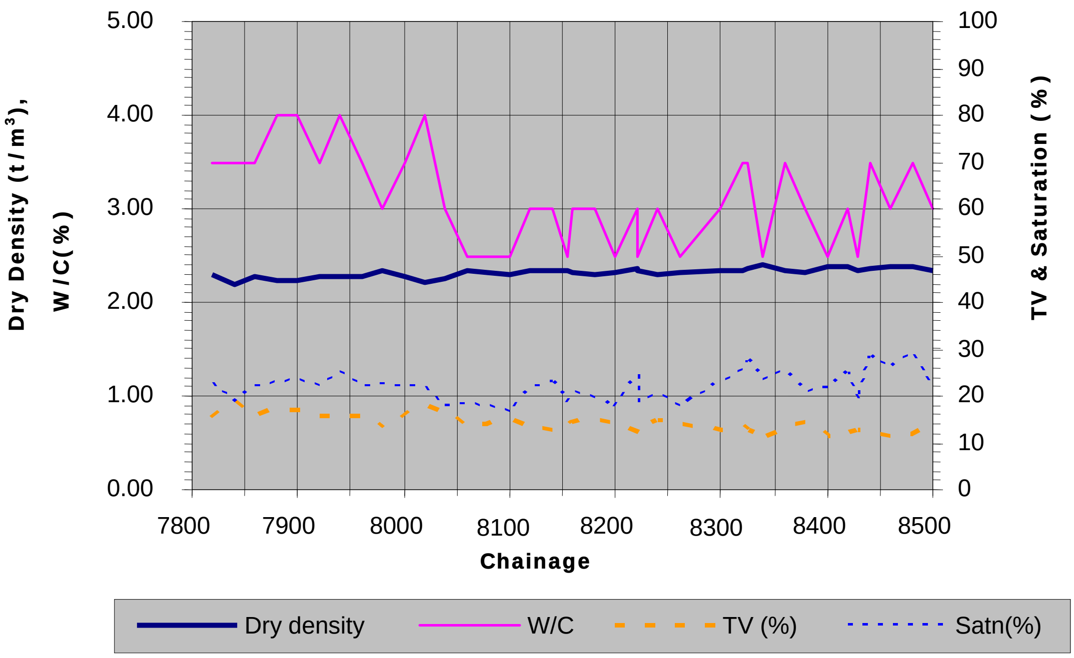

Figure 4.7 Density data prior to sealing.

The change in shape of the surface of the pavement was monitored using high speed laser equipment operated by Pavement Management Services. The first set of data was collected on 22 April 2006 and the second set approximately 12 months later. The measurements were taken with respect to the distance shown on the construction drawings and "6000 m" is 730 m passed RS 5.

The rut depth data is summarized in Figures 4.8 – 4.10.

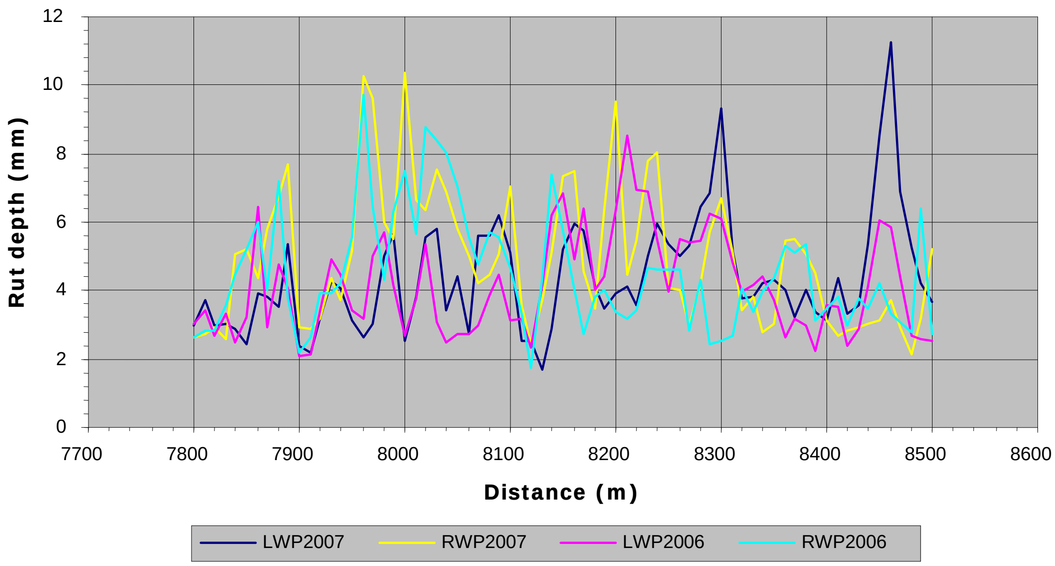

Figure 4.8 Rut depth Northbound Lane.

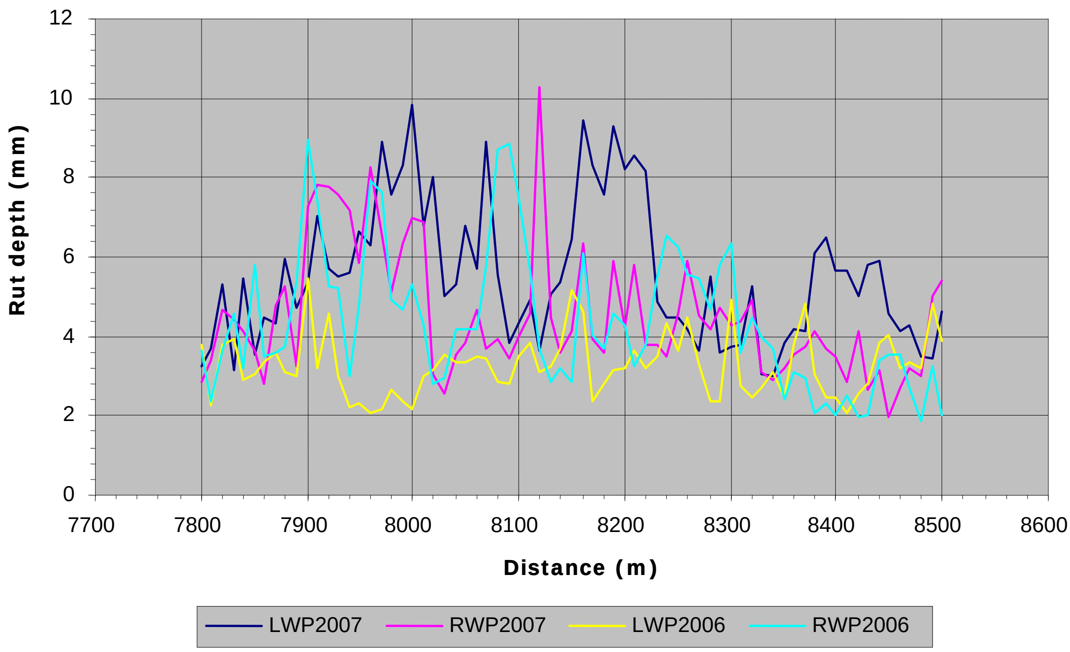

Figure 4.9 Rut depth Southbound Lane.

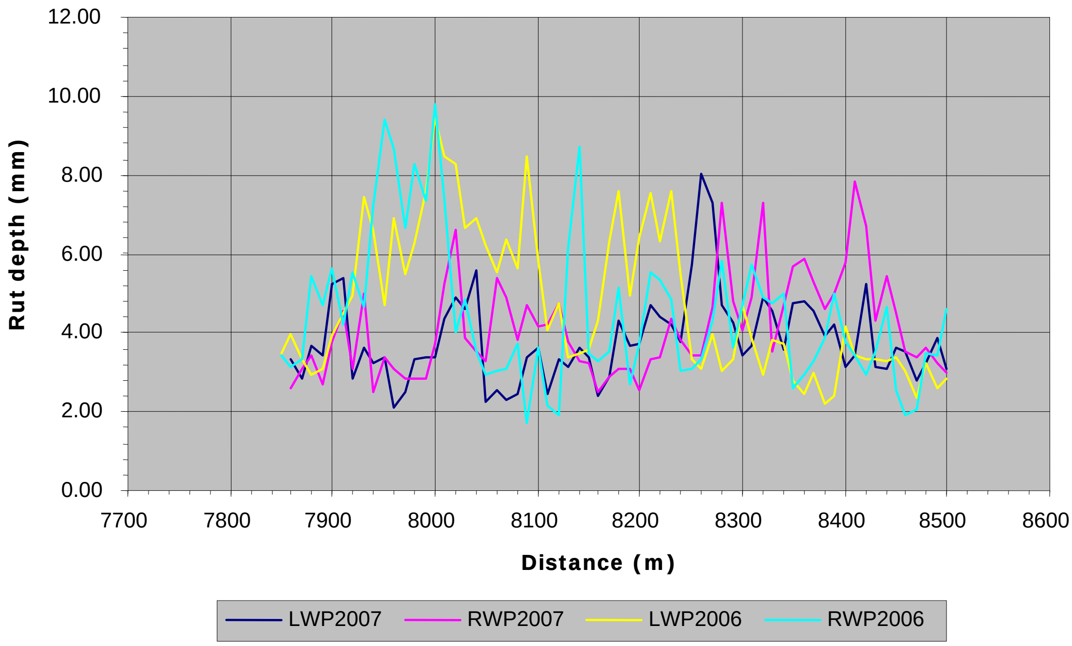

Figure 4.10 Rut depth Southbound Passing Lane.

The figures indicate that the rut depth in some sections decreased during the twelve months between measurements.

The maximum rut depth was slightly in excess of 10 mm which is similar to that recorded by Works Infrastructure at the Albany Interchange On-ramp.

The mean rut depth for each of the lanes in the test sections is summarized in Table 4.3 in comparison with similar measurements for the newly constructed road as a whole.

Table 4.3 Mean values of rut depth data.

Aggregate	Mean rut depth (mm)					
	Northbound		Southbound		SB Passing	
	2006	2007	2006	2007	2006	2007
SH5 as a whole	3.5	3.9	3.5	4.1	4.4	4.1
MatamataAP 40	4.4	4.5	4.1	5.9	5.9	3.5
AP40 + 7.5% Sand	4.6	4.9	4.3	5.4	4.7	3.9
AP40 + 15% Sand	3.7	4.1	3.1	4.1	3.5	4.3

The mean rut depth is small over all the test sections but slightly larger than for the road as a whole. However, it was noted that the rut depth where aggregate from the Waotu Quarry was used was generally lower than that for the test sections where aggregate from Matamata Quarry was used. The difference could also be related to the subgrade soil as much as to the basecourse per se. As far as the Matamata AP 40 is concerned the 15% sand blend had the lowest mean rut depth with a value similar to that for the road as a whole.

4.2.4.4 Discussion

The Tapapa Curves project has demonstrated that:

a) Blending aggregate and sand in the quarry is a relatively simple and cheap way to manufacture a more densely graded aggregate.
b) A suitably graded sand or quarry dust is required if a dense graded basecourse is to be manufactured by blending.
c) PSD of the 15% blend follows the n = 0.4 line reasonably well down as far as the top of the silt range.
d) In the plateau density test the15% blend had the highest density after the least number of passes.
e) The 15% blend had the lowest Total Voids of the three basecourse blends used in the test sections.
f) Twelve months after the road was opened to traffic, the 15% blend test section had the lowest mean rut depth of the three.

5 Conclusions and Recommendations

The main objectives of this project were:

a) to determine the optimum particle size distribution for maximum density and optimum water content for a selection of aggregates;
b) to identify the mechanism of rut development in new pavements by studying the changes in the characteristics of selected pavements;
c) to review available data to determine practical TV limits for construction; and
d) to incorporate the available results from this work into a full scale construction project as far as that is possible and to monitor the results.

5.1 Optimum particle size distribution

Three aggregates from different sources were tested in the laboratory. Samples from two greywacke quarries were tested in one laboratory while samples of andesite were tested in another. For these tests the AP40 aggregate was separated by sieving and then a quantity of each size range was mixed to give the required particle size distribution.

The results of each set of tests were remarkably consistent. An aggregate with grading following an *n* value of approximately 0.35 gave the highest density in each case.

5.2 Rut development

Studies of two pavements where ruts had developed soon after the pavements were put into service showed that densification in the basecourse or in the basecourse and subbase were the prime cause.

At Maungaiti Hill the softer aggregate produced to a coarse grading, degraded during construction and subsequent trafficking to cause severe deformation.

The main cause of the permanent deformation in the surface of the Kamo Bypass pavement was densification within the subbase and the basecourse layers after the road was opened to traffic. The Total Voids content of the basecourse was probably between 21 and 23% prior to sealing and decreased to between 15 and 19% after trafficking.

The additional compaction applied using heavily loaded truck axles at Hampton Downs increased the density and improved the uniformity of the basecourse layer. It also caused a significant increase in the modulus of the basecourse. The mean maximum rut depth value in each lane in December 2003 was generally less than 10 mm. This project showed that mean Total Voids values as low as 15% are achievable when additional compaction using heavily loaded pneumatic tyred vehicles is used.

5.3 Full scale test sections

This phase of the research was designed to explore the practicalities of constructing pavements using a dense graded basecourse and to identify any problems that might eventuate.

It was fortunate that both Works Infrastructure and Fulton Hogan were interested in aspects of the research. Fulton Hogan's interest stemmed from their experience at Hampton Downs and elsewhere while Works Infrastructure were actually pursuing similar procedures although for a different reason.

5.3.1 Upper Harbour Corridor - Albany Interchange - Eastern On-ramp

Works Infrastructure undertook an evaluation project to show that greywacke aggregate from Whitford Quarry was suitable for use on a heavily loaded motorway pavement. This involved both a preliminary trial at the quarry and the construction and monitoring performance of a full scale test section.

The quarry trial was only partially successful mainly because the coarse grading supplied by the quarry tended to segregate and was difficult to compact. For the subsequent Eastern On-ramp test section the quarry operator produced a denser aggregate by slowing the production rate of the Barmac crusher. This resulted in a basecourse with a grading curve closer to the mid-range of the M/4 envelope but at a significant cost.

The more densely graded aggregate along with an improved method of laying and compaction that was used for the test section on the Eastern On-ramp ensured that the Total Voids prior to sealing were low. The subbase had a mean value of 20% while the basecourse reached 15% which was a significant improvement from 20% achieved in the quarry trial.

The surface of the test section was monitored closely for the first two months using high speed laser equipment. Another measurement was taken after eight months and the final run occurred 20 months after the pavement was put into service. The average rut depth increased slightly over the two first two months and then virtually doubled again over the next eighteen months. However, at the end of that time the mean rut depth was generally less than 10 mm.

This project highlighted the practical difficulties of laying coarse graded aggregate i.e. one lying towards the lower side of the TNZ M/4 envelope, and the improvement in construction and performance that can be gained by using a dense graded aggregate. It also showed that mean Total Voids values of 20% for subbase and 15% for basecourse can be achieved with such aggregates.

5.3.2 Tapapa Curves

This project was designed to follow more closely the objectives of the research project. Here the dense graded aggregate was manufactured by blending the normal TNZ M/4 AP40 basecourse with a selected sand. Although the PSD of the blended aggregate was not entirely well graded the material compacted well and reached 13% Total Voids significantly lower than the normal AP 40 product.

5.3.3 Total Voids

The measurement of Total Voids has been shown to be a comparatively simple and accurate method for compaction control. The Solid Density of the rock from which the aggregate is crushed can be determined relatively easily using the ASTM C127:1980 and C128:1980 test procedure for Apparent Specific Gravity. This characteristic is generally constant for fresh quarried rock or river gravel and once established need only be checked occasionally.

5.4 Recommendations

As a consequence of the research carried out in this project it is recommended that:

a) the limits for particle size distribution set out in TNZ M/4 be changed so that the upper limit is described by an *n* value of 0.5 and the lower limit by an *n* value of 0.35;

b) quarry operators be permitted to add clean quarry fines or a suitable sand to ensure that their product fits well within the PSD limits and is as well graded as possible;

c) use of the laboratory density test for the control of compaction should be discontinued;

d) specification for the compaction of both subbase and basecourse should be controlled in terms of Total Voids;

e) Total Voids should be calculated using the Apparent Specific Gravity determined using ASTM C127:1980 and C128:1980 as a reference;

f) Apparent Specific Gravity should be measured by the aggregate producer and reconfirmed on an annual basis. The results should be made available to the construction industry.

References

Bartley Consultants Ltd, 2003. Works Infrastructure : *Transit New Zealand PSMC 002 Kamo Bypass Pavement Analysis SH1N RP 157/0.00 – 2.98 (Draft Report)*.

Bartley Consultants Ltd, 1996. Bloxam, Burnett and Olliver Ltd: *SH 1 Maungaiti Hill*.

Fulton Hogan Ltd, 2003. *Transit New Zealand 2/97-015/605 Mercer to Longswamp Four Laning Stage Two Quality Assurance Reports Northbound and Southbound Alignments.*

Jennings, D.N., Cunningham, J.K. & Thrush. M., 1998. *Practical Aspects of Basecourse Construction Control* NZ Geotechnical Society Roading Geotechnics '98. p 37-45.

Transit New Zealand. 2005. *Specification B/2: Specification for Construction of Unbound Granular Pavement Layers*.

Transit New Zealand, 2006. *Specification M/4: Basecourse Aggregate*.

Works Infrastructure, 2003. Upper Harbour Corridor Greenhithe Section Whitford Quarry Laydown Trial: Internal report December.

Works Infrastructure, 2003. Upper Harbour Corridor Greenhithe Section Eastbound Onramp Pavement Trial: Internal Report June.