

## **IPENZ TRANSPORTATION GROUP CONFERENCE 2013**

### **PAVEMENT SPECIFICATIONS: FIT FOR PURPOSE?**

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## ABSTRACT

Increasingly constrained road construction and maintenance budgets provides the ideal opportunity for technical innovations to challenge existing paradigms and provide even greater value for road agencies. As an example, New Zealand's specifications for pavement materials tend to be recipe based and are designed to produce premium aggregates for unbound granular pavement construction; however, these specifications are not always 'fit for purpose'. This paper describes a case study of a specification that was developed to produce a 'fit for purpose' pavement that reduced costs without compromising performance and conserved huge volumes of aggregate.

The performance based specification for design and construction of thin-surfaced flexible unbound granular pavements was developed in collaboration with Transit (now NZTA) to foster the use of marginal, non-conforming aggregates that give similar performance to premium basecourse materials. Another objective was to preserve prime quality aggregates.

A project completed under this specification is described, plus ten years of actual performance monitoring data demonstrates the benefits of the innovations that should be adopted on a wider scale throughout New Zealand.

Collaboration was the key to taking advantage of existing research and technical capability in order to provide value for money without unduly compromising levels of service.

## INTRODUCTION

No highway agency or local government wants to develop new pavement material and construction specifications for every possible alternative material. However, to encourage innovation and to give contractors the opportunity to trial new materials, NZTA has developed performance based specifications. These specifications set out the requirements for materials and road performance (e.g. rutting, roughness etc) at the end of the defects liability period, usually 1 to 3 years after the road has been opened to traffic. The objective is to allow a pavement to be built utilising a wider range of materials, provided it can be shown that these materials have adequate strength and durability for the design life.

Arnold (2000) reported that the benefits of performance specifications, compared with end product and recipe type specifications, include:

- The provision of a contractual environment that encourages innovation;
- Utilising contractor's experience;
- Focusing the client on performance rather than historical empirical relationships;
- Apportioning the risk between client and contractor appropriately;

Risks for the road authority in using performance based specifications include:

- Engaging a contractor that turns out to be unable or unwilling to construct the pavement to the specified quality;
- Paying for a product that does not meet the long term performance expectations.

The risks can be mitigated by:

- Mutual understanding between the road authority and contractors of the elements of the performance based specifications;
- Mandatory quality assurance requirements;
- Good prediction of long term behaviour.

The effects of performance based specifications are wide reaching and roles change significantly. The contractor is required to possess or out-source skills such as pavement

design and advanced materials testing. The road authority's role changes from designer/supervisor to that of quality auditor.

## PERFORMANCE REQUIREMENTS

The aim of developing performance requirements for the structural design and construction of flexible unbound pavements is to have a more direct relationship to required in-service performance. In an unbound pavement, the basecourse and subbase materials are required to:

1. Spread the wheel loads to reduce the load on the soft underlying subgrade and/or other weaker pavement materials;
2. Not fail in shear (i.e. shoving or rutting) with the application of wheel loads;
3. Have minimal deformation, where most of the deformation occurs in the subgrade;
4. Not deteriorate structurally over the design life;
5. Adequately hold and support the surfacing; and
6. Not be detrimental to the performance of the surfacing (e.g. cracking).

Table 1 Chip seal over unbound granular pavement: typical nominal layer thicknesses

Layer	Nominal Thickness (mm)
Chipseal	25 (1 <sup>st</sup> and 2 <sup>nd</sup> coat)
Basecourse	120 to 200
Subbase	150 to >300

The requirement to adequately spread the load over the subgrade is currently ensured by providing adequate pavement thickness as determined using the pavement thickness design procedures in the AUSTRROADS Pavement Design Guide (AUSTRROADS, 2004); typical nominal layer thicknesses are provided in Table 1.

Traditionally all the other requirements listed above are satisfied by using an unbound granular aggregate that complies with New Zealand's material specification for basecourse aggregate, M/4 (*Transit NZ, 1995*). This specification is a recipe for quarries to make a basecourse that has been proven over time to provide adequate performance in the road.

Materials that are less costly to produce than those complying with M/4 can meet the six performance requirements. The performance based specifications B/3 (*TNZ, 2000a*) and M/22 (*TNZ, 2000b*) provided a framework to allow these materials to be evaluated and used.

## B/3 SPECIFICATION AND M/22 NOTES

The *Performance Based Specification for Structural Design and Construction of Flexible Unbound Pavements* (referred to as B/3) had been developed to evaluate whether a pavement is meeting the six performance requirements stated above, at the end of its maintenance period; the acceptance criteria are set at a level that gives confidence the pavement will give adequate service over its design life. B/3 requires evidence that the materials used in the pavement have adequate strength and durability for the design life requested by the client before construction begins. *Notes for the Evaluation of Unbound Road Base and Sub-base Aggregates* (referred to as M/22 Notes) were developed to provide guidelines for providing that evidence. M/22 Notes are not mandatory and other supporting information proving the suitability of a material, such as road tests, may be needed to allow their use. B/3 also includes a number of checks at intermediate points in

the design and construction processes to ensure that the pavement is designed and constructed in a manner that is likely to produce the desired performance.

### Basecourse Requirements

M/22 uses the generally accepted definition of a basecourse as “the pavement material (stabilised or otherwise) forming the base, defined as the upper 100 to 200 mm of aggregate in a thin surfaced (less than 35mm) pavement”. The traffic induced stresses are greatest in this top part of the pavement. Therefore the highest shear strength and durability is specified for the basecourse.

The basecourse is required to be unbound to ensure shrinkage and tensile fatigue cracks do not reflect through the surfacing. Any stabilised materials used as a basecourse will undergo a tensile strength and linear shrinkage test. The stabilised material will not be accepted as a basecourse unless the tensile strength and shrinkage are below specified maximum values. Thus, this will normally limit the amount of cement added to a maximum of 2%.

To ensure the basecourse has sufficient durability and weathering resistance, the source rock is required to have the same crushing and weathering resistance as is required for high quality crushed rock complying with M/4. Often marginal aggregates will fail the weathering and crushing resistant requirements. A stabilising agent (e.g. cement) can be added to overcome this deficiency.

In M/22, in-service shear strength and deformation resistance of proposed aggregates can be proven by:

- Laboratory testing with the Repeat Load Triaxial (RLT) test;
- Full-scale testing at New Zealand’s accelerated pavement loading facility (CAPTIF) which is described elsewhere (*Pidwerbesky, 1995*);
- Acceptable performance on roads with documented maintenance and loading histories.

The Repeat Load Triaxial (RLT) test equipment applies a pulsating stress on the base-course sample to simulate the passage of one set of duals tyres with a standard 8.2 tonne axle load. Aggregate is tested in the RLT apparatus for 100,000 load cycles and the permanent deformation during the test is recorded. The test conditions simulate the worst in-service conditions. This could be saturated undrained at the maximum stress (load) conditions expected in service being applied. Materials that fail this test are those that have an increasing rate of deformation with increasing load cycles. The contractor has the flexibility to vary the RLT test parameters as they are required to guarantee the performance of the road for the specified defects liability period of 1 to 3 years. Alternatively, full-scale testing can be conducted at New Zealand’s accelerated pavement testing facility, CAPTIF (*Pidwerbesky, 1995*).

Acceptable performance on other roads requires independent and accurately documented maintenance and loading histories on a pavement in a similar environment. It would be necessary for the trials to have accurately recorded the maintenance history of the pavement and the change over time of parameters such as roughness and rutting. It would also be essential that the loading history was accurately known with actual measurements made of axle loadings.

A minimum soaked California Bearing Ratio (CBR) of 80% has also been specified. This allows the CBR test to be a screening test for materials being proposed as a basecourse before undertaking the RLT test.

## Sub-base requirements

TNZ M/22 defines the sub-base as any pavement material below the basecourse and above the subgrade or subgrade improvement layer. The overlying basecourse reduces the traffic induced shear stresses in the sub-base significantly and therefore the requirements are less stringent.

The specification allows the use of more than one sub-base material and recognises that the strength requirements for these materials are governed by their position in the pavement (depth) and strength assumed for design. As with the basecourse, M/22 requires that the subbase be durable and strong, however it can, unlike the basecourse, be bound.

Using bound layers in the subbase is acceptable as the unbound overlying basecourse material will prevent any cracks occurring in a sub-base material reflecting through to the surface. Therefore, there are no limits placed on the tensile strength or shrinkage of the sub-base material. This allows a sub-base to be a bound material should the design utilise a bound layer (modulus  $\geq 2000$  MPa) within the pavement.

To ensure the sub-base has sufficient durability and weathering resistance, the source rock is required to have a minimum crushing and weathering resistance similar to M/4, however with a lower load in the crushing resistance test. Should the source rock not meet the weathering requirements then the stabilised material is tested for weathering using a wet and dry brushing test as required for the basecourse.

In terms of strength requirements, a minimum soaked CBR of 30% has been specified for sub-bases, considered in the design as unbound, that are directly below the basecourse. For other sub-bases the minimum soaked CBR is governed by the amount of overlying material and design traffic as determined using the AUSTROADS thickness design chart for unbound granular pavements). If the pavement design requires a bound sub-base of a certain modulus then this will govern the minimum Unconfined Compressive Strength (UCS) based on a relationship between modulus and UCS in (*AUSTROADS, 2004*).

## DISCUSSION

B/3 provides the framework to reduce the risk to the client of using new and innovative materials by focussing on the performance of the pavement at the end of its maintenance period rather than the materials and processes that go into a producing that pavement. The performance criteria listed in B/3, if achieved at the end of the defects liability period, give confidence that the pavement will perform adequately for its design life. In addition, there are a number of intermediate checks which are actually either based on the Contractor's quality assurance plan or intermediate steps prior to the final acceptance criteria at the end of the defect liability period (1 to 3 years).

The major risk to the client and contractor is the inadequacy of currently available techniques for predicting pavement performance, both for the materials prior to construction and for the pavement immediately after construction. Procedures for predicting field performance, especially for recycled or other marginal materials, must be improved.

M/22 does not specify the use of any stabilisers, grading requirements or number of crushed faces, for example. It is the contractor's responsibility to provide a material that will meet the specified durability and strength requirements. The materials that meet the basecourse and subbase requirements will range from the unconventional to minor changes from traditional specifications such as M/4. Cost, availability and risk will be the driving factors as to the type of material chosen.

## CASE STUDY

Three pilot projects using B/3 were completed. On the first project, a realignment on State Highway 22 south of Auckland, construction was completed in 2001. The second project, the realignment on State Highway 6 in the South Island, was completed in April 2002. Construction of the third project, the Napier Hawkes Bay Expressway on the East Coast of the North Island, was completed in May 2003. All three projects included a 12 month defect liability period. Because the second project (Glenhope to Kawatiri) involved an alternative material in the basecourse, it is described in detail as a case study.

### Glenhope to Kawatiri project

This project involved 10 km of road realignment on State Highway 6 from Glenhope to Kawatiri, located 80 km south of Nelson in the South Island. The project construction period was 21 months and the project was officially opened on 19 April 2002, 3 months ahead of schedule.

The new alignment included 2 km of passing lanes and improved the driving alignment to a speed environment of 75 km/h. The improved alignment eliminated a number of accident "black spots". 4 km of the project passes through Kahurangi National Park, so this required some additional attention to environmental issues.

Local aggregates are alluvial deposits from the Buller River; the Gowan quarry is approximately 10 km south of the site. Thus, the contractor (Fulton Hogan) undertook a significant amount of investigation and laboratory work to establish the suitability of modified local basecourse products compared to aggregate that complies fully with M/4. The testing included repeat load triaxial testing. Several options on local basecourse specifications were presented with the tender.

### Pavement requirements and design parameters

State Highway 6 had an Annual Average Daily Traffic (AADT) count of 1500 vehicles per day (vpd), containing 12% heavy vehicles. For the required 25 year design life, this produces a design traffic loading of  $1.7 \times 10^6$  equivalent single axle loads (esa).

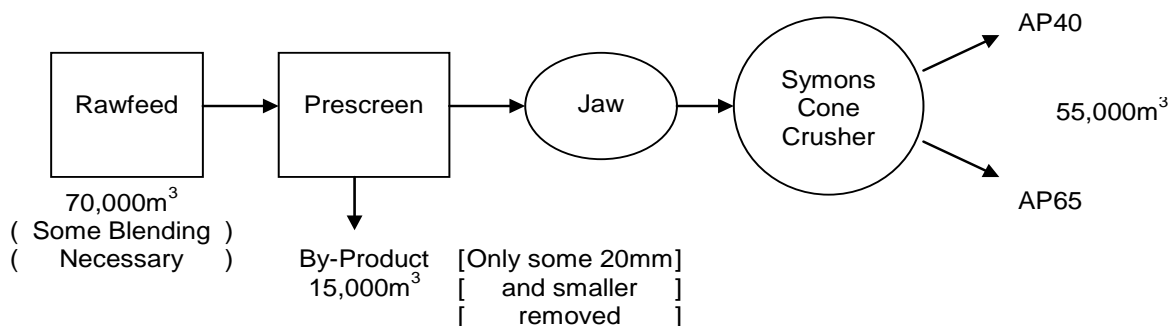
The assumed subgrade strength from earlier geotechnical investigations done at the feasibility stage of the project was a design subgrade CBR of 8. The modulus of the basecourse aggregate was 400 MPa for pavement design.

### Manufacture of the basecourse aggregate

Manufactured properties of the proposed quarry material were tested. The only substantial variation from M/4 (TNZ, 1995) specification for basecourse aggregate was the Broken Faces criteria: a minimum of 60% and 40% in the 37.5 to 19 mm and 19.0 to 4.75 mm fraction sizes, respectively, were the contractor's nominated criteria for the basecourse. For all other properties specified in the contractor's quality plan, the alternative material used for the basecourse met or exceeded the minimum requirements of M/4.

Normally, in order to comply with M/4, 70% of the particles larger than 4.75mm in basecourse aggregate must have broken faces; for this project, the basecourse aggregate characteristics for broken faces are in Table 2. Research conducted at New Zealand's accelerated pavement test track had confirmed that for state highways carrying low volumes of traffic, the percentage of broken faces in the basecourse aggregate could be reduced without significantly affecting the performance of the pavement (*Pidwerbesky, 1995*).

1. *Crushing Details for B/3 AP40 (Actually what happened)*



2. *Crushing Details for M/4 (What it would have been if we had to make M/4)*

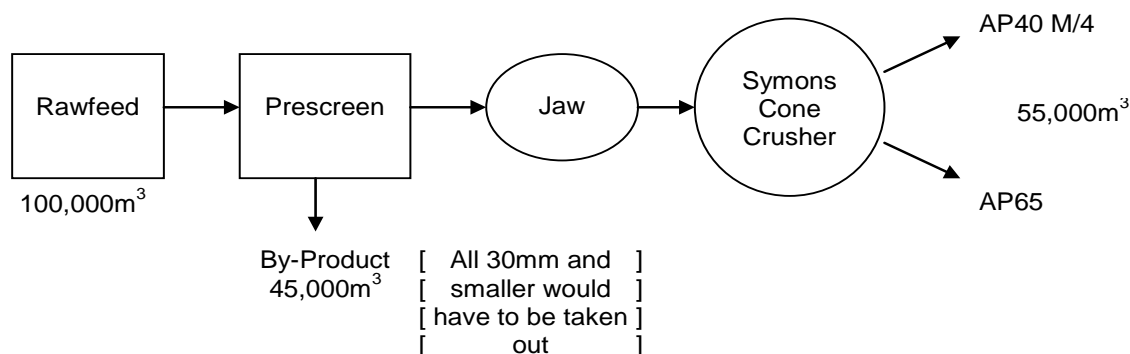


Figure 1 Aggregate Production Process for Case Study

The alternative material had a CBR of greater than 135 (M/4 requires a minimum CBR of 80%), and a modulus of 450 MPa (for RLT test conditions of 425 kPa deviator stress and 125 kPa confining pressure).

Table 2 Characteristics of Alternative Materials

Particle Size Range (mm)	Broken Faces (%)	
	Target in Quality Plan	Actual Achieved
37.5 – 19.0	60	64
19.0 – 9.5	40	52
9.5 – 4.75	40	43

Manufacture of the subbase and basecourse aggregates was all done at the Gowan Quarry. The Gowan Quarry has stratified layers of glacial and alluvial deposits. Large boulders in a sand and gravel matrix overlie finer gravel and sand layers. The excavator also mixed the material to ensure it was relatively homogeneous prior to crushing.

The crushing processes that would have been necessary (to fully comply with M/4) compared to what was required to achieve the specifically designed aggregate material are illustrated in Figure 1. The saving of 30,000 m<sup>3</sup> of raw feed had significant economic advantage as well as environmental benefits as that quantity of material is still in the pit that otherwise would have had to be quarried and wasted (there is no potential market for utilising this waste within an economical haulage distance of the pit).

## Construction

After compaction, the subgrade tests included nuclear density meter every 50 m, dynamic cone penetration (DCP) every 50 m in each lane, and Benkelman Beam deflection every 40 m in each wheelpath in each lane. This extensive testing regime was done to ensure that the subgrade condition was suitable for the pavement design above it; maximum allowable values for the deflection and DCP results were 2 mm and 25 mm/blow, respectively. If the subgrade was deficient, remedies included additional compaction and replacement of the material and re-compaction. As a last resort, a third option was to increase pavement layer thicknesses, but this was not required.

Pavement construction was done by conventional equipment and techniques. The specified material was generally a very good material to lay and productivities were no different to what they would have been laying standard M/4-compliant material. The material required 6 passes of the rollers to achieve the compaction density. The pavement laying conditions specified in the contractor's quality plan were all achieved.

The primary acceptance criteria for the completed basecourse, as far as the contractor's pavement designer was concerned, was a Benkelman Beam deflection of less than 1 mm and a coefficient of variation of less than 25% (coefficient of variation is the standard deviation of a sample of data divided by the mean of that data, and is a measure of uniformity).

The wearing course was a two coat (14 mm and 7 mm) chip seal. B180-penetration grade cutback bitumen was sprayed at a residual application rate of 1.8 l/m<sup>2</sup>.

## Post construction pavement performance

The acceptance criteria for pavement performance after one year were:

Rutting	Maximum rut depth of 10 mm
Texture Depth	Minimum 1.0 mm
Skid Resistance	GripNumber 0.60 (or SFC 0.60)
Roughness	Maximum 70 NAASRA counts/km (IRI 2.7) for a 100m section, and average of 55 NAASRA counts/km (IRI 2.1) over project length.

## Roughness and rutting

As part of this trial of the B/3 process, the highway agency and the contractor collaboratively measured the roughness of pavement sections prior to sealing, using a Dipstick Profiler. However, due to the time required to conduct this test and the relatively large variations in the results, due to the normal irregularities in the surface of an unbound pavement, this 'experiment' was concluded early in the project. Subsequently, roughness values for the sealed pavement were extracted from the normal high speed data survey conducted by Transit NZ annually. The roughness over the length of the project averaged 54 NAASRA counts/km (IRI 2.1), ten months after opening to traffic.

The rut depths were measured using a manual method every 50m, four months and 12 months after construction. Average rut depths have also been extracted from Transit's annual high speed data (HSD) survey conducted ten months after construction. The HSD data used is the maximum rut depth that occurred in a 20m section of roadway, whilst the manual data is the maximum rut depth at each 50m chainage. The data analysis results are tabulated in Table 3.



Table 3 Rut depths

Rut Depth (mm)					
	Mean	Std Dev	Average	Samples	Test Date
HSD	7.8	4.8	7	1370	April, 2002
Manual	5.4	2.8	5	384	October, 2002

Rut Depth, HSD Data (mm)					
Left Lane	8.1	4.93	7	684	April, 2002
Right Lane	7.6	4.67	6	686	April, 2002
Average	7.8	4.8	7		

Rut Depth, Manual Data (mm)					
Left Lane	5.3	3.03	5	192	October, 2002
Right Lane	5.5	2.50	5	192	October, 2002
Average	5.4	2.8	5		

As with all unbound granular pavements subjected to heavy vehicles, the rate of initial rutting under trafficking immediately after construction is relatively high, but soon levels off to the secondary rate of rutting. Because the two sets of measurements were not done at the same time, the difference in the two means (HSD vs manual) cannot be compared. However, the differences in the standard deviations can be compared, and this shows a significant difference between the two measurement techniques. Even though manual rut depth measurements are more costly, this form of measurement is probably more acceptable as a performance measure.

### Statistical analysis of deflection data

A primary indicator of pavement stiffness (and, thus, its expected performance) is its deflection under load. Therefore, an extensive programme of Benkelman Beam deflection testing was carried out on the subgrade and pavement layers; deflections were measured every 40 m in each wheelpath of each lane plus along the centreline. B/3 required that 95% of the values measured in a typical 1000 m<sup>2</sup> lot were to be better than or equal to the target values. However, this still allows for very poor sections to be accepted. One technique to improve this is to apply a more rigorous statistical analysis. In addition, this statistical analysis can assist in determining a suitable testing regime for future projects.

The mean pavement deflection was calculated at each chainage by averaging all transverse positions (that is, each wheelpath in each lane plus the centreline). The target deflection was 1 mm; 99% of the mean deflections were below 1 mm, while 94% were less than 0.8 mm. Standard deviations for the subgrade and pavement deflections were 0.43mm and 0.19mm, respectively.

As expected, deflections at the pavement surface were much more consistent than those for the subgrade, yielding a much smaller standard deviation for each individual section of road as well as for the entire road, as shown in Table 4.

The number of tests required to achieve a statistical level of confidence levels were also determined. Table 4 shows that the maximum number of deflection tests in any 1 km section of road required to provide a 90% confidence level is 17 per wheelpath, but on average only 10 deflection tests are needed. For a 95% confidence level the average maximum number of tests required was 38 per km. Thus, a 90% level of confidence can be statistically validated by testing deflections every 100 m in each wheelpath.

The confidence intervals used for determining the number of tests required are one-tailed, because no lower limit is required for allowable deflection. In other words, if sampling is carried out using the number of tests specified, then there will be a 90% (or 95%) certainty that the results are less than the target.

Table 3 Number of deflection tests required to obtain either 95% or 90% level of confidence of each wheelpath for 1 km sections

1 km Sections	Number of tests for 95% Confidence Level				Number of tests for 90% Confidence Level			
	LHS Outer	LHS Inner	RHS Outer	RHS Inner	LHS Outer	LHS Inner	RHS Outer	RHS Inner
0-1	33	27	35	33	8	7	9	8
1-2	12	18	36	42	3	4	9	11
2-3	33	24	45	50	8	6	11	13
3-4	34	26	19	60	9	6	5	15
4-5	54	64	49	51	14	16	12	13
5-6	34	70	50	24	9	17	12	6
6-7	32	37	65	22	8	9	16	6
7-8	15	32	23	22	4	8	6	5
8-9	13	30	22	28	3	7	6	7
9-10	22	48	36	38	5	12	9	9
<b>Mean</b>	28	37	38	37	7	9	10	9

Statistical analysis techniques are a valuable tool for both the contractor and road authority in quantifying the condition of the pavement and verifying compliance with acceptance criteria.

## Performance

The pavement has performed well during the past 10 years.

When FWD deflection testing was done over the site during 2003 to 2006, the 90<sup>th</sup> percentile peak deflections were consistently 0.6 mm (Figure 2).

Roughness has also been consistent. The 90<sup>th</sup> percentile roughness has been between 70 and 73 NAASRA counts every year from 2004 to 2012 (Figure 3).

The average rut depth increased slightly from 3.0 mm in 2004 to 4.5 mm in 2009, and has been constant since then. The 90<sup>th</sup> percentile rut depth increased from 4.5 mm in 2004 to 7.0 mm in 2010, and has been constant since (Figure 4). The rut depth performance of both lanes is nearly identical. Compared with the stable rut progression on a complete network dataset reported in Henning et al (2009), the actual mean rut depths on the Glenhope pavement exhibited similar performance until 6 years after construction, but then the rut depth has not increased, whereas the Henning et al (2009) rut progression model predicts that the rut depth would continue to increase at a steady rate.

In Figure 4, the difference between 2007 and 2008 performance monitoring data is entirely or at least partially due to additional lasers being added to the high speed measurement vehicle used on the state highways, which tended to increase the rut depth measured.

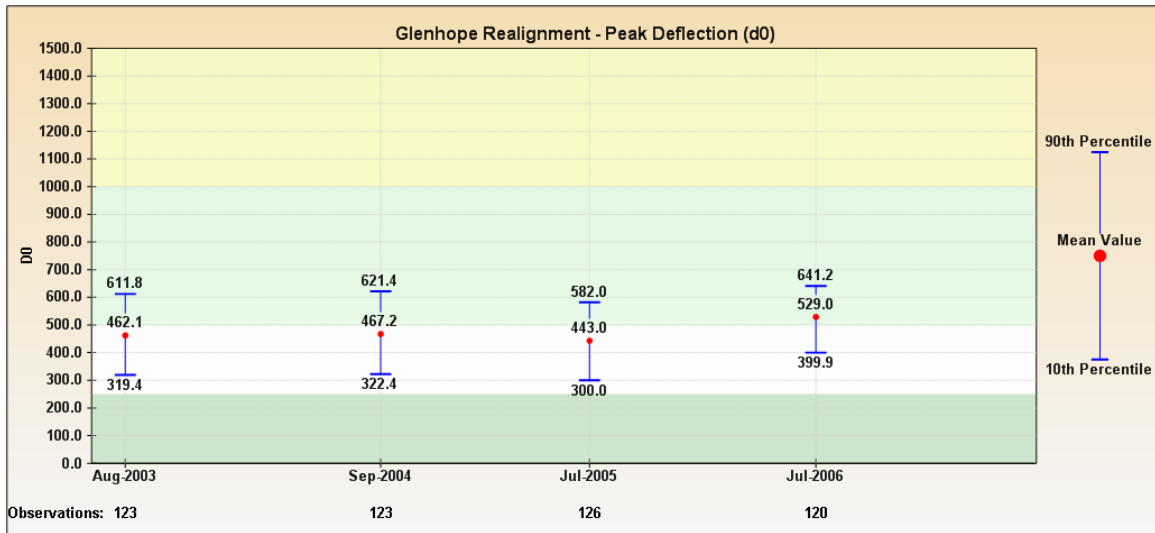


Figure 2 Peak Deflection

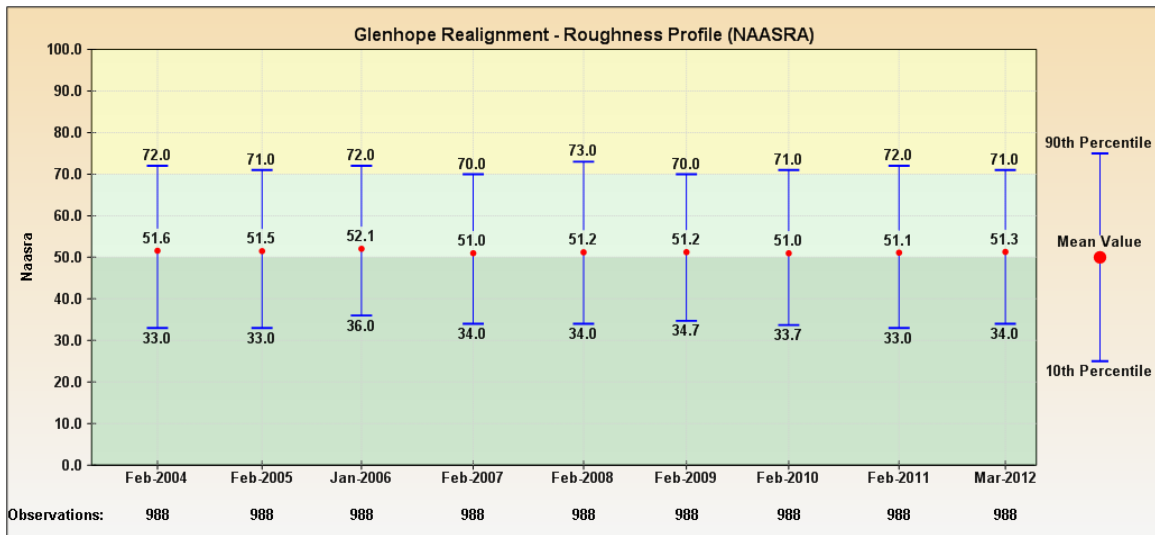


Figure 3 Roughness

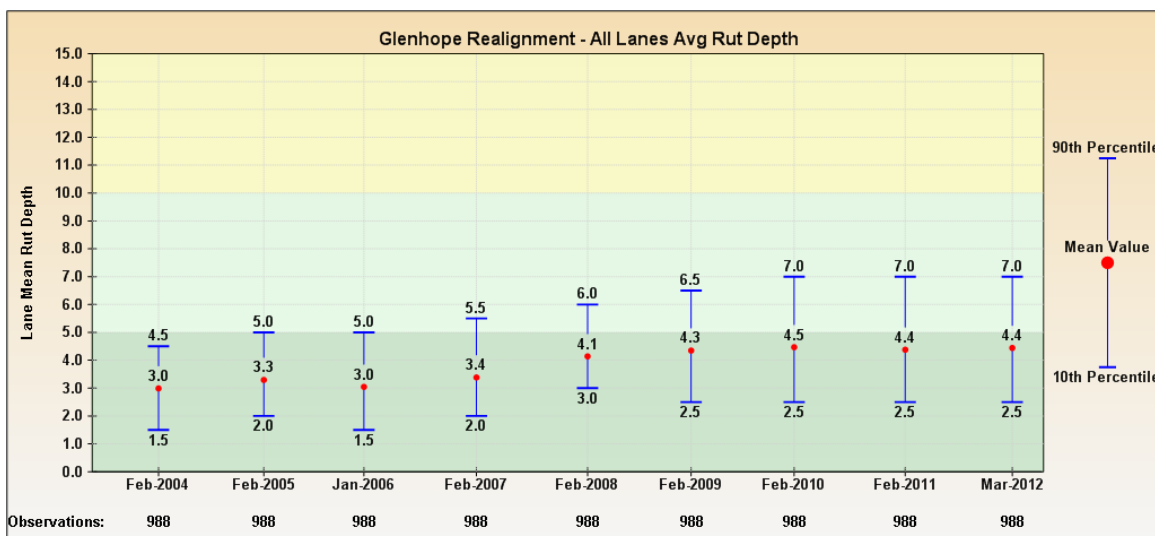


Figure 4 Average Rut Depth

## SUMMARY AND CONCLUSIONS

New Zealand's B/3 performance based specification for structural design and construction of flexible unbound pavements, including chip seal surfacing, was introduced in 2000 to foster the use of marginal and non-conforming materials that give similar performance to standard basecourse and sub-base materials. This paper provides an overview of B/3 and its accompanying document for materials, M/22: Notes for the Evaluation of Base and Sub-base Aggregates. Three pilot projects using B/3 were completed; details of performance data from one of these projects is presented as a case study.

Under the B/3 contract, the contractor was responsible for the design, construction and maintenance of the pavement and seal. The contractor had to demonstrate that design, materials, and construction techniques are appropriate by their quality assurance systems and that the pavement performance at the end of the defects liability period (1 to 3 years) is acceptable.

The following were checked to ensure that the performance criteria for the road have been met: surface shape and rut depth; roughness; texture depth; skid resistance; surfacing aggregate retention; and surface waterproofness.

Contractors undertaking work involving performance-based specifications require highly skilled and experienced pavement designers (either in-house or out-sourced); the road authority must also possess or have access to experienced, knowledgeable pavement engineers in order to be able to adequately assess submitted proposals.

The contractor completed an extensive suite of tests on the subgrade and pavement layers during and after construction, and analysed the data to determine statistically valid testing regimes for future projects. Statistical analysis techniques are a valuable tool for both the contractor and road authority in quantifying the condition of the pavement and verifying compliance with acceptance criteria, and should be an integral facet of performance-based contracts.

Road authorities and industry must collaborate, including sharing knowledge and expertise, in order for performance-based specifications to be successfully introduced and implemented. These parties must also work together to ensure that pavement research is relevant to their needs; better, more accurate and robust (but not complex) techniques must be developed for predicting pavement performance, so that the risks for the contractor and the road authority can be more readily quantified.

In addition to potential economic advantages, performance-based contracts can also preserve prime quality aggregates, so that road construction is achieved in an environmentally sustainable manner.

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