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BEST PRACTICE IN PAVEMENT DESIGN: WHAT IS IT?

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ABSTRACT

The purpose of the paper is to present a practical methodology for translating materials' knowledge and research into design and construction to achieve clients' expectations of pavement performance. This approach is demonstrated with case studies of actual pavement projects.

The AUSTROADS Guide to Pavement Technology provides a wealth of background and theory for a design practitioner to design a pavement that will satisfy the road owner's expected performance. However, best practice incorporates not only the final output from the AUSTROADS design guide, NZ Supplement and computer analysis but also engineering judgement, experience and practicalities of construction. Pavement performance can only be guaranteed by both a sound pavement design strategy and ensuring the design parameters are achieved in the construction process which requires direct communication between the designer and the construction crews, in terms of the construction methodology, materials and testing. Also, the paper brings together a range of information, research and developments, and in-service case studies of achieving quality, performance and value for money with a pragmatic design approach.

INTRODUCTION

Pavement design and performance modelling alone do not ensure that a pavement will provide the expected performance over its design life. The required level of performance can be assured only by ensuring that the design assumptions and parameters are implemented and achieved through the construction phase by the following key factors:

- Pavement design assumptions have to be validated by both field and laboratory testing prior to construction, and allow modification of the design to suit the variability of the in-situ material and the chosen construction methodology.

- The pavement design team must work closely with the construction crews, in terms of material design and optimisation, construction methodology and quality assurance testing, to ensure successful design, delivery and long term performance. Such a collaborative effort allows for early identification of potential issues that can then be either avoided or mitigated through design, and ensures that the pavement design is the best option in terms of risk management, resource utilisation and value for money.

- Test results during construction must be checked against the design to confirm the assumptions are still valid, allowing early identification of risk areas such as pavement variability and subgrade strength.

- Then, after the pavement is constructed, the pavement designer must check (or solicit feedback from asset managers) whether the pavement is performing as expected.

The above factors are at least as important as the pavement response models and CIRCLY calculations (MINCAD Systems, 2014) with respect to ensuring actual

performance meets or exceeds expectations. Regarding optimising pavement design, Major (1996) argued:

> *There needs to be integration of field experience with design capability and asset management understanding. Current separation of responsibilities is great, but has a high risk of sub-optimising.*
>
> *When a local maintenance engineer was also the designer and had a direct link to the execution of the work, it was practicable to operate at a significant level of risk if economies outweighed the costs of a small proportion of unpredicted early distress.*
>
> *Risk sharing must be regarded as a proper tool in seeking economy.*

Pavement design, materials, constructability, levels of service, expected performance, whole of life costs, and risk assessment must all be considered as part of a holistic approach to flexible pavement design, in order to optimise the process. Thus, the best available and most reliable practice for assuring the road owner of pavement quality is described below.

Practical procedure for pavement design and assuring pavement quality

The emphasis should be on devising an appropriate quality plan and ensuring that quality plans are implemented prior to and during construction, via training of staff, communication, best practice in construction and a rigorous testing regime. The material property and construction data must be assessed using valid statistical analysis techniques, and ensure statistically valid sample data is collected. For example, pre-construction activities crucial to a pavement rehabilitation design process are:

- Identification of pavement distress and failure mechanism(s)
- test pits at representative locations through the site
- evaluate properties of new and/or in situ materials
- project-level Falling Weight Deflectometer (FWD) analysis
- pavement historical performance data
- local practitioner knowledge and as-built records (if available) on previously undertaken treatments and their performance
- pavement strength requirements not only for predicted loading, but also surface requirements – i.e. deflection requirement for AC surfacing
- future demand on the site in terms of use, loading and surfacing.

During construction of an unbound or stabilised granular pavement, the most important properties to check are (a) compaction densities (including voids), (b) moisture content and (c) layer deflections with appropriate tools (which Clegg Hammer is not). Both maxima and minima must be specified for compaction (e.g. if compaction density is greater than 105% of target density, then either the target density must be re-validated or readings are incorrect), and Coefficient of Variation (CoV) tolerances to ensure uniformity. Uniformity in construction reduces the development of roughness;

roughness contributes to premature failure of a pavement. Yeo *et al*. (1996) reported that cost effective control of levels, moisture content and uniform compaction resulted in an estimated four times increase in pavement life, compared with 'normal' construction controls.

The emphasis is on constructing the pavement properly, with additional testing prior to and during construction, by assessing and assuring quality during construction – in other words, build the pavement right in the first place, within budget. Post-construction testing is less useful because by then, it is too late to cost-effectively correct any construction deficiencies. However, post-construction testing for both rehabilitation and greenfield pavements should include deflection tests, with engineering judgment applied to assessing bowl shape instead of relying on computer calculations only (i.e. analyse bowl shapes to assess if/where any weak spots and layers may be). Also, statistical analysis procedures should be more widely used, instead of absolute values. A more reliable and acceptable method would be to state such limits in terms of confidence levels (for example 95% or 99%) to allow for measurement errors, which can be significant, especially in high speed data.

Pavement Performance Indicators

The premise of any mechanistic-empirical flexible pavement design procedure is the ability to relate key structural response variables (such as deflections, stresses, and strains) to observed performance. This process relies on robust pavement performance models, which are typically based on regression equations that relate a material property, such as asphalt stiffness, to an observed distress, such as rutting or cracking. The AUSTROADS flexible pavement design procedure is based on multi-layer linear elastic theory and incorporates two primary design criteria:

1. Horizontal tensile strain at the bottom of asphalt or cement-bound layers, which governs fatigue cracking; and
2. Vertical compressive strain at the top of the subgrade, which governs vertical deformation/ rutting.

After a pavement is constructed and is in service, a road authority will normally have key performance indicators in the contract. Unfortunately, some techniques used to determine these performance indicators, such as pavement and subgrade modulus, and remaining life calculated from deflection bowls are too unreliable and inaccurate to be applied contractually.

In the absence of the deleterious effect of water ingress, unbound granular pavement performance is best predicted (with a correlation coefficient of 0.8) from rut development and peak deflections (d_0) after cumulative loading corresponding to between approximately 10% and 20% of the total life of the pavement (in terms of cumulative ESA) (Pidwerbesky, 1996). In other words, the deflections are very unstable (highly variable) in the first 2-3 years of, for example, a 25 year design period; it is only after that initial loading, that the deflection data begins to predict performance. Ioannides and Tallapragada (2013) concluded that "*determining the long-term performance of a pavement using observations spanning over a small fraction of its design life and a set of purely statistical/empirical algorithms poses significant engineering interpretation challenges*."

There is no discernible relationship between peak surface deflection or other deflection bowl parameter evaluated, and cumulative load repetitions during the initial traffic compaction phase, as is assumed by pavement performance models (Pidwerbesky, 1996). Deflection Ratio (DR) and Curvature Function (CF) are unreliable indicators of

pavement condition and performance; there is no correlation between DR, CF and subsurface strains, especially with respect to the vertical compressive strain in the subgrade, which is the fundamental basis of the rutting criteria for all flexible pavement design procedures.

Back-calculation

One means of estimating the expected life of a new or rehabilitated pavement has been to back-calculate stresses and strains within a pavement using FWD deflection bowls and mathematical transfer functions. However, back-calculation techniques based on FWD deflection bowls are inaccurate for (a) predicting remaining life of pavements at a project level and (b) estimating pavement and subgrade material properties (such as CBR) because:

1. The transfer functions (for calculating strains and stresses from assumed property values and measured deflections, then translating those strains into pavement life in terms of repeated standard axle loadings using a performance relationship) are based on regression analyses and are never calibrated for specific projects. The CBR's derived during the back calculation procedure are only intended to be relative and approximate, and used only in the context of pavement design overlays (the designer must have a subgrade design CBR as an input). This CBR value is only for the modeling requirements, and cannot accurately reflect the actual CBR of the subgrade. That has to be measured in the lab or inferred from in situ tests. In Table 1, example data from actual projects shows the variability in subgrade CBR values derived from different techniques.

Table 1: Subgrade Bearing Capacity Values derived from Different Techniques

Subgrade Bearing Capacity	**Rehabilitation Project**		
Parameter	**A**	**B**	**C**
CBR inferred from in-situ Scala Penetrometer	4%	4-5%	4%
Isotropic Modulus Backcalculated	69 MPa	35 MPa	86 MPa
Anisotropic Modulus Equivalent[(1)]	100 MPa	52 MPa	113 MPa
Laboratory soaked Subgrade CBR	15%	25%	
Subgrade CBR assumed for design	**5**	**4**	**5**

(1) Modulus back-calculated from FWD deflection bowl: 10th percentile isotropic subgrade stiffness converted to practical equivalent anisotropic stiffness (E_{ISO}=0.67x$E_{ANISO(vert)}$) (Tonkin & Taylor, 1998).

2. The principal criterion for calculating the remaining life of unbound or stabilised granular pavements using back-calculation techniques is the vertical compressive strain in the subgrade under that pavement. However, shear and/or deformation within unbound granular pavement layers are most often the principal cause of pavement failure – not deformation of subgrade (i.e. which is supposedly governed by the subgrade strain criterion). The subgrade strain criterion was only ever intended to be used for design purposes and provides reasonable values, given the cumulative effect of assumptions made during the design process.

This is why predictions of material properties and remaining life from back-calculation procedures (based on FWD deflection bowls) are poorly correlated with actual performance (AUSTROADS, 2003; Hansby, 2007; Litwinowicz, 2004; Werkmeister *et al*. 2005). Even in controlled conditions, as in the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), the back-calculated moduli were unrepresentative of the in situ properties of the pavement layers and subgrade (Pidwerbesky, 1996). For a road agency to rely on back-calculation procedures based on FWD deflection data to determine the remaining life of a specific pavement, then models and algorithms used in the procedure must be robustly validated for the specific conditions of each site.

CASE STUDIES

Two case studies are described below in which various aspects of the above concepts and design procedures have been applied. The first case study involved a performance based specification for design and construction, and the second case study was a design-build project.

State Highway 6 Glenhope to Kawatiri

In 2001, the New Zealand state highway agency developed a performance based specification for design and construction of thin-surfaced flexible unbound granular pavements, which was trialed 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, so that road construction is achieved in a sustainable manner that mitigates any adverse environmental effects.

A project completed in 2002 under this performance-based specification is described in detail elsewhere (Pidwerbesky, 2013); ten years of actual performance monitoring data demonstrates the benefits of the innovative specification that should be adopted more widely.

For a 25 year design life, the design traffic loading was 1.7×10^6 ESA. From geotechnical investigations, the subgrade design CBR was assumed to be 8%. The resilient modulus of the basecourse aggregate was 400 MPa.

Normally, for unbound granular basecourses in New Zealand, 70% of the particles larger than 4.75mm must have broken faces; the flexible pavement design alternative for this project allowed for changes in 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. For all other basecourse aggregate properties, the alternative material met or exceeded the minimum requirements.

The crushing processes that would have been necessary (to fully comply with the standard basecourse aggregate specification) compared to what was required to achieve the specifically designed aggregate material would have increased the waste stockpile by 30,000 m^3. This reduction in waste 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 was no other viable use for the potential waste within economical cartage distance of the site.

The acceptance criteria for pavement performance after one year included:

Rutting Maximum rut depth of 10 mm

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.

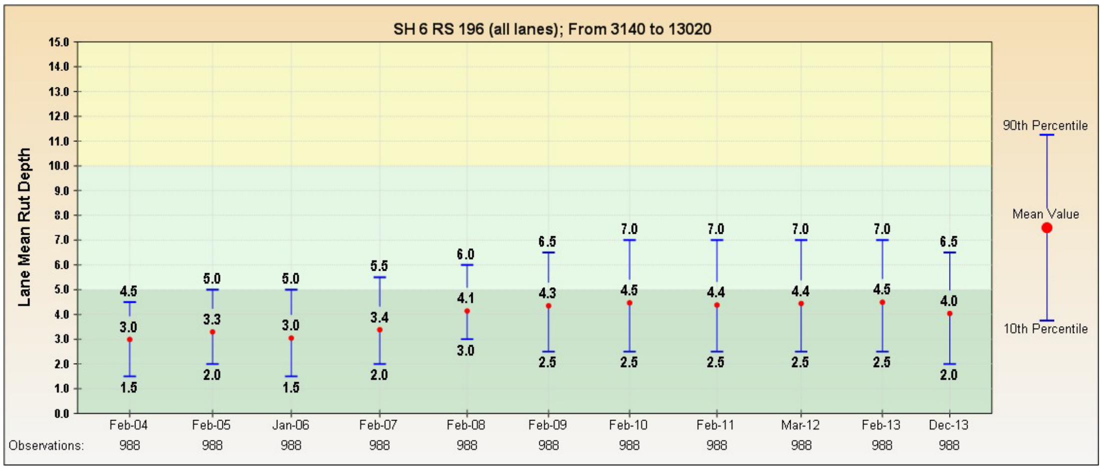

Figure 1: Lane Mean Rut Depth for State Highway 6 Glenhope to Kawatiri Realignment

The pavement easily achieved the above criteria and has performed extremely well during the past 12 years. When FWD deflection testing was done over the site during 2003 to 2006, the maximum peak deflections were consistently 0.6 mm. Roughness has also been consistent - the 90th percentile roughness has been between 70 and 73 NAASRA counts every year from 2004 to 2014. 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 90th percentile rut depth increased from 4.5 mm in 2004 to 7.0 mm in 2010, and has been constant since. The rut depth performance of both lanes is nearly identical (Pidwerbesky, 2013).

This project demonstrated that normal requirements for premium basecourse aggregates, especially with respect to broken faces and particle size distribution, can be modified and still achieve the desired performance, and the benefits of a holistic approach to pavement design while emphasising quality during construction to achieve the required performance.

Christchurch Southern Motorway – Stage 1

One of the NZ Transport Agency's key objectives for this project was extending the work already underway on sustainability in construction and maintenance activities. Thus, two of the unique aspects of this design-build project were: (a) the award two years prior to the construction contract of a supply contract for 60,000 m^3 of recycled crushed demolition concrete (RCC) (this was prior to the Canterbury earthquakes); and (b) the contract required that the pavement incorporate RCC. Prior to this project being awarded, NZTA and the contractor had already carried out research proving the value of RCC in flexible pavements including that it could produce a pavement of superior quality to those utilising virgin materials. NZTA also encouraged a sustainable approach by tenderers in respect to both materials and methodology.

Details of this project are provided elsewhere (Pidwerbesky *et al*. 2012), but key aspects are summarised here. The motorway consists of three zones: Zone A, Zone B and Zone C. Zone A involved rehabilitation of an existing road, Zone B is a new four lane greenfields divided carriageway, and Zone C involved construction of a new two lane road parallel and adjacent to an existing two lane highway, to create a divided carriageway. The pavements were constructed between 2010 and 2012.

Achieving all of the above was achievable only with a flexible, practical approach to pavement design; the final pavement designs for each zone are described below.

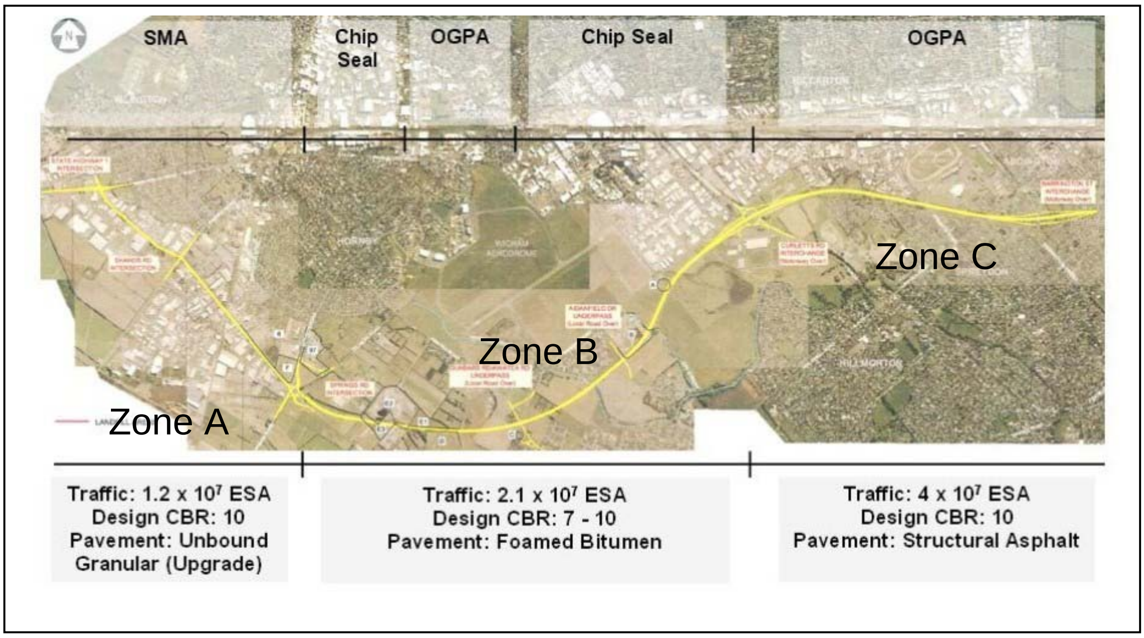

Figure 2: Pavement Design Details for Christchurch Southern Motorway, Stage 1

Pavement design for Zone A involved utilising the majority of the existing pavement structure as the deflections were generally less than 1.2 mm and the pavement was made up of clean aggregate with excellent source properties. The existing basecourse aggregate was of high quality, but had a significant proportion of rounded stone with the broken faces proportion less than 30% so this layer was modified to increase the percentage of broken faces and create an optimum blend for stabilising.

The final design involved milling out 100 mm of pavement which comprised approximately 45 mm of wearing courses and 55 mm of existing basecourse aggregate. 100 mm of virgin aggregate was then placed and the basecourse layer was stabilised to a depth of 150 mm, resulting in a layer with a combination of existing and virgin aggregate with the proportion of broken face exceeding 50% and a fines content suitable for stabilising. 1.8% cement was used as the modifying agent.

Utilising the existing pavement and designing a cement modified pavement significantly reduced the amount of excavation of old pavement and replacement with new aggregate required. This in turn reduced cartage movements from quarries. All aggregate that was milled out of the existing road was stockpiled and utilised in the cycleway formation on the project so that no material was wasted.

The Zone B pavement was founded on either natural in-situ river gravel subgrade, or a subgrade improvement layer formed from imported pit run material. The 250 mm thick subgrade improvement layer was used in areas where the natural subgrade material in cuts was a moisture sensitive silt layer. The subgrade design CBR was 10%; however, in all cases the measured CBR exceeded 20%.

The 200 mm thick subbase was formed from AP65 Recycled Crushed Concrete (RCC). The 150 mm thick basecourse was formed from a specifically designed virgin aggregate blend which was optimised for foamed bitumen stabilising and reducing aggregate wastage. The layer was then stabilised in situ with 2.7% bitumen and 1.5% cement.

Using the RCC eliminated the extraction and cartage of 120,000 tonnes of virgin aggregate to the project - equating to over 4200 truck and trailer movements or 105,000 km of cartage. Prior to this project, in Christchurch, demolition concrete was disposed of in a clean fill.

The use of foamed bitumen technology to form a high quality but cost effective basecourse layer eliminated the need for additional aggregate excavation and cartage that would be required for a thicker unbound pavement. In order to form an unbound pavement with an equivalent design life, a minimum of 75 mm of additional gravel would be required on the pavement. This equated to over 41,000 tonnes of additional aggregate, 1450 truck and trailer movements or 36,250 cartage km.

The Zone C pavement was founded on embankments constructed from imported gravel. These embankments were typically in the order of 4m to 5 m in height and therefore the subgrade had a conservative design assumption of 10%; however in all cases the tested CBR exceeded 20%.

The subbase was formed from a 200 mm thick layer of RCC which was cement stabilised with 3.5% cement in order to form a rigid anvil on which the structural asphalt basecourse could be constructed. The RCC subbase layer was stabilised in-situ. The subbase layer was specifically designed to be a stiff layer to prevent deflection which may cause fatigue related deterioration in the overlying asphalted layers. However, in order to prevent significant cracking in the stiff RCC subbase layer and potential subsequent reflective cracking in the asphalt layers, the stabilised RCC subbase was micro-cracked using a vibratory steel drum roller on low frequency vibration and high amplitude, which prevents formation of larger shrinkage or flexure induced cracks. In order to ensure that the micro-cracking was occurring (as it is difficult to visually inspect the cracks due to the small size) deflection testing was done prior to and post micro-cracking. Following micro-cracking, the average increase in subbase deflection was 0.2 mm, which was significant given that the deflections prior to the induced cracking were in the range of 0.2-0.3 mm. Finally, a polymer modified membrane seal was applied to the micro-cracked subbase layer to further mitigate any reflective cracking by providing a highly elastic membrane interface.

The 125 mm thick structural asphalt basecourse was formed from two layers of AC14 asphalt mix specifically designed to incorporate 30% reclaimed asphalt pavement (RAP). A 40 mm thick stone mastic asphalt or open graded porous asphalt wearing course (depending on the stress condition at different locations on the project) was applied over the 125 mm structural asphalt base.

Use of the RCC supplied by NZTA significantly reduced the amount of virgin aggregate extraction and cartage required whilst utilising what was previously considered a waste product in Canterbury. In addition, the heavy cement stabilisation also substantially reduced the amount of aggregate that would be required to form an equivalent layer from an unbound material. The use of RAP in the structural asphalt layers saved the equivalent of 5,500 tonnes of virgin aggregate and 275,000 litres of bitumen.

In this project, the pavement designer, contractor and highway agency collaborated to allow the use of a new (to New Zealand) construction methodology and robust pavement designs that minimised the use of virgin materials, maximised the use of recycled materials and optimised the pavement layer thicknesses, to create a flexible pavement that will perform well.

CONCLUSIONS

In order to achieve the required performance, the final output from any flexible pavement mechanistic design process has to be based not only on the design guide and computer analysis but also incorporate engineering judgement, experience and practicalities of construction. Pavement performance can only be guaranteed by both a sound pavement design strategy and ensuring the design parameters are achieved by construction processes which involve direct communication between designer and construction crews, in terms of the construction methodology, materials and testing.

Back-calculation procedures based on FWD deflection data should not be used to estimate the remaining life of a pavement unless the models and algorithms used in the procedure have been robustly validated for the specific conditions of each site.

This paper described a practical holistic approach to pavement design that has been successfully applied to a number of projects, which has resulted in substantial reductions in the use of virgin materials, extraction and cartage, and related energy usage, without compromising pavement performance.

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