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STABILIZED SUB-BASES FOR HEAVILY TRAFFICKED ROADS IN THE PHILIPPINES

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Abstract: The use of stabilized sub-bases for road pavements reduces or prevents the deterioration that is a consequence of poor material selection, difficult construction conditions and, in some cases, a low standard of construction quality control. Mitigation of these problems is important for both flexible and rigid pavements. This paper describes the early performance a full-scale flexible pavement trial that was built to evaluate the performance of alternative pavement and material designs using stabilized sub-bases and compare it with that of a traditional pavement design in use in the Philippines. The key variables in the research are the thickness of the stabilized layer which was varied continuously along each trial section and the strength of the sub-base layer which was varied in different sections. The trial pavement carries heavy traffic. Conclusions are drawn which suggest that the use of stabilized sub-bases significantly extends the life of the pavement.

Key Words: Cement stabilization, Road sub-bases, Pavement performance.

1. INTRODUCTION

The full-scale trial described in this paper forms part of the Pavement Investigation Research Project (PIR) which is being carried out under the Sixth Road Project in the Philippines. The overall objective of the PIR is to implement a programme of research aimed at improving the performance of road pavements, through a better understanding of the available materials and the transport demands, and the adaptation of modern techniques to the Philippine climate and traffic.

This full-scale trial addresses the use of stabilized sub-bases for relatively heavily trafficked roads. Cement stabilization is sometimes used for the roadbases of 'semi-flexible' roads (i.e. roads with a structural asphalt layer as the surfacing) but reflection cracking in the asphalt concrete (AC) resulting from the shrinkage cracks that occur in the stabilized layer are often a

problem unless the AC is very thick. Using a standard crushed stone layer as the roadbase and stabilizing the sub-base instead provides a way of obtaining a road with very good load spreading properties and also prevents reflection cracks. This type of construction has been very successful in various parts of the world and would be an appropriate solution for the Philippines. Thus the specific objective of this part of the programme was to determine the long-term structural performance of the trial sections and to develop design charts based on sub-base thickness, sub-base strength and traffic carrying capacity. The trial is located on the Nasugbu to Batangas City road in Batangas Province.

2. PROPERTIES OF THE TRIAL MATERIALS

2.1 Trial Design

The trial comprises four sections with the same roadbase and surfacing for all sections. These were a 200mm graded crushed stone roadbase and 100mm asphalt concrete surfacing. The strengths and thicknesses of the sub-bases are described in Table 1. The thickness of sub-base was continuously varied along the stabilized sections as shown in the table. To identify which ends of the sections had the thickest or thinnest sub-bases the ranges in the Table are reversed be consistent with the stationing. For example, the sub-base of section 2 was 350mm at station 142+440 and it reduced to 200mm at station 142+520.

Table 1 Experimental Sections

Section	Target UCS of Stabilized Sub-base	Thickness of Sub-base (mm)	Stationing		Length
			from	to	(m)
1	Control section	350	142+340	142+440	100
2	3 MPa	350 to 200	142+440	142+520	80
3	5 MPa	200 to 350	142+520	142+600	80
4	1 MPa	350 to 200	142+600	142+700	100

Note UCS = Unconfined compressive strength measured according to BS1924 (1990)

The works involved the complete removal of the existing surfacing and roadbase. The sub-base was then excavated and stockpiled for re-use. On Sections 2, 3 and 4 it was processed with the appropriate quantity of cement and replaced. On Section 1, an aggregate sub-base was used.

2.2 Subgrade and Aggregate Sub-base

The properties of the subgrade and aggregate sub-base are shown in Table 2.

The aggregate sub-base was constructed in two equal layers each 175mm thick. At the design density of 95% of the maximum dry density obtained in the T180 compaction method

(MDD), the material had a CBR of 28% in the soaked condition. This is marginally lower than the design CBR of 30%.

Table 2 Properties of the Subgrade and Aggregate Sub-base

Property		Subgrade	Aggregate Sub-base
Liquid Limit	%	50	18
Plastic Limit	%	30	11
Percent passing 75µm	%	74.4	9.6
Type Class		A-7-5	A-2-4 (0)
Optimum Moisture Content	%	22	11.8
CBR at 95% of MDD (soaked)	%	8	28

2.3 Stabilized Sub-bases

The raw material was a well-graded, non-plastic, sandy gravel. Cement was added to the material to determine the compaction characteristics for each blend and the required cement content for each section was determined. Table 3 gives a summary of the strengths obtained after 7 days curing.

Table 3 Strengths of the Stabilized Sub-bases

Section	Strength Range (MPa)
2	2.8- 4.7
3	5.3 – 5.4
4	Not available

The stabilized sub-bases were constructed in two layers. To achieve the required thickness variations to develop the design charts, the first layer was constructed with a varying thickness from 220mm to 70mm so that the second layer could be constructed with a constant thickness of 130mm throughout. Once compaction was completed, the surface was covered with plastic sheeting for seven days to provide a temporary curing membrane.

2.4 Crushed Aggregate Roadbase and Bituminous Surfacing

The crushed aggregate roadbase, made from an approved blend of materials, met the specifications given in Overseas Road Note 31 (TRL, 1993) and the bituminous surfacing met the DPWH Standard Specifications for Highways, Bridges and Airports, Volume II 1995 (DPWH, 1995).

3. PERFORMANCE MONITORING

The performance monitoring included surface condition measurements, Falling Weight Deflectometer (FWD) tests, Dynamic Cone Penetrometer (DCP) tests (TRL, 1999), and measurement of the traffic loading. This paper covers the total (present and future) traffic carrying capacity as estimated by the structural number approach from DCP tests and by the deflections.

For unbound materials the CBR values under standard test conditions have been used in specifications for many years and non-destructive tests such as the DCP test have been correlated successfully with CBR values for assessing the 'strength' of unbound materials in existing roads. For cement or lime-stabilized layers the unconfined compressive strength is appropriate and, for bitumen-stabilized layers, the Marshall stability is the most common measure of strength.

4. ANALYSIS

The trials are only two years old and, to date, the cumulative loading is 2.4×10^6 equivalent standard axles (esa). They are expected to carry traffic successfully for many years. At this stage it is not expected that any differential performance will be detectable between sections. Thus the purpose of this analysis is to examine the structural data and compare it with models or empirical evidence from elsewhere to verify likely long-term performance.

The construction of the trials, summarised in Chapter 2, indicates that, in general, the trials have been constructed to a very good standard, but one that should be readily achieved in normal construction practice.

4.1 Subgrade Strength

One of the most important parameters to determine in any pavement design is the strength of the underlying subgrade because it is this that we are protecting from damage by building a pavement, and it is this that has the greatest influence on the structural design. Figure 1 shows how the *insitu* subgrade strength varies along the site and Figure 2 shows the statistical distribution of its value. Variability of this magnitude is perfectly normal for fine-grained soils because the CBR is extremely sensitive to density and to moisture content, both of which show considerable in situ variation.

For pavement design purposes we require either an average value or a lowest 10-percentile value, depending on which design method is used, but we also want to know the values throughout the year (to use the full AASHTO method) or the values at the time of year when the subgrade is at its weakest. Other methods require the value under soaked conditions.

Figure 2 shows that the lower 10-percentile for design (ignoring at this stage any seasonal variations that might reduce the value for design) is about 5%. The median value is about 9.5%. Such variability is perfectly normal and underlines the importance of taking this into account properly in analysis.

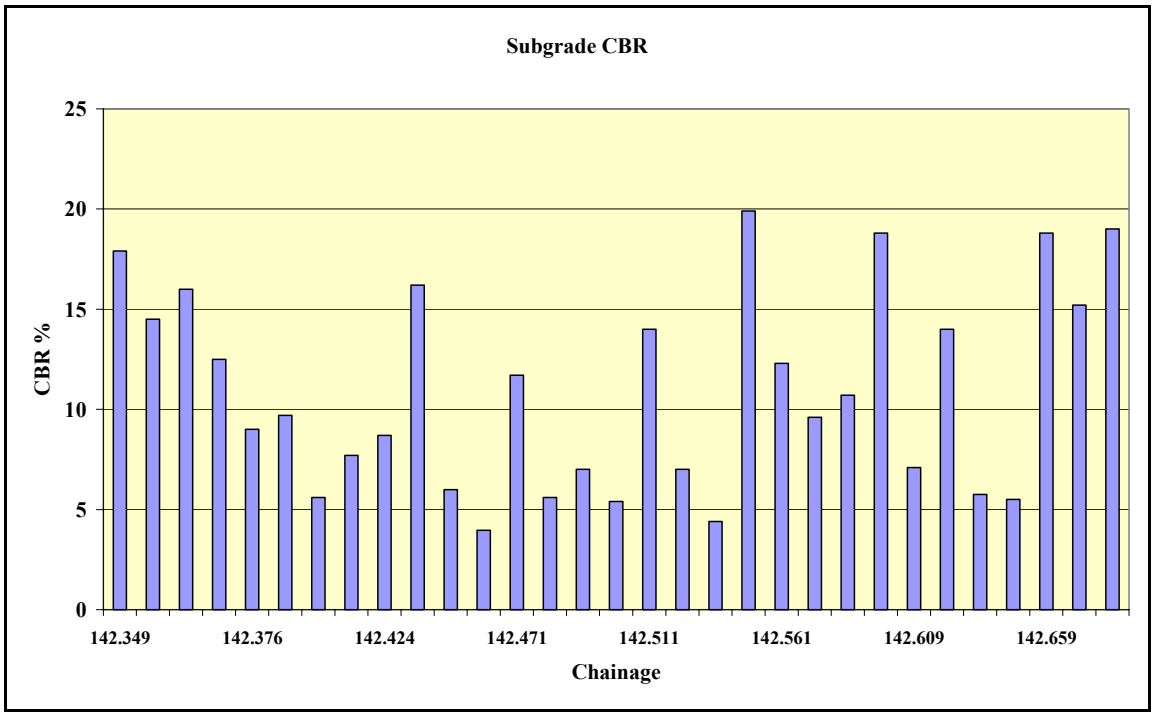

Figure 1 Subgrade Strength Distribution along the Site

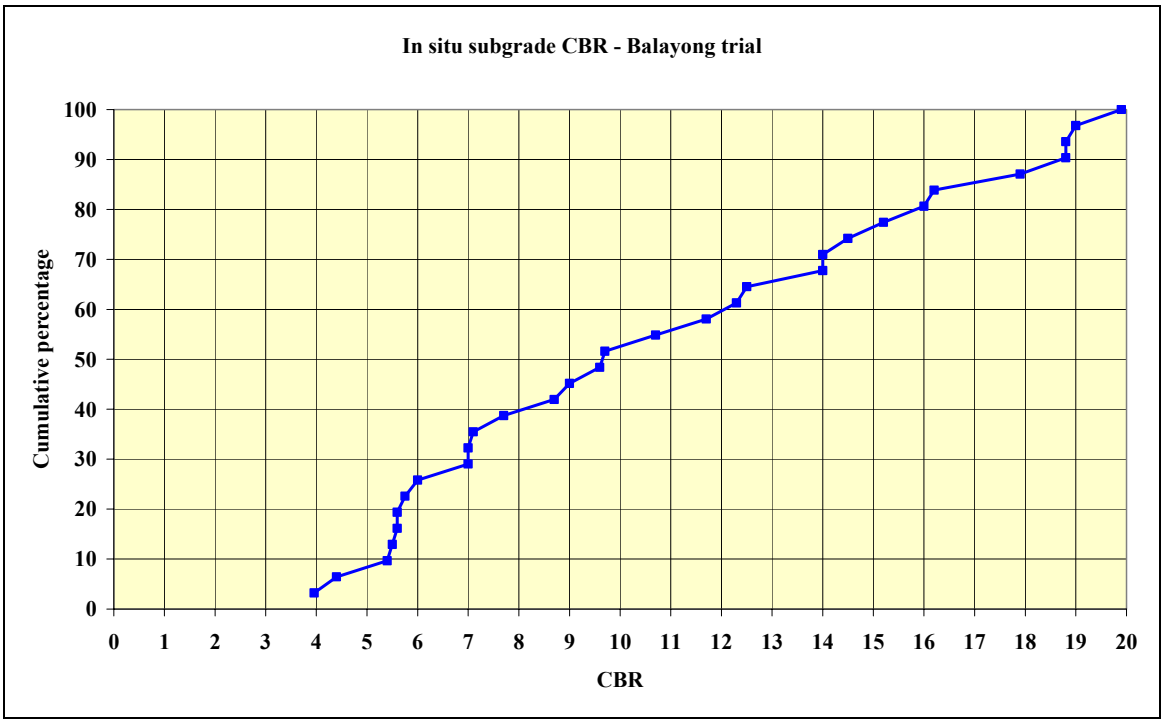

Figure 2 Distribution of Subgrade CBR's for Balayong Trial

The key to successful design is to take proper account of the *variability in both time and along the site* and this is essential when developing design methodologies, as we are doing in this project. Unfortunately it is rarely possible to do both of these in a scientifically robust manner simply because it is not possible to conduct a sufficiently comprehensive experiment

and to take sufficient measurements over a long enough period of time. Thus engineering judgement also has an important part to play in the final analysis. At this stage of the project there are very good results along the site at a particular time of the year from the DCP tests. There are also soaked CBR tests at the specified field density (7.7%), but only for one sample. In subsequent monitoring of the trial, the in situ strength will be measured again. In principle, back-analysis of FWD data could also assist with this, but the results obtained, so far, show that this was not very successful and are not reported in this paper.

4.2 Structural Number

The DCP analysis program automatically calculates the modified structural number and takes account of the reductions in contribution with depth as explained in the HDM 4 manual (HDM 4, 2002) based on the paper by Rolt and Parkman (2000). To distinguish this from the basic AASHTO structural number (SN) (AASHTO,1993) it is referred to as SNP. The stabilized sub-base had to be drilled out in Sections 2 and 3 and the SNP values calculated manually based on the unconfined compressive strengths measured on samples of the as-laid material. The SNP values along the site are shown in Figure 3.

The variability reflects two things. First it mainly reflects the variability of the subgrade strength. Secondly, the process of drilling out the stabilized sub-base can cause errors for various reasons. Sometimes the drilled hole is deeper than the depth of the layer and, under these circumstances, the sub-base appears to be thicker than it really is. Sometimes part of a layer can be weaker than expected because debris from the drilling operation partially fills the hole and gives a false, low reading. The data from the DCP tests were analysed with these issues in mind and with knowledge of the layer thicknesses from the construction details. Thus the final SNP values are best estimates based on these considerations.

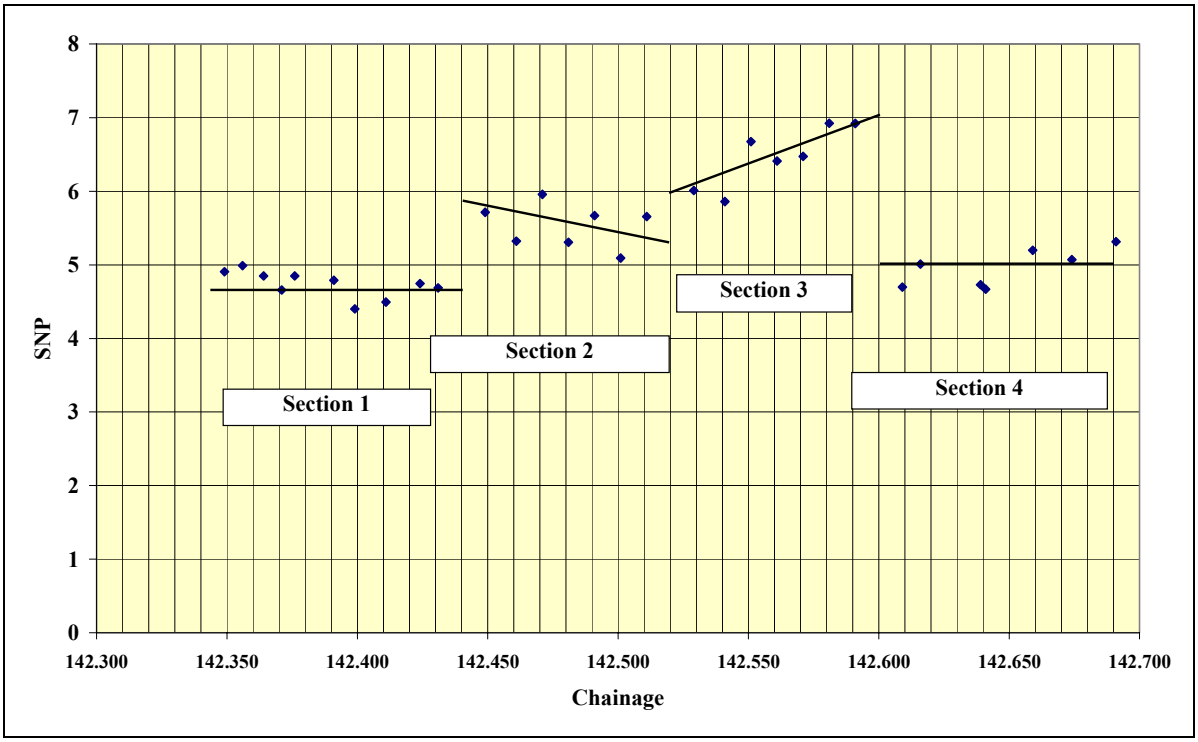

Figure 3 Structural Number (SNP) along the Site

The expected traffic carrying capacity has been estimated using the AASHTO method (AASHTO, 1993), based on the measured values presented herein and the following assumptions,

Change in PSI		=	2.5	
Reliability		=	90%	
(a)	Standard Deviation term S_o	=	0.5	when using mean values of SNP
(b)	Standard Deviation term S_o	=	0.0	when using the individual SNP values.

In Method (a) the value of S_o has to be 'guessed' by engineering judgement. Method (b) assumes that all the variability occurs in the subgrade and in the pavement layer thicknesses and strengths (all of which have been measured). [This assumption is true because traffic is the same on each section and so traffic variability is not an issue at this stage.]

The best way to analyse the data is on a point by point basis. The actual traffic capacity of each section (as a whole) is related to the chosen level of reliability and the statistical distribution of the point-specific data. For example, using 90% reliability we simply select the lower 10-percentile of the capacity distribution. Table 4 shows the results. It should be noted that Sections 2, 3 and 4 have systematically varying sub-base thicknesses and so the range of likely traffic is shown for Sections 2 and 3. The varying sub-base thickness of Section 4 is not apparent in the SNP values because it is swamped by variability in subgrade strength.

Table 4 Projected Traffic Capacity based on AASHTO (10^6 esa)

Section	Mean Value of SNP	Method (a) (10^6 esa)	Method (b) (10^6 esa)
1	4.74	4.0	10.5
2	5.53	13	30 - 75
3	6.47	46	90 - 300
4	4.95	5.5	17

For Method (a) to agree with Method (b) the value of S_o would have to be between 0.15 and 0.3. This is very low for normal construction practice but, of course, this is a very well-controlled trial. Method (b) is inherently the most accurate because no assumptions of variability are required.

The previous calculation makes use of the combined SNP value that includes the subgrade strength. The basic AASHTO method uses subgrade strength and SN separately. The effect of the subgrade is slightly different in the two methods. In the AASHTO method the subgrade contribution is higher than in the TRL method and so the projected traffic capacities will be higher. By way of comparison, the projected traffic capacity has also been calculated using the original AASHTO approach. Method (c) assumes,

Change in PSI = 2.5
Reliability = 90%
The Standard Deviation (S_o) is the measured values from the previous calculations namely 0.18, 0.22, 0.27 and 0.13 for Sections 1, 2, 3 and 4 respectively.

Table 5 shows the predicted traffic carrying capacity.

Table 5 Projected Traffic Capacity based on AASHTO (10^6 esa)

Section	Mean value of CBR	Mean SN	Method (c) (10^6 esa)
1	11.8	3.90	57
2	7.7	4.81	91
3	11.8	5.90	300+
4	12.2	4.31	150

At this stage we would expect method (b) to be the most reliable because it depends on actual SNP values measured at each test point and so the variability occurs in the SNP value itself rather than in separate components that make up its value and which could combine in unpredictable ways.

4.3 Deflection (FWD) Analysis

The deflection data were normalised to a load of 50KN and a temperature of 30°C. The central deflection at 5m intervals along the site is shown in Figure 4.

The deflection values clearly identify the sections with cement stabilized sub-bases as having higher effective elastic moduli than the control section with no stabilization. The strongest section is clearly Section 3 with the lowest deflections. However, the difference between Sections 2 and 4 is less clear. Unfortunately the contractor failed to report the unconfined compressive strengths achieved in Section 4 and so confirmation of the strength of the sub-base in this section will have to be obtained in a subsequent survey. Section 2 is slightly stronger than the target value and it looks as if Section 4 may also be stronger, however, as with structural number, the deflections also reflect the subgrade strength, hence some variability is to be expected.

The deflections do not give very much indication of the effect of the thickness of the sub-base. Indeed the lowest deflections in Section 2 actually occur at the thinnest end of the section but an increasing deflection is apparent from chainage 142+440 to 142+490 in Section 2 and possibly from 142+620 to 142+650.

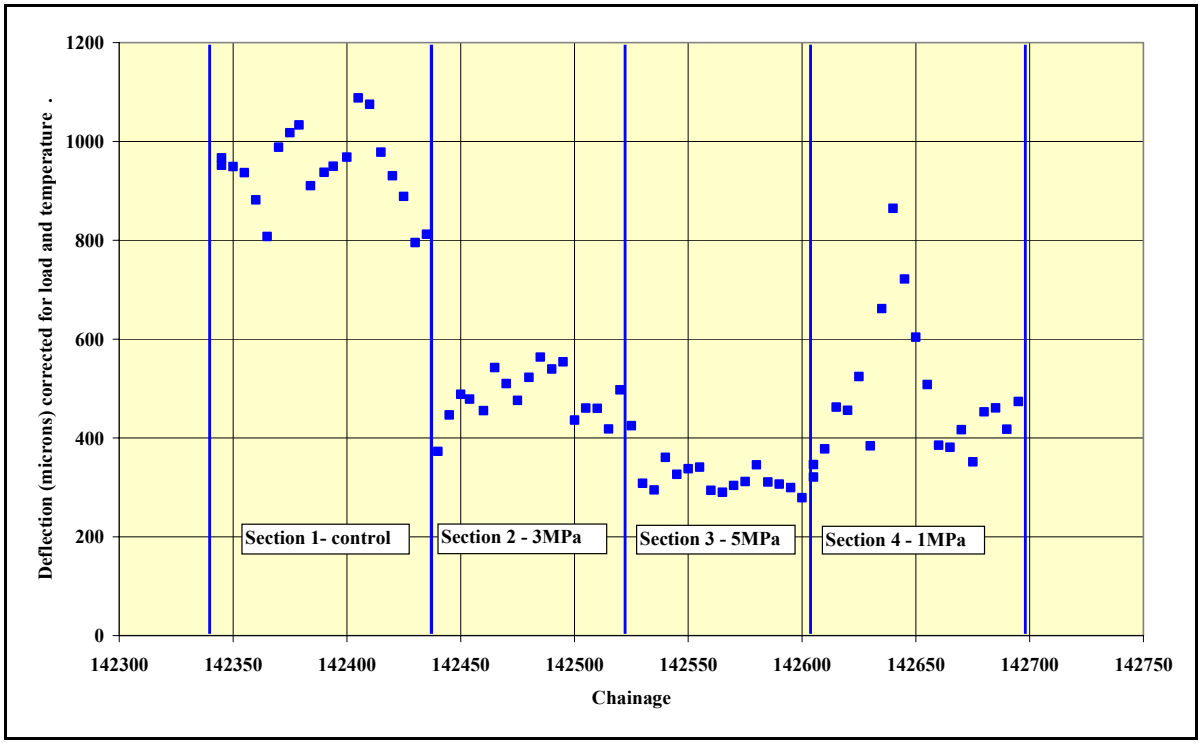

Figure 4 Central Deflections along the Site

Deflections on Section 1, the control section, are higher than expected. The relationship between SNP and deflection (shown in Figure 5 for many different pavements) almost always shows considerable scatter and deviations from the mean line are indicative of the likely long-term behaviour of the pavement. In this case the deflections for Section 1 are considerably higher than would be expected from the DCP tests or, conversely, the SNP values from the DCP tests are high for some reason. The measurements from Sections 2, 3 and 4 are all close to the 90% line on the graph. This line represents conditions where the measured SNPs and deflections show no anomalous behaviour; i.e. the layers are both strong and behave elastically so that load spreading is good and deflections are low. It is under these conditions that the traffic carrying capacities predicted from either deflections or SNP values are most likely to agree with each other.

In order to compare the likely performance of the road with empirical evidence from elsewhere it is necessary to convert the FWD central deflection to an equivalent deflection measured with a Benkelman beam because many previous empirical correlations between deflection and traffic carrying capacity were done using Benkelman beam deflections. Usually these were also carried out at a different load. Based on UK data this conversion is,

$$FWD = (BB - 40)/1.09 \quad (1)$$

Where FWD (microns) is measured at 50KN and BB (microns) is measured under an axle load of 62.3 KN. At this stage of this project it is not worthwhile making this conversion and so no attempt at predicting *absolute* performance from deflection data has been made in this report.

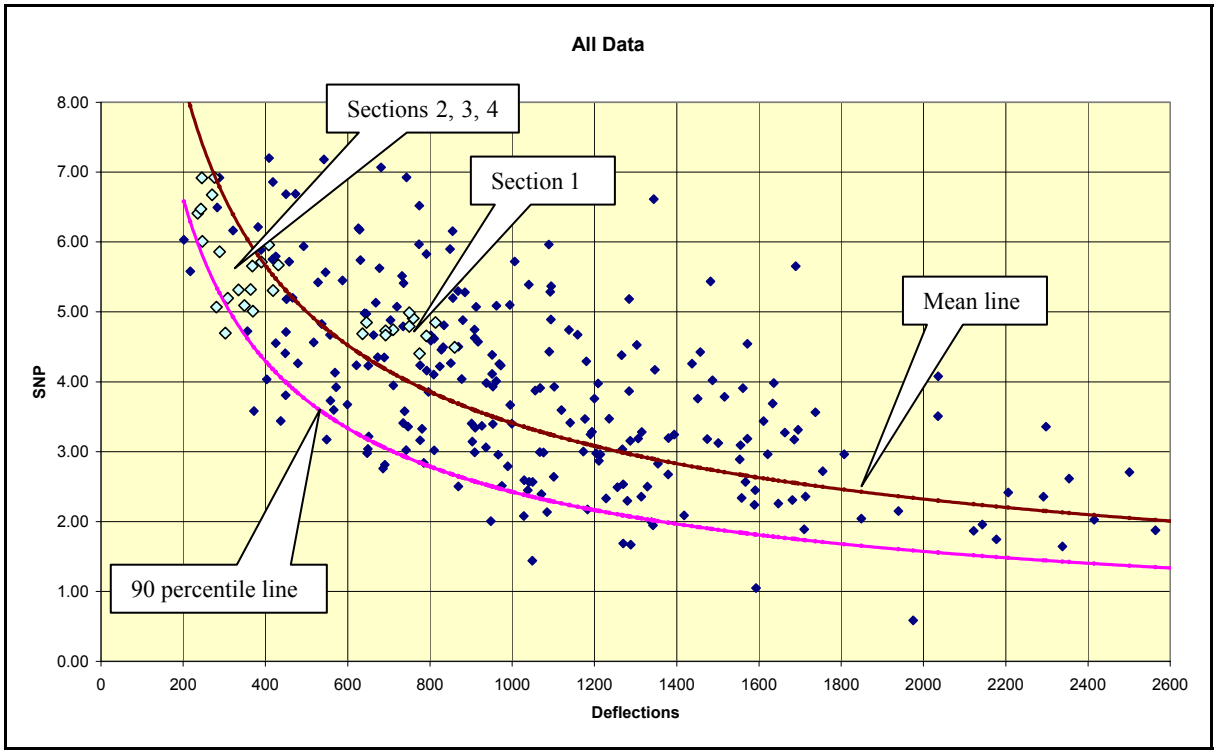

Figure 5 SNP from DCP Tests versus Deflection (the open diamonds are the Balayong site)

The relationship between deflection and traffic carrying capacity obtained from the empirical studies which led to TRL Report 833 (Kennedy and Lister, 1978) is

$$\text{Traffic capacity} = 1.852*(10^9)*(D_1^{-3.055}) \quad (2)$$

Where D_1 is the Benkelman beam deflection in microns, traffic is in millions of standard axles and the equation is for 90 percent reliability. If we use this equation to estimate the ratio of traffic carrying capacities between each section we obtain, very approximately, the ratios shown in Table 6, normalised to Section 3 at a value of 100.

Table 6 Comparative Traffic Carrying Capacities by Two Methods

Section	FWD Mean D_1	Estimated Traffic Capacity Ratios	
		FWD 90% Reliability	SN Approach Method (b)
1	1075	3.5	11.7
2	554	25	33
3	341	100	100
4	721	11.7	18.9

This compares with the ratios obtained from the structural number analysis as shown in the last column of Table 6. The poor agreement for Sections 1 is a consequence of the fact that

the SNP/FWD data do not plot on a smooth curve (Figure 5). The deflection values for Section 1 indicate a relatively short life whereas the SNP values indicate a reasonable life. Possible problems with the interpretation of DCP measurements or with the calculation of SNP may be to blame but the possibility is that Section 1 is showing anomalous results and requires additional site testing. Section 4 shows considerable variability in deflection values but less so for the smaller number of SNP values. High variability in subgrade strength on this section is the primary cause and, although the anticipated overall life of Section 4 is less than Section 2 there are parts of Section 4 that should perform as well as Section 2.

At this stage all of these estimates are very approximate indeed. The behaviour of cement-stabilized sub-bases is quite complex in that they exhibit two distinct phases of behaviour with a very long transitional phase in between. In the first phase the sub-base acts as a cemented 'slab' with shrinkage cracks at intervals but with good interlock between the 'blocks'. In this phase the deflections will be at their lowest. Eventually the interlock deteriorates and the blocks may break up, becoming smaller as the binding property of the cement is slowly lost through the tensile stresses that are developed. This is an indeterminate middle phase. Eventually the cemented slab becomes so fragmented that it behaves essentially like an unbound crushed stone layer. Even in this final state it is usually strong. Thus not only is the life of such a structure likely to be quite long, it is also very difficult to predict. In fact we would expect that once the UCS of the sub-base exceeds an initial critical value, the eventual failure of the road will not depend on the stabilized sub-base at all. This, of course, remains to be proven.

5. CONCLUSION

Considering, in particular, method (b) which is thought to be the most accurate, the analysis to date indicates that the use of stabilized sub-bases is likely to extend the service life of heavily trafficked roads, especially if a very strong stabilized sub-base (Section 3) is used. However, this prediction is based on the early performance of the experimental section and so, as expected, the period of the additional life cannot be more accurately determined until further monitoring, followed by additional analysis is completed. To achieve this additional life in the most economic manner, it is necessary to quantify the actual strengths and thicknesses required for a particular cumulative traffic loading. It will then be apparent to what extent the additional cost of cement (as the stabilizer in this case) can be offset by the use of thinner layers for the simple circumstances where road levels can be lowered or in the more complex case where the road level must be maintained and the difference is made up by the use of low cost local material. As well as considerations of construction unit costs, the benefits will be best quantified by whole life costing which takes into account the additional service life that is achieved and all the elements of the cost of transport, namely; construction, maintenance and road user costs.

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