EARLY PERFORMANCE REPORT: STABILIZED SUB-BASES, BALAYONG TRIAL, PHILIPPINES

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PREFACE

This report forms one of the deliverables from the DFID funded project on Stabilized Sub-bases for Heavily Trafficked Roads which is being undertaken as part of a co-operative research programme with the Bureau of Research and Standards, DPWH, Philippines. It also forms one of the deliverables under Project 3 within the overall Pavement Investigation Research project which is being undertaken with DPWH with the support of the Asian Development Bank.

The purpose of this component of the co-operative research project is:

> *To extend the service lives of flexible and rigid pavements by increasing the use of appropriate pavement design methods and material specifications.*

The objective of Project 3 is

> *To develop methods of using the indigenous materials in the Philippines so that they can be used with confidence for road construction and other civil engineering purposes.*

Under the pavement investigation research, project 3 is divided into two parts namely: 'the use of marginal materials for road construction' (project 3.1) and project 3.2, 'developing stabilised sub-base specifications for both flexible and rigid road pavements in the Philippines'.

The cost of road construction and associated environmental degradation can be greatly reduced if locally available materials, found near the road alignment, can be used in construction, thereby reducing the extraction and haulage of expensive high quality aggregates. Such materials may often be of marginal engineering quality in terms of standard specifications but, by modification and/or suitable design and construction methods, their use can be very cost effective. The methodology of the research is based on successful previous research on indigenous marginal materials in other countries.

Use of stabilized sub-bases for heavily trafficked roads, using materials of either marginal quality (originally) or those of better quality, can effectively counter: poor materials availability or selection; poor construction control; poor drainage and the general effects of the ingress of water. Used with unbound road bases in flexible pavements it can also prevent or reduce reflection cracking in the upper layers of the pavement and improve the overall service life of the pavement.

This report is one of 26 which are being delivered under this part of the pavement investigation research project. These reports are:

No	Title	Report code	Type
1	Identifying and mapping marginal materials in the Philippines		Project report
2	Distribution of gravel-sized and fine particulate materials from Mount Pinatubo, Philippines	PR/OSC/172/99	Project report
3	Outline design for pilot scale trial on the Zambales coastal road		Project report
4	Making good use of volcanic ash in the Philippines.	PA 3594/00	Conf. paper

5	The use of volcanic ash in bituminous mixes	PR/OSC/138/98	Project report
6	A study of the volcanic ash originating from Mount Pinatubo, Philippines	PR/INT/194/01	Project report
7	Investigation into the use of Lahar as fine aggregate in hot rolled asphalt and asphaltic concrete wearing courses	PR/INT/220/01	Project report
8	Specifications and guidance for construction: Pilot trials on the Nasugbu to Batangas City Road, Batangas Province: Lahar Asphaltic Concrete and Hot Rolled Asphalt (Station 96+665 to 96+994)		Project report
9	Outline design for pilot scale trials using weathered volcanic rock and soft limestone on the Malicboy to Macalelon road in Quezon Province.		Project report
10	Specifications and guidance for construction: Pilot trials on the Malicboy to Macalelon road, Quezon Province; site Agdangan		Project report
11	Agency estimate for a pilot trial on the Malicboy to Macalelon road, Quezon Province; site Agdangan		Project report
12	Construction report: Pilot trials on the Nasugbu to Batangas City Road, Batangas Province: Lahar Asphaltic Concrete and Hot Rolled Asphalt (Station 96+665 to 96+994)		Project report
13	Construction report: soft limestones and weathered volcanics as roadbases trial (Agdangan)		Project report
14	Performance of volcanic ash in bituminous mixes	PR/INT/282/04	Project report
15	Performance of marginal materials in roadbases: Soft limestones and weathered volcanics		Project report
16	Specification for using lahar and volcanic ash in bituminous mixes		Project report
17	Specifications for the use soft limestone		Project report
18	Specification on the use of weathered volcanics		Project report

PROJECT 3.2

19	Outline design for a pilot scale trial on the Nasugbu to Batangas City road.		Project report
20	Literature review: Stabilised sub-bases for heavily trafficked roads	PR/INT/202/00	Project report
21	Specifications and guidance for construction: Pilot trials on the Nasugbu to Batangas City Road, Batangas Province: Site B, Mabini Junction (Station 142+340 to 142 + 700)		Project report
22	Specifications and guidance for construction: Pilot trials on the Nasugbu to Batangas City Road, Batangas Province: Site A, Santa Teresita pilot trial (Station 135+450 to 135+610)		Project report
23	Construction report: stabilized sub-bases (Bauan/Balayong trial)	PR/INT/281/04	Project report
24	Performance report: stabilized sub-bases Balayong trial		Project report
25	Final report		Final report
26	Guidelines on Stabilised Sub-bases		Project report

EARLY PERFORMANCE REPORT: BALAYONG/BAUAN PILOT TRIAL STATION 142+340 TO 142+700

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EARLY-PERFORMANCE REPORT: BALAYONG/BAUAN PILOT TRIAL STATION 142+340 TO 142+700

1 INTRODUCTION

The pilot trial described in this document forms part of the Pavement Investigation Research Project (PIR) which is being carried out under the Sixth Road Project (SRP), ADB Loan No. 1473-PHI. The overall objective of the PIR is to implement a programme of research aimed at improving the performance of road pavements in the Philippines, through a better understanding of the available materials and the transport demands, and the adaptation of modern techniques to the Philippine climate and traffic.

The pilot trial described herein forms part of Project 3-2 which addresses the use of stabilized sub-bases for heavily trafficked roads. This is one of two pilot trials that have been constructed to investigate the performance of stabilized sub-bases with respect to their technical suitability and cost. The trial is located on the SRP project LZH-D; the structural overlay of the Nasugbu- Palico-Batangas City road. A location map is shown in Plate 1.

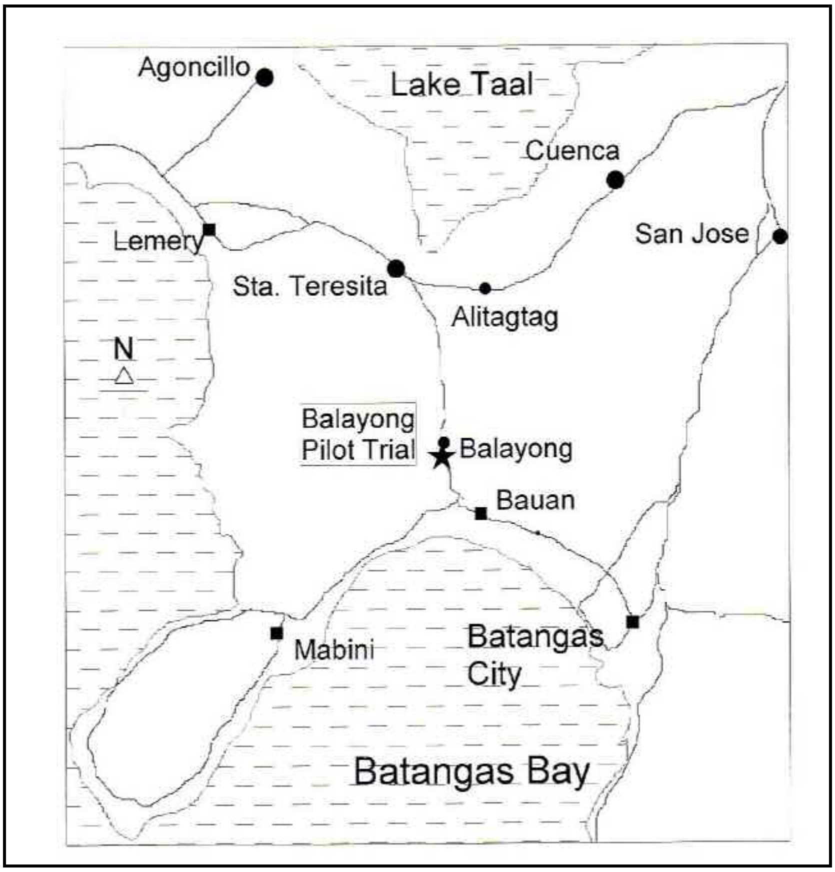

Plate 1 Location of Balayong trial, Batangas Province

The design of the trials, the construction specifications and details of construction itself have been described in previous reports numbered 19 to 23 in the list above namely,

19 Outline design for a pilot scale trial on the Nasugbu to Batangas City road
20 Literature review: stabilised sub-bases for heavily trafficked roads
21 Specifications and guidance for construction: pilot trials on the Nasugbu to Batangas City Road, Batangas Province: site B, Mabini Junction (Station 142+340 to 142 + 700)
22 Specifications and guidance for construction: pilot trials on the Nasugbu to Batangas City Road. Batangas Province: site A, Santa Teresita pilot trial (Station 135+450 to 135+610)
23 Construction report: stabilized sub-bases (Bauan/Balayong trial)

In this report only those details from the previous reports that are necessary for understanding the performance analysis are repeated. For full details the reader should consult the previous reports.

2 PROPERTIES OF THE TRIAL MATERIALS

2.1 Trial design

The trial comprises four sections on the left-hand side of the road. The right hand side makes a fifth section. A description of the basic components of the trial is given in Table 2.1.

Table 2.1 Experimental sections

Section	Lane	Target UCS of stabilized sub-base	Thickness of sub-base	Road base	Surfacing	Station, Km		Length
						from	to	m
1	Left	Control section	350	200mm crushed stone	100mm AC	142+340	142+440	100
2	Left	3 MPa	200-350	200mm crushed stone	100mm AC	142+440	142+520	80
3	Left	5 MPa	200-350	200mm crushed stone	100mm AC	142+520	142+600	80
4	Left	1MPa	200-350	200mm crushed stone	100mm AC	142+600	142+700	100
5	Right	Standard works		250mm cement treated	100mm AC	142+340	142+700	360

The stabilized sub-base trial sections have been constructed in the left or westbound lane with necessary expansion into the right or eastbound lane of the road of approximately 0.6 metres. The works involved the complete removal of the existing asphalt concrete surfacing and existing cement treated base. Thereafter, the sub-base was excavated and stockpiled. In Sections 2, 3 and 4 it was processed with the appropriate quantity of cement and replaced. In Section 1 it was replaced with an aggregate sub-base.

In three of the experimental sections the subgrade level was varied to accommodate the varying thickness of stabilized sub-base.

The specifications for the construction of the trial were those in the DPWH Standard Specifications for Highways, Bridges and Airports, Volume II 1988 and as modified by the special provisions of Contract LZH-D. Although the newer 1995 specifications (DPWH 1995) were available, the 1988 version was chosen because it was in use for the construction under the Contract for the main works. The modifications to the Standard Specifications that pertain to this pilot trial can be found in report number 23.

The quality of materials for the surfacing and the road base met or exceeded (by design) those required in the DPWH Specifications for these layers. The unbound layer used for the sub-base in the control section was also designed to exceed the standard specifications by increasing the CBR requirement from 25 to 30%.

The experimental layout is shown in the drawings in Appendix A.

2.2 Subgrade

The properties of the subgrade are shown in Table 2.2.

Table 2.2 Properties of the blended subgrade

Property	Value
Liquid Limit	50%
Plastic Limit	30%
Plasticity Index	20%
Percent passing 75μm	74.4%
Type Class	A-7-5
Maximum Dry Density (T180)	1.48 Mg/m^3
Optimum Moisture Content	22%
CBR at 95% of MDD (soaked)	8%

Results from the field density tests carried out after compaction are given in Table 2.3.

Table 2.3 Field density results for the blended subgrade used on the left side

Laying date (estimated)	Lane	Section	Layer	Representing station: from Km	to Km	length m	Test Location Km	Off set from CL m	Results Relative Density %	Rel. MC %
12-Apr-02	L	1	1	142.340	142.440	100	142.404	3.2	95.7	88.6
12-Apr-02	L	1	1	142.340	142.440	100	142.360	1.0	94.7	83.2
12-Apr-02	L	1	1	142.340	142.440	100	142.460	1.7	96.6	87.3
29-Apr-02	L	2	1	142.440	142.520	80	142.455	2.7	99.7	86.1
29-Apr-02	L	2	1	142.440	142.520	80	142.480	3.0	97.8	85.9
29-Apr-02	L	2	1	142.440	142.520	80	142.505	2.8	100.5	84.5
17-May-02	L	3	1	142.520	142.600	80	142.535	1.2	100.4	90.0
17-May-02	L	3	1	142.520	142.600	80	142.560	3.1	105.2	81.5
17-May-02	L	3	1	142.520	142.600	80	142.535	3.3	106.4	86.3
25-May-02	L	4	1	142.600	142.660	60	142.670	3.1	99.7	86.7
25-May-02	L	4	1	142.600	142.660	60	142.650	2.5	100.1	75.0
25-May-02	L	4	1	142.600	142.660	60	142.650	1.0	98.9	80.8
14-Jun-02	L	4	1	142.660	142.700	40	142.680	2.1	99.5	94.9
4-Jun-02	R	5	1	142.365	142.415	50	142.320	1.0	99.5	82.2
4-Jun-02	R	5	1	142.365	142.415	50	142.400	2.1	95.9	80.9
7-Jun-02	R	5	1	142.340	142.365	25	142.350	2.1	95.2	67.4
7-Jun-02	R	5	1	142.415	142.455	40	142.435	3.0	97.1	76.1
7-Jun-02	R	5	1	142.415	142.455	40	142.450	2.5	99.6	74.3
11-Jun-02	R	5	1	142.455	142.520	65	142.470	2.1	98.3	77.4
11-Jun-02	R	5	1	142.455	142.520	65	142.500	3.0	99.7	63.2
21-Jun-02	R	5	1	142.520	142.580	60	142.535	1.8	91.9	146.9
21-Jun-02	R	5	1	142.520	142.580	60	142.570	2.7	93.6	73.4
?	R	5	1	142.580	142.700	120	??	??	??	??

2.3 Unstabilized sub-base

The engineering properties of the blended sub-base used in Section 1 are given in Table 2.4 and the grading is given in Figure 2.1.

Table 2.4 Properties of blended unstabilized sub-base material

Engineering Property	Value
Liquid Limit	18%
Plastic Limit	11%
Plasticity Index	7%
Percent passing 75µm	9.6%
Type Class	A-2-4 (0)
Maximum Dry Density (T180)	1.98 Mg/m^3
Optimum Moisture Content	11.8%
CBR at 95% of MDD	28%
Swell at 100% MDD	2.2%

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Figure 2.1 Grading of the blended sub-base used for Section 1

The blended material was processed to the optimum moisture content at the stockpile. Placement extended across the shoulder to the side-slope, as required in the design. The sub-base was constructed in two equal layers each 175mm thick (compacted).

At the design density of 95% of the MDD obtained in the ASTM T180 compaction method, the material had a CBR of 28% in the soaked condition. This is marginally lower than the design CBR of 30%. The material also exhibited swelling while being wetted from the compaction moisture content to the soaked moisture content. The reported swell was 2.2%.

The densities obtained after compaction in the field are shown in Table 2.5. The minimum density obtained in the field was 97% relative to the heavy compaction method, ASTM T180.

Table 2.5 In-situ density of unstabilized sub-base

Laying date	Lane	Section	Layer	Representing station:			Test Location		Results	
				from Km	to Km	length m	Km	Off set from CL m	Relative Density %	Rel. MC %
15-Apr-02	L	1	1	142.340	142.440	100	142.351	1.0	100.2	59.3
15-Apr-02	L	1	1	142.340	142.440	100	142.382	3.0	100.1	71.2
15-Apr-02	L	1	1	142.340	142.440	100	142.429	2.8	99.2	81.4
17-Apr-02	L	1	2	142.340	142.440	100	142.363	2.1	98.8	66.9
17-Apr-02	L	1	2	142.340	142.440	100	142.390	2.2	97.0	98.3
17-Apr-02	L	1	2	142.340	142.440	100	142.420	3.0	98.5	80.5

2.4 Stabilized sub-bases

Construction of the stabilized sub-base sections was carried out by re-using the existing sub-base material from the work site. The material was excavated and transported to the batching plant area at Banilad and stockpiled ready for processing with cement. The raw material was tested and found to be a well-graded, non-plastic, sandy gravel. The maximum dry density was 1.816 Mg/m^3 and the optimum moisture content was 14.5 per cent. The grading is shown in Figure 2.2. Strength and therefore cement requirements had been determined in the design of the pilot trial. Table 2.6 gives the design requirements.

Table 2.6 Target unconfined compressive strengths for sub-base from laboratory design

Requirement	Unit	Properties		
		Sect 4	Sect 2	Sect 3
Designed UCS	MPa	1	3	5
Cement required	% by weight	1	3	5
Volume of material	m^3	172	138	138
Quantity of cement per section	40 Kg bags	61	147	245

Note that the section lengths differ

Using material sampled from the stockpile at the batching area, cement was added and specimens were formed to confirm each of the design blends. A summary of the test results compared with each of the target strengths is given in Table 2.7.

Table 2.7 Unconfined compressive strengths of sub-base achieved during construction

Section	Target UCS (MPa)	Actual UCS (MPa)	Relative Dry Density %
2	3	3.99	96.0
3	5	6.53	94.1
4	1	2.01	96.6

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Figure 2.2 Grading of the stabilized sub-base material

Thereafter, to determine the strength of the materials that were being laid, the stabilized materials were sampled from the truck loads of batched material before they left the batching plant. A summary of the strengths obtained on these materials is given below in Table 2.8. Data for Section 4 (target 7-day strength of 1MPa) was not provided by the contractor.

The sub-bases were constructed in two layers. To achieve the required thickness variations, 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. The two layers were bonded together by applying a cement slurry immediately before placement of the second layer.

The field density results for the stabilized sub-bases are shown in Table 2.9. Once compaction was completed, the surface was covered with plastic sheeting to provide a temporary curing membrane over the exposed works for a period of 7 days or until the next (upper layer) was placed.

Table 2.8 Sub-base strengths (7 day cure) achieved during construction

Laying date	Section	Layer	Representing station			UCS strength MPa
			From	To	Length (m)	
30 Apr 02	2	1	142.440	142.520	80	4.5
30 Apr 02	2	1	142.440	142.520	80	4.1
30 Apr 02	2	1	142.440	142.520	80	4.66
30 Apr 02	2	1	142.440	142.520	80	4.28
01 May 02	2	2	142.440	142.520	80	3.77
01 May 02	2	2	142.440	142.520	80	2.83
17 May 02	3	1	142.520	142.60	80	5.27
17 May 02	3	1	142.520	142.60	80	5.37
18 May 02	3	2	142.520	142.60	80	5.37
18 May 02	3	2	142.520	142.60	80	5.32

Some complications occurred during the construction of Section 2. Field density testing of the second layer of stabilized sub-base in Section 2 indicated that the material was loose and broke out of the density hole too easily. It was considered to be unstabilized. This was attributed to either delays in delivery or low moisture contents during batching, or both. On 6th May, 2002, the contractor agreed to re-stabilize the second layer. Seventy-five bags of cement (each 40kg) were required. The work was carried out on 8th May, 2002. The in situ material was ripped using a grader and the bags of cement were spotted over the area. Blending was completed by a grader, then compaction was carried out. Cement slurry was *not* used to re-bond the newly processed layer with the lower layer.

Table 2.9 Field density results for stabilized sub-base

Laying date	Lane	Section	Layer	Representing station:			Test Location		Results	
				from Km	to Km	length m	Km	Off set from CL m	Relative Density %	Rel. MC %
30-Apr-02	L	2	1	142.440	142.520	80	142.450	2.8	99.7	54.8
30-Apr-02	L	2	1	142.440	142.520	80	142.480	1.2	99.0	53.8
2-May-02	L	2	2	142.440	142.520	80	142.510	2.1	97.8	55.3
2-May-02	L	2	2	142.440	142.520	80	142.480	1.5	95.8	60.2
2-May-02	L	2	2	142.440	142.520	80	142.350	2.7	101.0	72.3
9-May-02	L	2	2	142.440	142.520	80	142.460	2.3	100.8	79.0
9-May-02	L	2	2	142.440	142.520	80	142.490	1.0	98.3	72.3
9-May-02	L	2	2	142.440	142.520	80	142.510	3.2	99.1	60.8
17-May-02	L	3	1	142.520	142.600	80	142.530	2.3	100.3	59.8
17-May-02	L	3	1	142.520	142.600	80	142.565	1.2	102.6	81.1
17-May-02	L	3	1	142.520	142.600	80	142.580	3.1	97.6	75.5
18-May-02	L	3	2	142.520	142.600	80	142.534	3.0	100.6	77.9
18-May-02	L	3	2	142.520	142.600	80	142.571	2.0	99.8	80.5
28-May-02	L	4	1	142.600	142.660	60	142.615	2.1	100.5	88.4
28-May-02	L	4	1	142.600	142.660	60	142.650	2.0	97.7	75.0
29-May-02	L	4	2	142.600	142.660	60	142.610	1.5	103.4	88.2
29-May-02	L	4	2	142.600	142.660	60	142.645	2.9	99.6	88.5
18-Jun-02	L	4	1	142.660	142.700	40	142.685	1.0	96.9	76.4
18-Jun-02	L	4	1	142.660	142.700	40	142.665	2.1	97.6	69.8
19-Jun-02	L	4	2	142.660	142.700	40	142.678	1.0	99.4	95.5

2.5 Crushed aggregate base course

The approved blend and the properties of the individual materials are summarised in Table 2.10. The road base was constructed in one layer. Compaction was carried out using both vibratory and pneumatic-tyred rollers until the specified density was achieved (Table 2.11).

Table 2.10 Blend and properties of the graded crushed stone base

Property	Specification	AASHTO	Aggregates(Rockworks)			Sand	Lime
		Test	G1	3/4	3/8	Crushed	Agricultural
Blend	%		30	23	15	30.5	1.5 (1)
Specific G (2)	NA	T85 (2)	2.690	2.680	2.524	2.603	2.467
Absorption	NA		1.380	1.630	2.970	1.960	NA
Soundness (3)	<12	T104-77	5.8	5.4	6.95	7.2	
Abrasion	< 45%	T96	24.4	26.6	27.9	NA	NA
Fractured faces	100%		100	100	100	NA	NA
Plasticity	NP	T27	NP	NP	NP	NP	

Notes (1) Of total weight of dry blended aggregates
(2) Oven dry
(3) Sodium Sulphate Soundness test

Table 2.11 Crushed stone road base field densities achieved

Laying date	Lane	Layer	Representing station:			Test Location		Results	
			from Km	to Km	length m	Km	Off set from CL m	Relative Density %	Rel. MC %
3-May-02	L	1	142.340	142.440	100	142.350	1.2	95.8	85.4
4-May-02	L	1	142.340	142.440	100	142.360	1.2	102.8	89.5
7-May-02	L	1	142.340	142.440	100	142.380	3.0	100.0	94.0
7-May-02	L	1	142.340	142.440	100	142.383	1.3	100.0	81.2
7-May-02	L	1	142.340	142.440	100	142.390	1.3	100.3	75.0
7-May-02	L	1	142.340	142.440	100	142.410	2.9	101.7	95.3
7-May-02	L	1	142.340	142.440	100	142.410	2.9	101.8	92.8
7-May-02	L	1	142.340	142.440	100	142.430	2.1	100.9	80.9
7-May-02	L	1	142.340	142.440	100	142.430	2.1	100.6	86.0
7-May-02	L	1	142.340	142.440	100	142.430	3.0	100.3	88.5
9-May-02	L	1	142.440	142.520	80	142.450	2.9	103.0	89.3
9-May-02	L	1	142.440	142.520	80	142.455	2.9	103.3	86.0
3-May-02	L	1	142.440	142.520	80	142.480	1.2	104.7	83.4
9-May-02	L	1	142.440	142.520	80	142.480	2.9	106.5	78.1
9-May-02	L	1	142.440	142.520	80	142.480	2.9	105.9	87.1
9-May-02	L	1	142.440	142.520	80	142.510	3.0	100.3	94.3
9-May-02	L	1	142.440	142.520	80	142.510	3.0	100.0	95.3
21-May-02	L	1	142.520	142.600	80	142.530	2.0	101.2	94.3
21-May-02	L	1	142.520	142.600	80	142.550	3.0	102.3	84.7
21-May-02	L	1	142.520	142.600	80	142.580	2.4	102.3	91.3
6-Jun-02	L	1	142.600	142.660	60	142.611	2.1	100.6	94.3
6-Jun-02	L	1	142.600	142.660	60	142.640	2.7	101.4	96.9
19-Jun-02	L	1	142.665	142.700	35	142.685	1.6	102.9	109.1

2.6 Bituminous surface course

The bituminous mix design used for the Asphalt Concrete wearing course was the same as that approved for the main Contract LZH-D. In accordance with the mix, the grading of the aggregates were to conform to Grading B of the Standard Specifications and the asphalt cement content was between 5 and 7% of the total mix by weight. Samples were taken from the paver on each day of laying to determine the properties of the actual mixed material being laid. The results of the Marshall testing are shown in Table 2.12. The standard test used 75-blow compaction.

Table 2.12 Marshall properties on the laid material

Laying date 2002	AC (Bitumen) content %	Flow (avg) mm	Air voids %	VMA %	VFB (VFAC) %	Stability		
						0.5 hrs Kg	24 hrs Kg	Loss %
10 May	5.71	3.1	3.5	14.8	76.1	1670	1288	22.9
23 May	5.69	3.1	3.5	14.8	76.4	1606	1328	17.3
1 Jun	5.73	3.1	3.8	14.9	74.6	1612	1302	19.2
5 Jun	5.61	3.2	3.5	14.7	76.0	1671	1301	22.1
7 Jun	5.78	3.0	3.8	14.9	74.7	1580	1227	22.3
8 Jun	5.76	3.1	3.8	15.0	74.5	1593	1294	18.8
14 Jun	5.69	3.1	4.0	14.7	72.8	1233	1023	17.0
24 Jun	5.56	3.9	3.9	14.7	73.8	1252	1007	19.6
25 Jun	5.66	3.1	3.8	14.8	74.6	1214	1041	14.3
17 Jul	5.63	3.5	3.8	14.5	73.6	1436	1250	13.0

The asphalt concrete wearing course was constructed in two layers, each 50mm thick. Cores were taken after construction and the thickness and density of the compacted pavement (relative to the bulk specific gravity : AASHTO –T166) were obtained. These are given in Table 2.13.

Table 2.13 Core thickness and density

Date 2002	Lane	Representing station		Core location Km	Thickness mm	Compaction %
		From Km	To Km			
10 May	L	142.340	142.440	142.360	50	98.6
10 May	L	142.440	142.540	142.460	55	99.9
10 May	L	142.540	142.600	142.560	67	99.8
8 Jun	R	142.340	142.700	142.350	51	99.1
14 Jun	R	142.340	142.700	142.450	46	99.4
25 Jun	R	142.340	142.700	142.550	51	99.2
26 Jul	R	142.340	142.700	142.650	60	98.8

2.7 Eastbound traffic lane (to Bauan)

Details of the construction works carried out on the eastbound lane can be found in the construction report.

3 PERFORMANCE MONITORING

The primary purpose of the monitoring and analysis is 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 (as well as on the other variables that affect structural design and performance (subgrade strength, reliability, and so on)).

The purpose of a pavement is to prevent failure of the subgrade caused by traffic loads and therefore a fundamental requirement of the pavement structure is to reduce the magnitude of the loads imposed on the subgrade to tolerable levels. This is the ‘load spreading’ function of the pavement and it depends simply on the thickness and the elastic modulus of each layer. The transient deflection of the pavement under a standard load is a very good measure of the load spreading ability of the structure.

At the same time the layers of the pavement must not fail in any way. Thus the loads being applied at the critical points in the pavement layers must also be kept to safe levels. Deflection is not a good measure of this because a single stiff layer may reduce deflections to a low level but may attract high levels of stress or strain in doing so. Thus the pavement needs to be ‘balanced’ so that this does not occur and the strength (usually the shear strength or flexural strength but also fatigue properties) of each layer needs to be sufficient to prevent failures in the pavement layers. For unbound materials the most common method of measuring ‘strength’ is the CBR test. The CBR values of materials under standard test conditions have been used in specifications for many years and non-destructive tests such as the dynamic cone penetrometer (DCP) test have been correlated successfully with CBR values for assessing the ‘strength’ of unbound materials in existing roads (TRL, 1999). 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. [For a more analytical approach, fatigue properties are also important but these cannot be measured easily.]

Although elastic modulus tends to increase as ‘strength’ increases, the correlation between the two is rather poor. This is a pity because it means that pavement engineers have to be concerned with both separately.

The Structural Number (SNP) of a road pavement (a concept developed during the AASHO Road Test in the USA in early 1960’s) is a useful measure of overall strength of a pavement and is easily calculated from DCP measurements (for unbound layers) and other strength measures for bound layers.

In order to illustrate the need for both strength and elastic modulus measurements, Figure 3.1 illustrates the relationship between SNP and deflection obtained from tests on many different roads. The variability shows that road pavements can be strong (high SNP) but also have poor load spreading ability (high deflections) and, conversely, relatively weak road structures may spread load quite well. The lower 90% line shown in the Figure represents ‘good’ pavements where the load spreading is high i.e. good interlock and high modulus but the thicknesses and strengths (SNPs) are relatively low. Thus in assessing a road, both SNP and deflections need to be considered. Use is made of this idea in the analysis which follows.

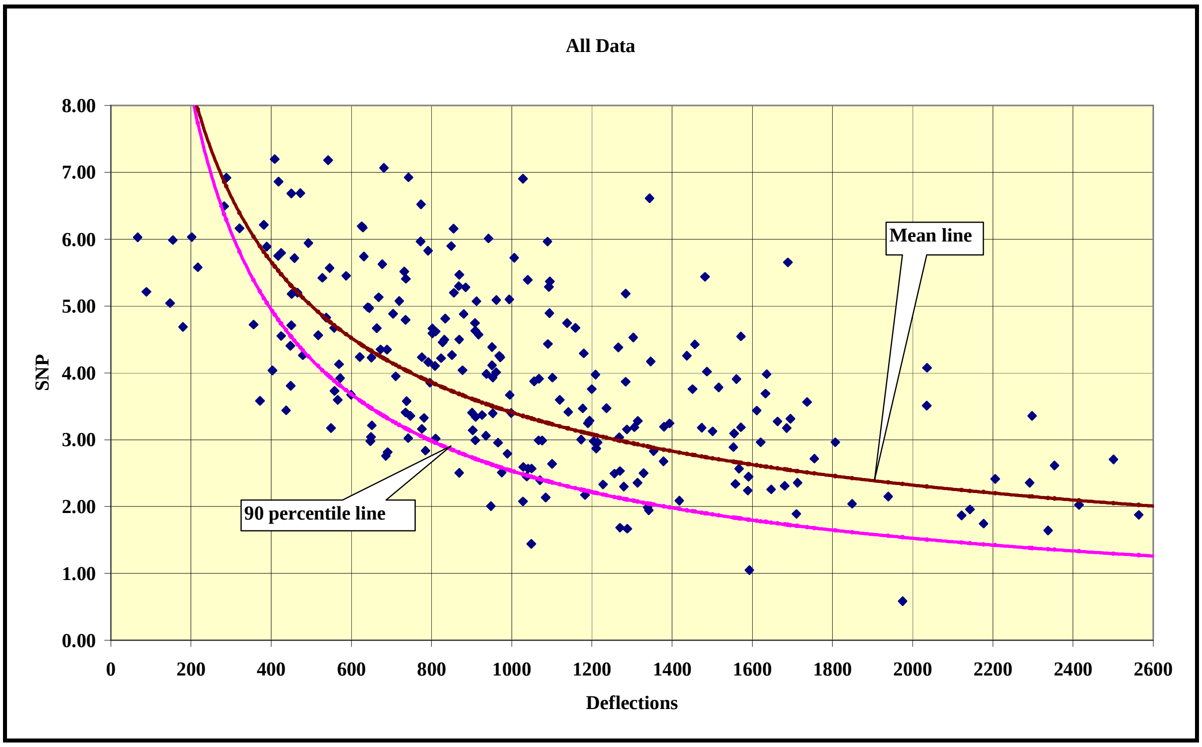

Figure 3.1 Relationship between deflection and structural number

3.1 Surface condition

Surface condition can reflect both structural deterioration, which generally affects the whole pavement, and also the condition of just the surfacing, which may simply be 'wearing out' (e.g. reduced skid resistance, reduced texture depth, age-related cracking). The trial pavements are only two years old and so structural deterioration is not expected, nor has it been observed on Sections 2, 3 and 4. However, some deterioration has been observed on part of Section 1. This has been investigated and is described in Chapter 5.

Although surface condition is important, the surfacing of the trial is a standard mix and any deterioration of this that is not structural (e.g. skid resistance, texture depth etc.) is not of direct relevance; in other words it is nothing to do with the effect of the stabilized sub-base and therefore not relevant to this analysis. Nevertheless, the surface condition has been monitored as a matter of routine but is not discussed in this report.

3.2 Traffic

The traffic volume and loading on the site was obtained from manual classified counts and an axle load survey carried out in 2000, and from automatic classified traffic counts carried out in March 2004. Seasonality factors obtained from the KAMPSAX study (DPWH 1995) were applied to the data. Sixty-five per cent of the heavy vehicles were estimated to be loaded, again, in accordance with the findings of the KAMPSAX study. An equal number of vehicles travel in each direction. The classified average annual daily traffic AADT (both directions) is given in Table 3.1. Motor tricycles are included in the number of light vehicles. The annual loading (2004) in one direction (over the experimental site) is estimated to be 1.2 million equivalent standard axles.

Table 3.1 Traffic volume, 2004

Light vehicles	Large Buses	2-axle trucks	3-axle trucks	4-axle trucks	5-axle trucks	AADT
5,856	93	375	592	29	112	7,056

3.3 Deflection testing

Deflections were measured using a Phoenix (now Carlbro) falling weight deflectometer (FWD). The nominal test load was 50kN, giving a plate pressure of 700kN/m^2, but individual results differed slightly from this and so each result was normalised using linear scaling.

3.3.1 Temperature corrections

In the Philippines the road temperature usually increases quite considerably during the normal period of the day when measurements are being made. This raises the question as to whether temperature corrections should be applied to the basic data. This, in turn, depends on the purpose of making the deflection measurements and the type of analysis that is to be done. In general, deflections will increase with temperature as the stiffness of the asphalt layer decreases, thereby transmitting a higher load to the layers below. In order to make direct empirical comparisons with data measured at different times of day, it is common practise to normalise the data to a standard temperature. This is done by repeating a number of the measurements at set points throughout the day, as the temperature increases, and developing a relationship between deflection and temperature.

Unfortunately such a relationship depends on the age of the bitumen and the thickness of the asphalt layer, amongst other things, and therefore universal models have not been developed. Thus each time measurements are made it is important to repeat some of them at different temperatures to measure the temperature effect. Fortunately it is often found that the temperature effect is small and often negligible. In this project temperature corrections were deemed necessary. The derivation of a temperature correction methodology for the FWD deflection bowl is shown in Appendix E. All results have subsequently been subject to such a correction using a standard temperature of 30^{o}C.

3.3.2 Back-analysis of deflection bowl

The purpose of back-analysis is to derive the elastic modulus of each of the pavement layers. Unfortunately, despite much research effort on the subject, back-analysis remains fraught with difficulties. For example, only pavement layers that substantially affect the load spreading ability of the pavement will make a sufficient contribution to the deflection value to enable the properties of that layer to be measured. Thus thin or weak layers do not make a sufficient contribution to the shape of the deflection bowl and so their elastic properties cannot be assessed very accurately.

A related difficulty arises when one of the layers is stabilized with cement. Such a layer makes such a high contribution to the deflection values that deducing the properties of the other layers becomes difficult.

Finally, the well known problem posed by materials that do not behave in a linear elastic manner, as assumed in almost all back-analysis programs, also prevents satisfactory interpretation of the deflection data. This is a particularly serious problem because the analysis programs appear to calculate the modulus values in a mathematically satisfactory manner but the values obtained can be very erroneous indeed, especially for sub-base layers. Nevertheless, for a research project of this kind it is always a good idea to examine the data using as many different levels of analysis as is practicable because some valuable insights might be made during the analysis process that will help to improve the interpretation of FWD data.

The FWD data at the basic level allows comparisons with empirically derived performance data relating deflection to service life, as discussed in Section 4 below. In principle the data also enable a more theoretical approach to the analysis of likely performance but, although promising, the behaviour of roads containing cement stabilized layers is too complex for analytical analysis to be much more than an interesting academic exercise.

3.4 DCP tests

The second and very valuable method of assessing the overall strength of a pavement and examining its behaviour and likely long-term performance is by means of the DCP test. However, although DCP testing has become well established, it, too, is not without its pitfalls and these are described in Appendix F.

For this project a series of DCP tests were made along the trial site adjacent to some of the deflection test points. Whenever a pavement layer was found to be too strong for the DCP cone to penetrate, a hole was drilled through the layer and the DCP testing continued through the hole. The analysis of the data was carried out using the TRL DCP (TRL, 1999) analysis program (TRL, 2004) and the results are shown in Appendix C. Whenever it was necessary to drill through a layer, the automatic analysis built into the program showed the layer to be weak since the probe slid easily through the drilled hole. Thus the automatic print-outs needed to be corrected by hand. Appendix D summarises the results of the DCP analysis and also records the Structural Number values (SNPs) computed from the DCP tests. The strength coefficients for the layers that were drilled were obtained from the unconfined compressive strengths measured from the samples taken from the delivery trucks during construction (Table 2.8 above) using the standard relationship,

$$a = (750 + 386*S - 8.83*S^2)*10^{-4}$$

where

a = the strength coefficient
S = unconfined compressive strength in MN/m^2 measured after 7-days curing using the standard method.

The DCP was able to penetrate the sub-base of Section 4 (the weakest of the stabilized sub-bases) and so the relationship between DCP penetration rate and CBR was used together with the relationship between CBR and strength coefficient a_3 in the normal way.

$$a_3 = 0.01 + 0.065(Log_{10}CBR)$$ for sub-base material

$$a_2 = [29.14(CBR) - 0.1977(CBR)^2 + 0.00045(CBR)^3]*10^{-4}$$ for road base material

The SNP values allow direct comparison with the AASHTO method of design (AASHTO, 1993) as shown below.

4 ANALYSIS TO DATE

The trials are only two years old and 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 with models or empirical evidence from elsewhere to verify likely long-term performance.

The construction report, 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 4.1 shows how the insitu subgrade strength varies along the site and Figure 4.2 shows the statistical distribution of its value.

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.

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 below show that this was not very successful.

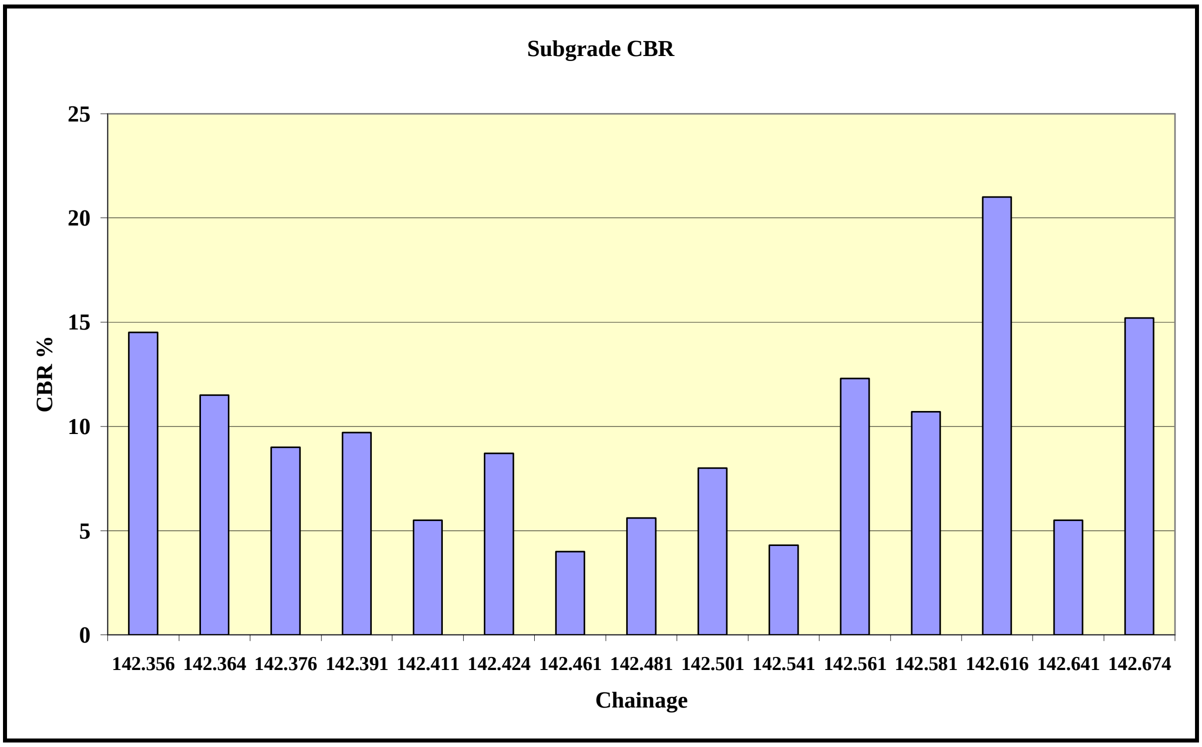

Figure 4.1 Subgrade strength distribution along the site

Figure 4.2 shows that the lower 10-percentile for design (ignoring at this stage the seasonal minima) is about 4.5%. The median value is about 9.0%. Such variability is perfectly normal and underlines the important of taking this into account properly in analysis. Figure 4.1 indicates that the subgrade at the ends of the site might be slightly stronger than in the middle, and therefore the variability could be reduced by subdividing the site, but there are not enough DCP tests at this stage for this to be reliable.

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 (HDM4, 2002) based on the paper by Rolt and Parkman (2000). 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 4.3.

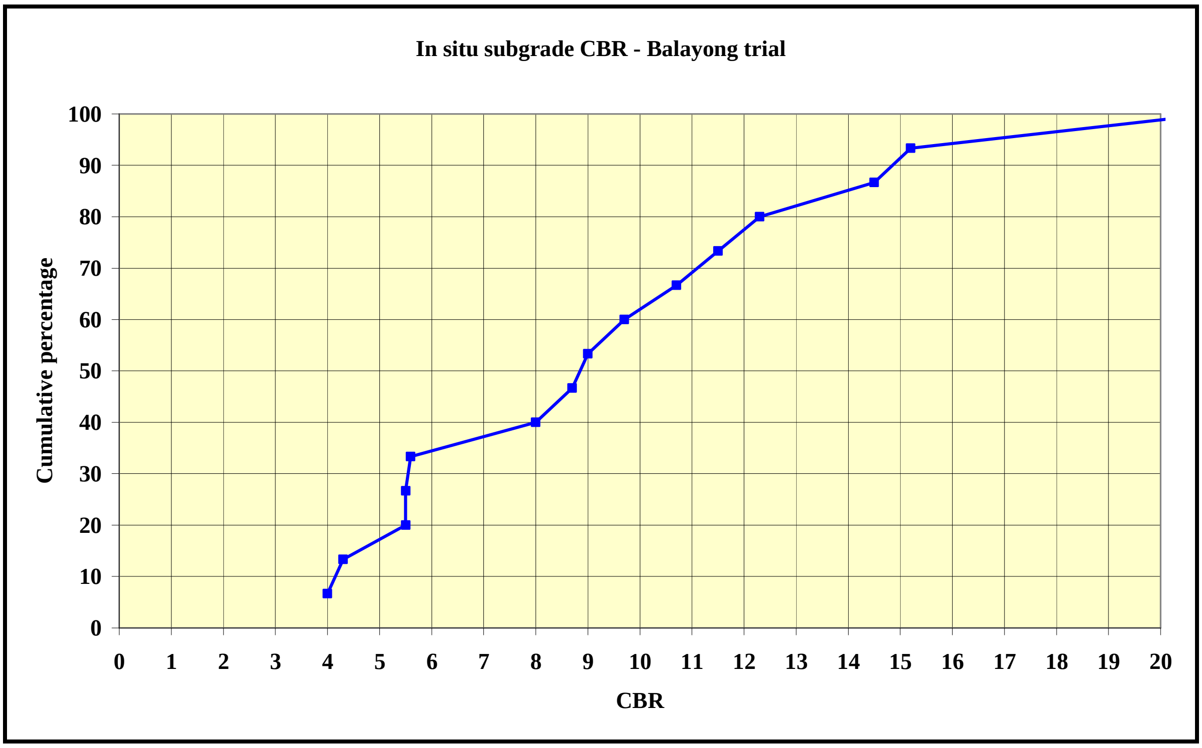

Figure 4.2 Distribution of subgrade CBR's for Balayong trial

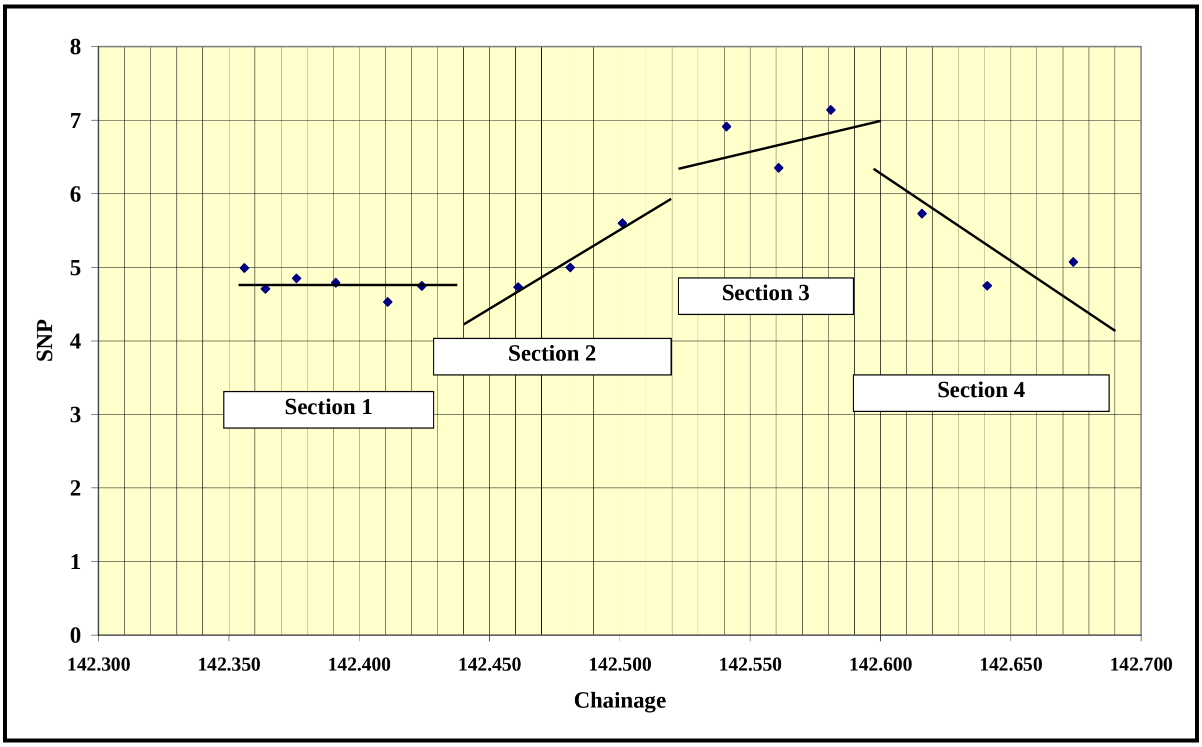

Figure 4.3 Structural number (SNP) along the site

The variability reflects two things. First it reflects the variability of the subgrade strength. Secondly, the process of drilling out the stabilized sub-base has caused errors because sometimes the drilled hole is deeper than the depth of the layer. Under these circumstances the sub-base appears to be thicker than it really is and the SNP is then correspondingly too high. Sometimes it appears that the lower part of the sub-base is weaker than expected but this is also probably a consequence of the drilling operation. In this case the drill has not penetrated right through the stabilized layer but debris from the drilling operation partially fills the hole and gives a false, low reading. Further analysis of the DCP data is required to identify these issues and to obtain a better estimate of SNP based on the known thickness of the sub-base at each point obtained from the levels.

This variability is particularly noticeable in Section 2. Here the SNP was expected to decrease as the chainage increases from 142.440 towards 142.520 i.e. as the sub-base thickness decreases, but it appears to increase.

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

Change in PSI	=	2.5	
Reliability	=	95%	
(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 lower 10-percentile SNP

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.] At this time there are only three DCP tests (and therefore three SNP values) for each section (except Section 1) and so it is not possible to calculate mean and 10-percentile values with a sufficiently high level of statistical accuracy. Also the test sections have been deliberately designed with varying sub-base thicknesses, and therefore SNP values, to help refine the design thicknesses that will result from this project, thus calculating the expected life of each section in this way is merely an average or a first approximation. However, using mean values as calculated from the three results and the *lowest* value instead of the lower 10-percentile value, we obtain Table 4.1 showing likely traffic carrying capacities.

Table 4.1 Projected traffic capacity based on AASHTO (10^6 esa)

Section	Mean value of SNP	Method (a) (10^6 esa)	10-percentile (lowest test result) of SNP	Method (b) (10^6 esa)
1	4.77	4.2	4.53	12
2	5.11	6.4	4.73	17
3	6.80	59	6.35	170
4	5.18	7.1	4.75	18

For Method (a) to agree with Method (b) the value of S_o would have to be between 0.1 and 0.15. This is very low indeed and is quite unrealistic. Method (b) is inherently the most accurate but requires more data for the 10-percentile to be estimated accurately.

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 so, by way of comparison, the projected traffic capacity has also been calculated using the original AASHTO approach. Thus Method (c) assumes,

(c) Standard Deviation term S_o = 0.5 when using mean values of subgrade CBR and SN,

and we obtain Table 4.2 showing the likely traffic carrying capacity.

Table 4.2 Projected traffic capacity based on AASHTO (10^6 esa)

Section	Mean value of CBR	Mean SN	Method (c) (10^6 esa)
1	9.8	4.35	33
2	5.9	4.48	12
3	9.1	6.28	300+
4	13.9	4.31	69

Method (a) differs greatly from method (c) simply because the contribution of the subgrade to SNP derived from the AASHTO equation differs from that assumed in the alternative SNP method based on other design charts and which also takes account of the decrease in contribution with depth.

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 4.4.

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. Based on just three DCP tests, the subgrade of Section 4 may be slightly stronger than average and therefore the deflections ought to be lower, relatively, than expected merely from the strength and thickness of the layers.

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.

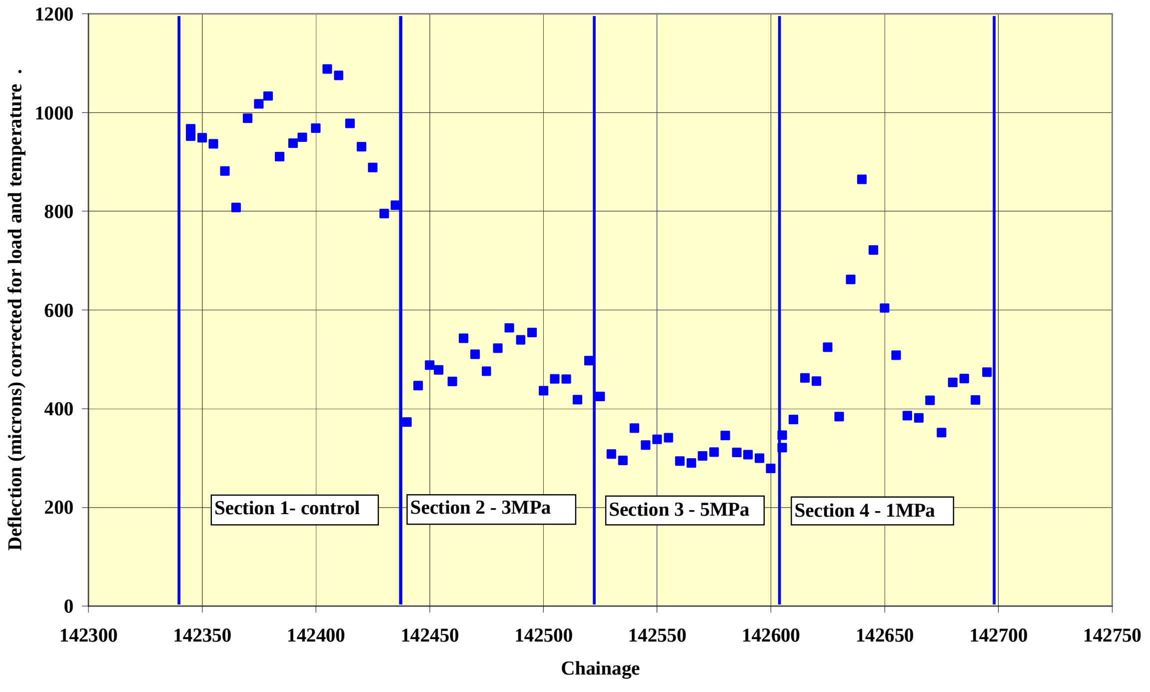

Figure 4.4 Central deflections along the site

Deflections on Section 1, the control section, are higher than expected. The relationship between SNP and deflection (Figure 4.5) 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 are correct (i.e. no anomalous behaviour - see Appendix F) and the layers are behaving elastically so that load spreading is good and therefore 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.

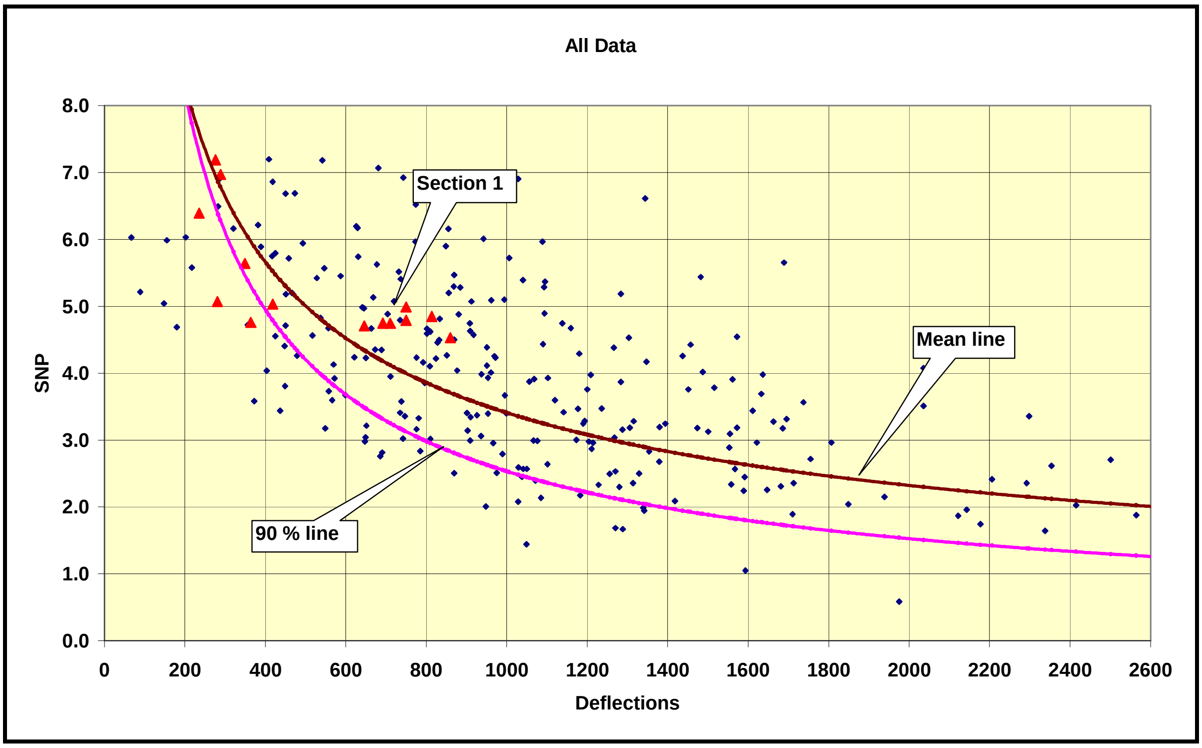

Figure 4.5 SNP from DCP tests versus deflection (the triangles are the Balayong site)

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,

$$FWD = (BB - 40)/1.09$$

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.

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

$$\text{Traffic capacity} = 2*(10^{8})*(D1^{-2.78})$$

Where D1 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, those shown in Table 4.3.

Table 4.3 Comparative traffic carrying capacities by two methods

Section	FWD Mean D1 (90%)	Estimated Traffic capacity ratios	
		FWD	SN approach method (b)
1	1075	1	1
2	554	5.8	1.4
3	341	20	14
4	721	3	1.5

This compares with the ratios obtained from the structural number analysis as shown in the last column of Table 4.3. The poor agreement is a consequence of the fact that the SNP/FWD data do not plot on a smooth curve (Figure 4.5). The deflection values for Section 1 indicate a relatively short life whereas the SNP values indicate a reasonable life and this clearly influences the ratios in the Table. Possible problems with the interpretation of DCP measurements (Appendix F) or with the calculation of SNP (described above) may be to blame but the possibility needs to be considered that Section 1 is showing anomalous results. Section 1 has been examined in more detail as described in Chapter 5.

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.

4.3.1 Back analysis

The program *Modulus* (Metric Modulus, 2002) was used to calculate the elastic modulus of each layer. Various methods were tried using 3-layer models (sub-base and road base combined) and 4-layer models. The most successful results were obtained when the modulus of the AC surfacing was fixed at a realistic value for the temperature conditions of the measurements and only E_2 and $E_{subgrade}$ were allowed to vary, where E_2 is the effective modulus of the combined sub-base and road base. By successful we mean that the value of E_2 along the site followed the expected trend (Figure 4.6) and the values appeared to be reasonable. Nevertheless, the values of $E_{subgrade}$ appeared to follow the trend of E_2 (Figure 4.7) and did not correlate at all well with the DCP data (Figure 4.8).

It does not appear that back-analysis is very helpful in interpreting the condition of the trial pavement and will not be discussed further in this report.

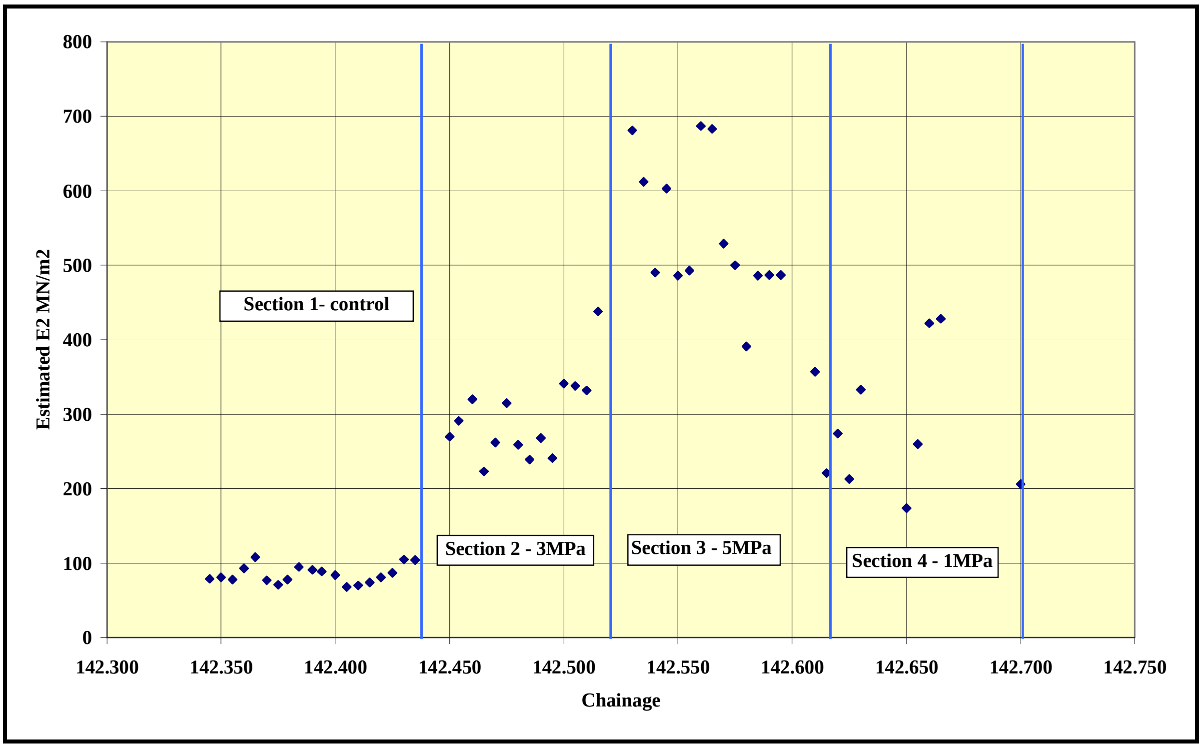

Figure 4.6 E_2 from back analysis of FWD with base and sub-base combined

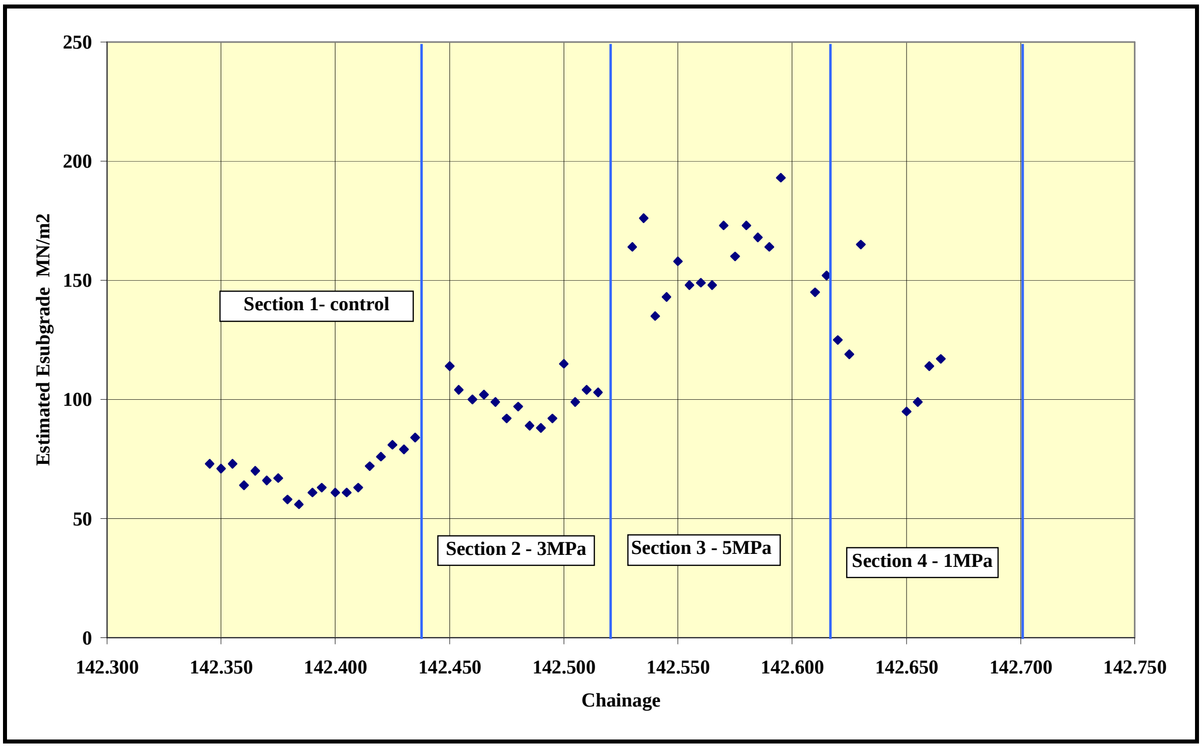

Figure 4.7 Subgrade modulus from back analysis of FWD with base and sub-base combined

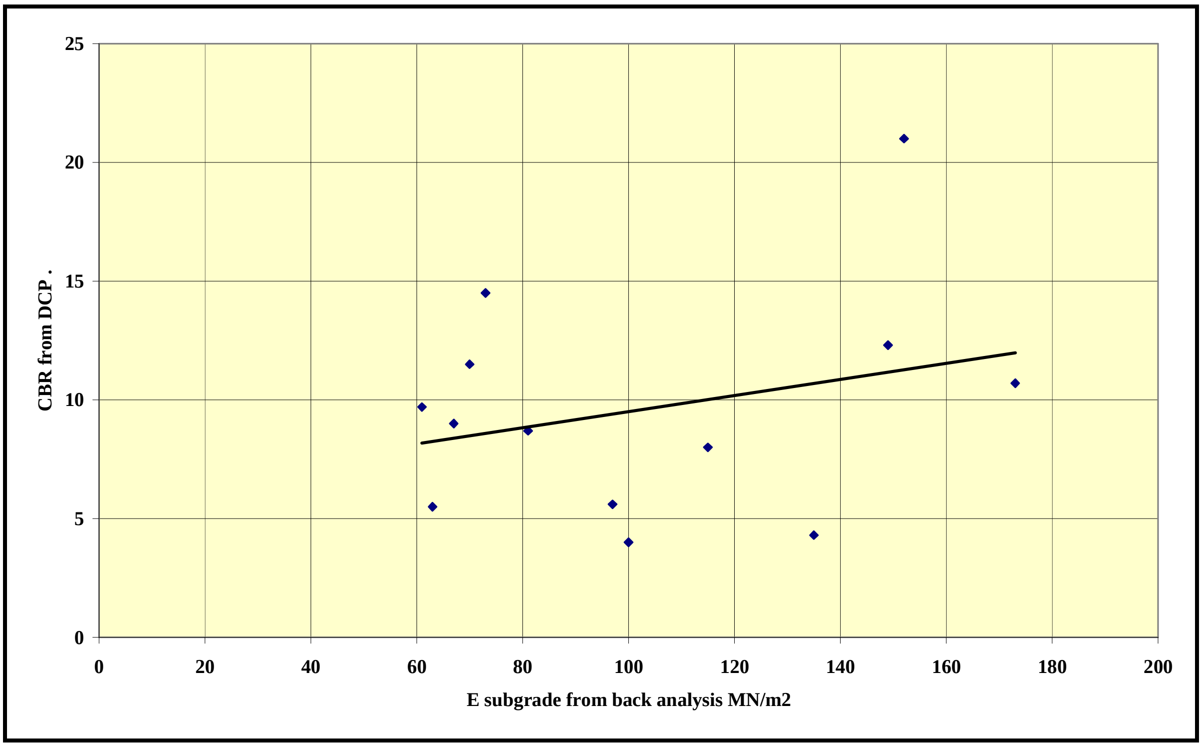

Figure 4.8 Subgrade modulus from FWD versus subgrade CBR from DCP

5 EARLY PERFORMANCE AND CONSTRUCTION PROBLEMS

Although most of the experimental site was constructed satisfactorily there was one area that has given problems and needs to be removed from the experiment. The problems are described in this Chapter.

High deflections and rutting in Section 1

Part of Section 1, between chainages 142+390 and 142+440, began to show signs of deterioration during the first year of trafficking. One important clue to the problem was that deflections tests made when only 50mm of AC had been laid were lower than when the full surface had been built. This suggested that there might be a lack of bond between the two layers and that the top layer was slipping on the lower one. Another indication of sliding was that the white painted edge line was displaced outwards, towards the shoulder, along the problematic length. Further investigation consisted of additional DCP tests in November 2002 and laboratory testing of the pavement and subgrade materials sampled from test pits excavated in the outer wheel-path in February 2003. The excavations were carried out using a large mechanical shovel. When the excavation began (by cutting through the surfacing) the top layer slid off the lower layer providing further evidence of a lack of bond between the two layers. This did not occur when the shovel was used in other areas of Section 1 where no deterioration had occurred.

The DCP tests indicated that the base was very strong in the deteriorated length but that the sub-base strength was marginal. Laboratory testing of each pavement layer showed that the road base easily exceeded the specifications, that the sub-base just met the specification on soaked CBR requirements, and that the sub-base failed to meet the specifications on a component of the grading requirements. These tests did not indicate any reasons for such early deterioration.

The most likely explanation for the deterioration is that the top layer of the surfacing was not bonded to the lower layer. This is because the deterioration was observed so soon after construction when the cumulative traffic loading would not have been a significant factor even though the sub-base material does not fully meet the specifications. The possible reasons for the ‘sub-standard’ sub-base are considered below:

1. It was noted in the construction report that severe rains during the construction of the sub-base led to the washing out of fines to this part of the section, which is on a lower grade than the remainder. The section was reprocessed but it is possible that an excess of fine, plastic material remained and is the cause of the marginal strength.

2. The specification requires the material to possess some plasticity. This requirement is usually to ensure that the material is bound, to a limited degree, and is relatively impermeable. However plastic materials lose strength when they become wet. This usually occurs as the moisture condition in the pavement “moves” from its condition at construction to its equilibrium condition in the early years after construction.

3. In the Philippines potential sub-base materials are often naturally non-plastic (as in this case) and fines containing clay are added. This is a difficult process because the parent material is usually wet. Adding clay, which is also wet, can easily lead to “balling” of the clay where the clay is not evenly distributed in the blend. Materials testing may not readily identify this problem.

4. There is no requirement in the specification for identifying, and therefore limiting, the use of materials that have a swelling potential; neither for the clay that is to be added as a binder nor for the blended material. Swelling, as moisture content increases, will lead to a loss of density and consequently to a greater loss in strength than from the increase in moisture content alone. Non-plastic sub-base materials do not swell, and so are far less sensitive to loss in strength arising from moisture changes.

It is apparent that a number of factors have probably led to the sub-base material being of marginal quality in the vicinity of the deteriorated area. The problems that have occurred are, in part, indicative of a number of deficiencies in the standard specification for sub-bases. It will also be apparent that sub-base problems such as these strengthen the argument for introducing stabilized materials because such materials are not sensitive to these problems, nor are they sensitive to changes, usually increases, in moisture content after construction.

Deterioration has continued in the form of severe rutting and cracking, and this part of Section 1 is clearly not appropriate as an experimental control, and has been eliminated from the experiment.

6 CONCLUSIONS

The analysis to date indicates that the use of stabilized sub-bases is likely to extend the service life of heavily trafficked roads. In particular, the use of a very strong stabilized sub-base (Section 3) indicates that the additional life may be between 14 and 20 times that of a road constructed using a conventional unbound sub-base (see Table 4.3). 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.

7 ACKNOWLEDGEMENTS

This Report was jointly produced by the Infrastructure Division of TRL Limited (Director Mr M. Head) and by The Bureau of Research and Standards, (Director Mr A. V. Molano, JR), Department of Public Works and Highways, Philippines on behalf of the Department for International Development, UK and on behalf of the Department of Public Works and Highways, GoP. The research was carried out in the Research and Development Division of BRS and their valuable co-operation has been essential to the success of this project.

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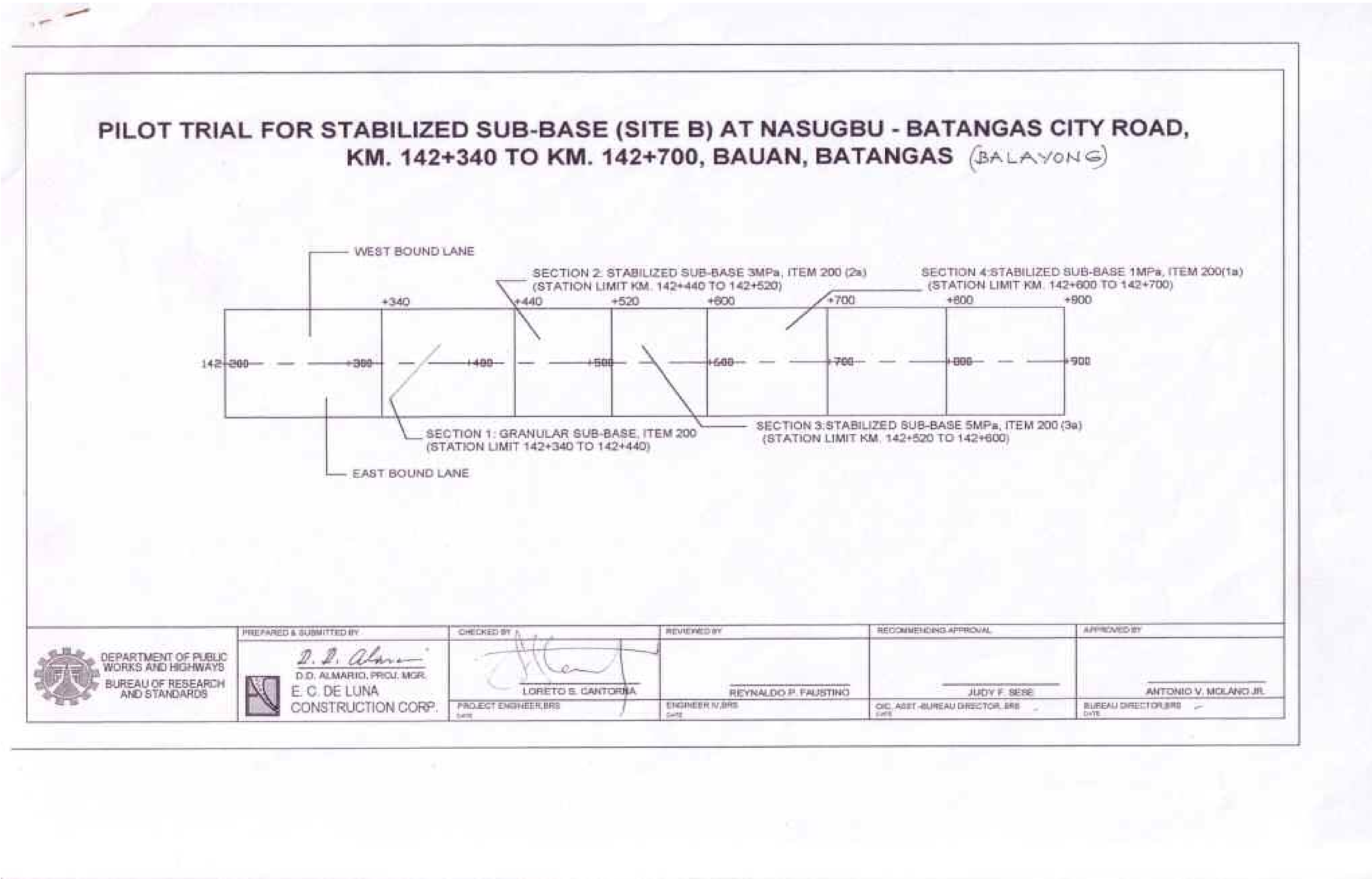
APPENDIX A

SITE LAYOUT



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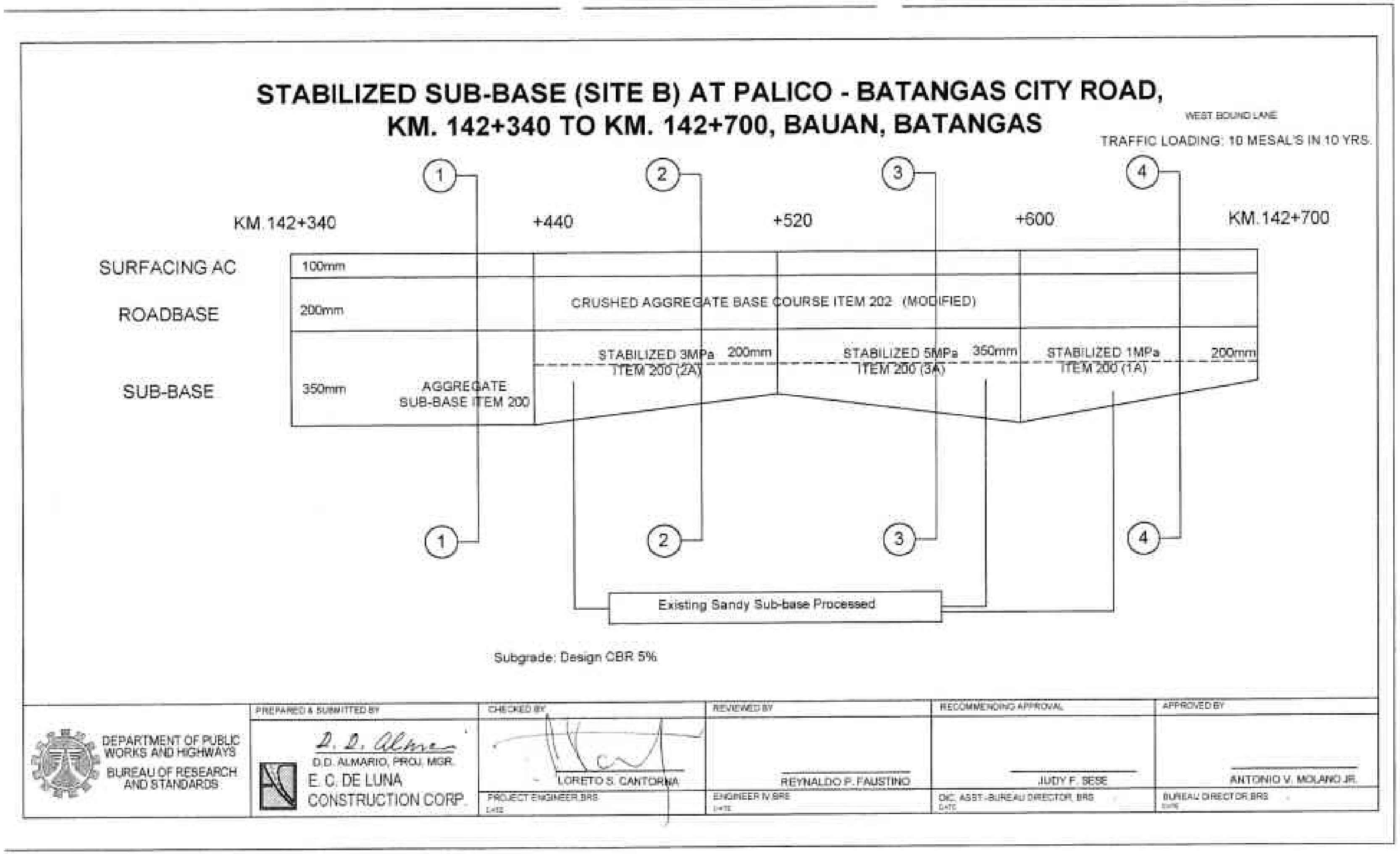
SITE LAYOUT



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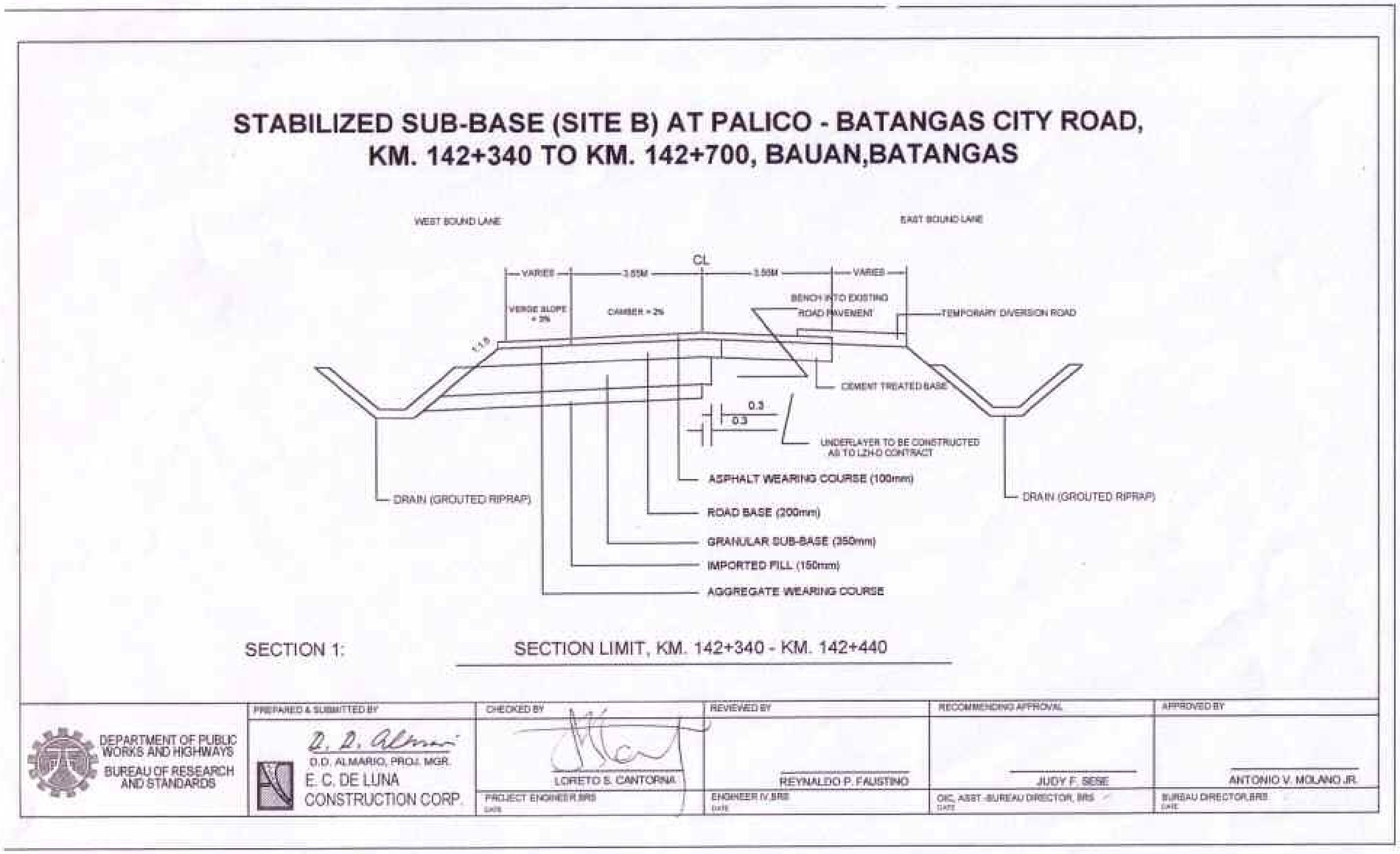
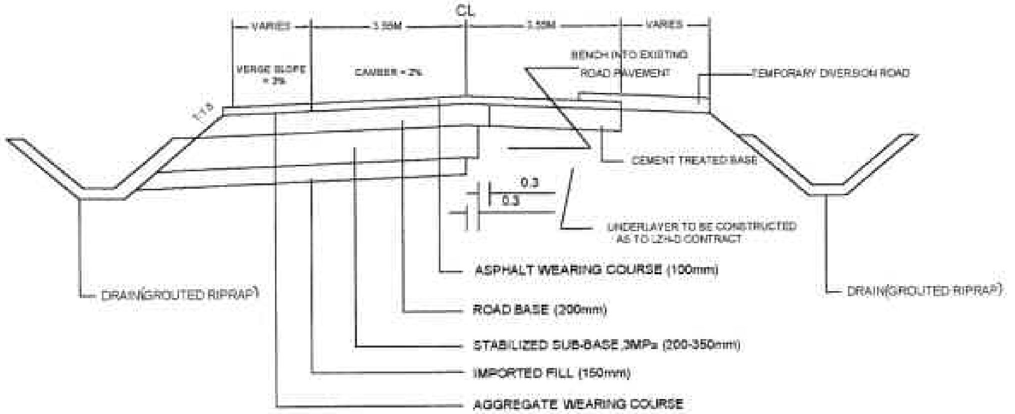
SITE LAYOUT



APPENDIX A

SITE LAYOUT

STABILIZED SUB-BASE (SITE B) AT PALICO - BATANGAS CITY ROAD, KM. 142+340 TO KM. 142+700, BAUAN,BATANGAS



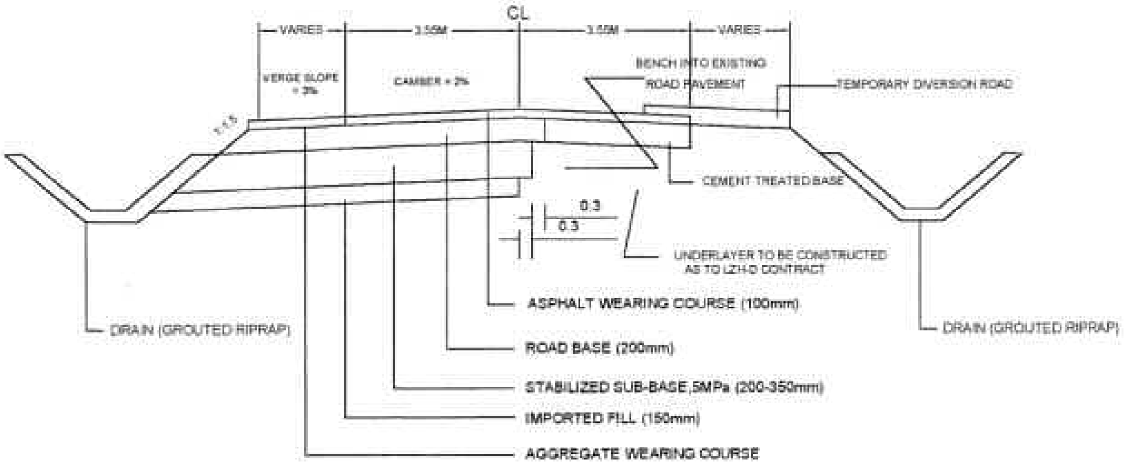
SECTION 2: SECTION LIMIT, KM. 142+440 - KM. 142+520

	PREPARED & SUBMITTED BY	CHECKED BY	REVIEWED BY	RECOMMENDING APPROVAL	APPROVED BY
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS BUREAU OF RESEARCH AND STANDARDS	D. D. Almario D.D. ALMARIO, PROJ. MGR. E. C. DE LUNA CONSTRUCTION CORP.	LORETO S. CANTORNA PROJECT ENGINEER,BRS DATE	REYNALDO P. FAUSTINO ENGINEER IV,BRS DATE	JUDY F. SESE OIC, ASST.-BUREAU DIRECTOR, BRS DATE	ANTONIO V. MOLANO JR. BUREAU DIRECTOR,BRS DATE

APPENDIX A

SITE LAYOUT

STABILIZED SUB-BASE (SITE B) AT PALICO - BATANGAS CITY ROAD,
KM. 142+340 TO KM. 142+700, BAUAN,BATANGAS



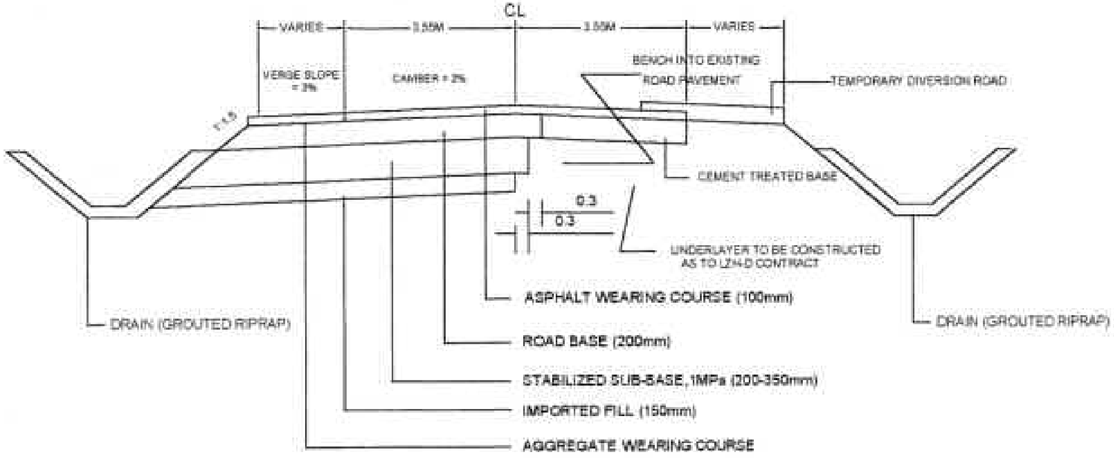
SECTION 3: SECTION LIMIT, KM. 142+520 - KM. 142+600

	PREPARED & SUBMITTED BY	CHECKED BY	REVIEWED BY	RECOMMENDING APPROVAL	APPROVED BY
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS BUREAU OF RESEARCH AND STANDARDS	D. D. Almario D.D. ALMARIO, PROJ. MGR. E. C. DE LUNA CONSTRUCTION CORP.	LORETO S. CANTORNA PROJECT ENGINEER,BRS DATE	REYNALDO P. FAUSTINO ENGINEER IV,BRS DATE	JUDY F. SESE OIC, ASST. BUREAU DIRECTOR, BRS DATE	ANTONIO V. MOLANO JR. BUREAU DIRECTOR,BRS DATE

APPENDIX A

SITE LAYOUT

STABILIZED SUB-BASE (SITE B) AT PALICO - BATANGAS CITY ROAD, KM. 142+340 TO KM. 142+700, BAUAN,BATANGAS



SECTION 4: SECTION LIMIT, KM. 142+600 - KM. 142+700

	PREPARED & SUBMITTED BY	CHECKED BY	REVIEWED BY	RECOMMENDING APPROVAL	APPROVED BY
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS BUREAU OF RESEARCH AND STANDARDS	D. D. Almario D.D. ALMARIO, PROJ. MGR. E. C. DE LUNA CONSTRUCTION CORP.	LORETO S. CANTORNA PROJECT ENGINEER, BRS DATE	REYNALDO P. FAUSTINO ENGINEER IV, BRS DATE	JUDY F. SESE OIC, ASST.-BUREAU DIRECTOR, BRS DATE	ANTONIO V. MOLANO JR. BUREAU DIRECTOR, BRS DATE

APPENDIX B

FWD DEFLECTIONS

Lane	Chainage	Decimal	Deflections Standardised by Load and Temperature									Plate conditions		Temperature		
			Radial Offset (mm)									Pressure	Load	deg C		
		Time	0	210	330	510	810	1270	1500	1800	2100	kPa	kN	Air	Surf	40mm
L	142340	11.22	479	462	418	323	198	109	84	74	61	700	50	38.1	33.5	30
L	142345	11.08	967	802	621	433	247	130	97	85	70	700	50	37.2	34.1	30
L	142345	11.18	952	795	616	431	246	127	97	84	68	700	50	37.8	34.1	30
L	142350	11.07	949	770	617	446	258	130	97	84	67	700	50	36.4	34.3	30
L	142355	11.05	937	779	615	434	247	125	94	81	66	700	50	36	36.1	30
L	142360	11.02	882	738	618	456	279	145	109	92	75	700	50	35	36.1	30
L	142365	10.98	807	690	559	411	246	134	103	88	71	700	50	35	35.9	30
L	142370	10.97	989	792	626	457	271	144	110	92	73	700	50	34.4	36.3	30
L	142375	10.95	1017	820	651	467	269	141	106	90	72	700	50	34.3	36	30
L	142379	10.92	1033	857	676	499	302	169	132	113	89	700	50	34.6	35.3	30
L	142384	10.90	910	826	693	513	303	169	131	113	91	700	50	34.5	34.9	30
L	142390	10.90	937	790	635	467	278	159	126	110	89	700	50	35.1	34.9	30
L	142394	10.87	950	828	641	455	267	154	123	107	87	700	50	35.9	34.4	30
L	142400	10.85	968	838	677	485	284	156	122	105	84	700	50	36.7	34.4	30
L	142405	10.82	1088	916	714	500	283	155	123	103	81	700	50	36.6	34.2	30
L	142410	10.80	1075	867	673	473	276	154	120	100	78	700	50	38.6	35.3	30
L	142415	10.78	978	833	632	424	244	129	104	87	66	700	50	39.5	36.7	30
L	142420	10.77	930	763	590	409	232	125	94	77	58	700	50	39.5	36.2	30
L	142425	10.73	888	724	556	380	212	117	90	74	56	700	50	41.5	35.8	30
L	142430	10.70	795	700	551	386	219	116	89	73	54	700	50	39.8	35.5	30
L	142435	10.68	812	664	524	372	214	110	82	66	48	700	50	38.4	34.3	30
L	142440	10.67	373	318	279	231	161	92	69	55	41	700	50	37.7	34.3	30
L	142445	10.65	447	342	267	204	147	91	69	57	42	700	50	37.8	35	30
L	142450	10.63	488	368	279	209	147	92	70	60	45	700	50	37.6	35.4	30
L	142454	10.62	478	380	296	225	155	99	77	65	50	700	50	37.1	35.1	30
L	142460	10.60	455	378	307	237	161	98	77	65	49	700	50	37	35.6	30
L	142465	10.57	542	421	330	252	169	99	74	62	47	700	50	38.4	36.7	30
L	142470	10.55	510	399	319	247	170	101	78	65	51	700	50	38.6	36.2	30
L	142475	10.53	476	408	339	267	182	106	80	67	51	700	50	37	35.4	30
L	142480	10.52	523	434	343	263	176	102	77	64	49	700	50	36.1	34.4	30
L	142485	10.50	563	461	366	280	189	113	86	71	54	700	50	35.3	34.5	30
L	142490	10.48	539	443	358	274	185	113	89	75	58	700	50	35	34.7	30
L	142495	10.45	554	462	366	273	178	108	86	74	59	700	50	36.4	35.6	30
L	142500	10.43	436	347	278	210	141	90	70	60	48	700	50	38.7	38.2	30
L	142505	10.42	460	398	323	248	165	101	80	69	56	700	50	39.7	37.9	30
L	142510	10.40	460	386	315	240	157	95	75	65	51	700	50	40.4	37.5	30
L	142515	10.38	418	372	307	238	160	98	78	68	55	700	50	41.1	36.6	30
L	142520	10.37	497	422	342	254	162	100	82	74	59	700	50	41.4	37.3	30
L	142525	10.35	425	337	274	211	145	94	76	65	53	700	50	42.2	37.4	30
L	142530	10.32	308	232	176	133	101	72	58	53	41	700	50	45.3	38.7	30
L	142535	10.32	295	244	195	148	102	55	40	32	25	700	50	45.2	38.1	30
L	142540	10.17	361	315	243	178	120	77	60	50	38	700	50	38	36.3	30
L	142545	10.13	326	266	213	164	118	74	58	47	35	700	50	38.7	36.4	30
L	142550	10.10	338	271	210	154	103	63	50	42	32	700	50	39.3	37.2	30
L	142555	10.08	341	279	221	165	112	68	52	44	34	700	50	38.5	36.8	30
L	142560	10.07	294	249	205	161	112	68	53	44	33	700	50	38.1	36.6	30
L	142565	10.05	290	251	209	165	114	67	51	43	31	700	50	37.9	37.1	30
L	142570	10.03	304	251	201	149	95	54	42	35	26	700	50	38.5	37.7	30
L	142575	10.02	312	280	225	166	103	57	43	36	27	700	50	38.4	38	30
L	142580	9.98	345	286	226	164	101	54	38	30	20	700	50	37.8	37.6	30
L	142585	9.97	311	261	210	157	101	55	41	34	25	700	50	38.6	37.8	30
L	142590	9.95	307	264	217	165	105	55	40	34	25	700	50	39.3	38.3	30
L	142595	9.93	299	240	189	139	86	49	37	32	24	700	50	39.7	39.5	30
L	142600	9.92	279	205	136	95	67	41	32	28	21	700	50	39.8	40.7	30
L	142605	16.82	346	284	220	159	102	59	44	38	28	700	50	30.4	36.3	30
L	142605	9.88	321	280	221	162	105	61	47	40	30	700	50	38.2	41	30
L	142610	16.78	378	311	237	172	113	69	53	46	34	700	50	30.6	36	30
L	142615	16.77	462	343	260	182	113	67	51	43	32	700	50	30.5	35.6	30
L	142620	16.68	456	367	284	208	136	79	60	52	40	700	50	31.1	34.3	30
L	142625	16.65	524	410	319	231	145	82	62	51	39	700	50	31.4	34.4	30
L	142630	16.62	384	315	235	161	100	61	46	40	29	700	50	31.5	34.8	30
L	142635	16.58	662	535	430	324	215	133	107	91	71	700	50	31.7	35.1	30
L	142640	16.55	865	724	570	414	260	150	118	101	83	700	50	31.9	35.4	30
L	142645	16.52	721	604	477	351	223	131	104	89	73	700	50	32.1	35.2	30
L	142650	16.47	604	514	406	287	175	101	79	67	52	700	50	32.5	35.1	30
L	142655	16.45	508	422	326	243	168	103	79	64	47	700	50	32.8	36.1	30
L	142660	15.92	385	342	280	215	146	87	65	53	38	700	50	34.9	34.6	30
L	142665	15.90	381	337	277	216	148	83	62	51	36	700	50	35	34.6	30
L	142670	15.57	417	362	300	232	155	87	66	53	38	700	50	33.4	34.7	30
L	142675	15.53	351	309	253	194	127	73	55	46	35	700	50	34.3	35.1	30
L	142680	15.52	453	385	319	245	161	87	63	50	35	700	50	33.8	35.3	30
L	142685	15.48	461	389	318	241	154	84	63	51	38	700	50	34.7	37.4	30
L	142690	15.38	418	336	264	195	127	71	52	43	31	700	50	33	36	30
L	142695	14.57	474	343	230	111	76	55	44	38	29	700	50	33.3	32.7	30

APPENDIX C

DCP ANALYSIS

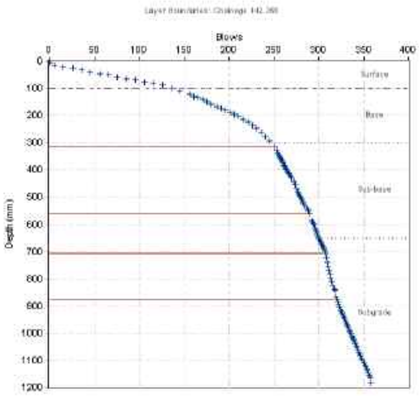
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

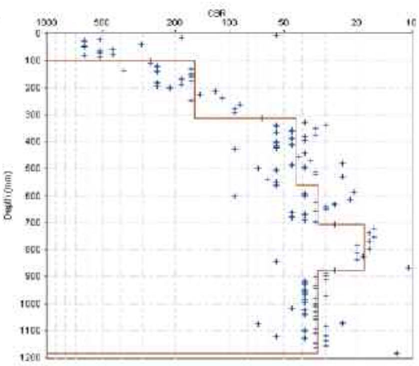
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.356	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.88	155	214	314	Base	0.14
2	6.31	43	246	560	Sub-Base	0.11
3	8.22	33	148	708	Sub-Base	0.10
4	14.08	18	169	877	Subgrade	0.00
5	8.11	33	308	1185	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.21	1.21	1.21
Sub-Base	1.63	1.63	1.55
Subgrade	--	1.65	1.15
Pavement Strength	4.41	6.06	5.48

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

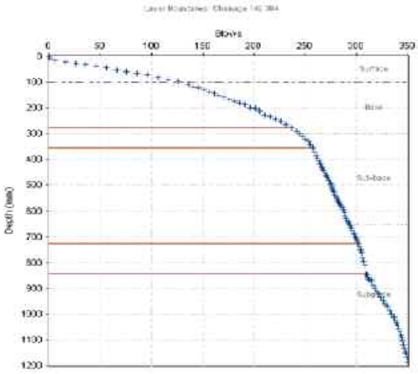
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

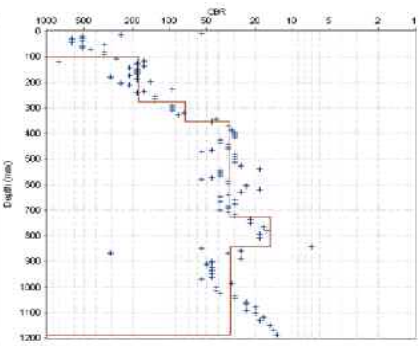
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.364	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.63	181	176	276	Base	0.14
2	3.76	74	79	355	Sub-Base	0.11
3	8.24	32	371	726	Sub-Base	0.10
4	16.86	15	118	844	Subgrade	0.00
5	8.37	32	343	1187	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.00	1.00	1.00
Sub-Base	1.84	1.84	1.68
Subgrade	--	1.53	0.96
Pavement Strength	4.41	5.94	5.21

CBR Relationship:

TRL equation: $\log_{10}(\text{CBR}) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

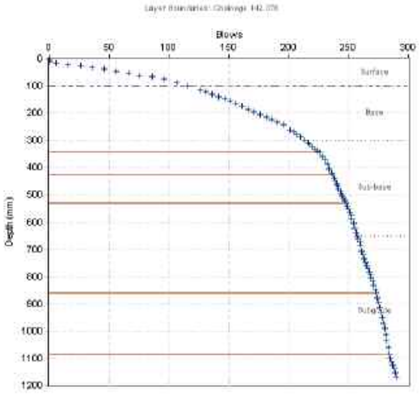
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

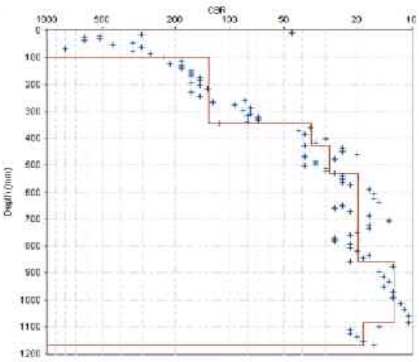
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.376	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	03/03/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	2.22	130	244	344	Base	0.14
2	7.55	36	83	427	Sub-Base	0.10
3	9.36	28	103	530	Sub-Base	0.10
4	13.16	20	329	859	Sub-Base	0.09
5	20.45	12	225	1084	Subgrade	0.00
6	14.00	19	84	1168	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.38	1.38	1.38
Sub-Base	1.89	1.89	1.66
Subgrade	--	1.39	0.76
Pavement Strength	4.84	6.23	5.37

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

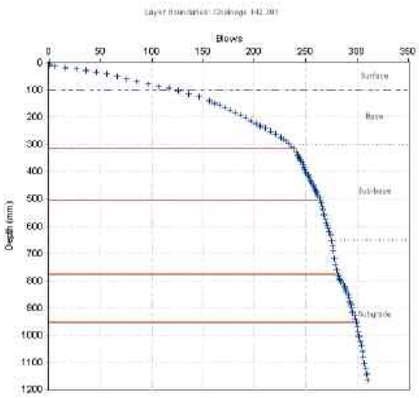
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

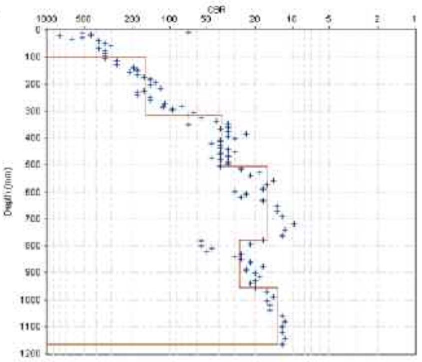
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.391	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.85	157	216	316	Base	0.14
2	7.23	37	188	504	Sub-Base	0.10
3	16.06	16	273	777	Sub-Base	0.08
4	9.83	27	177	954	Subgrade	0.00
5	19.27	13	212	1166	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.23	1.23	1.23
Sub-Base	1.66	1.66	1.52
Subgrade	--	1.44	0.88
Pavement Strength	4.46	5.90	5.20

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

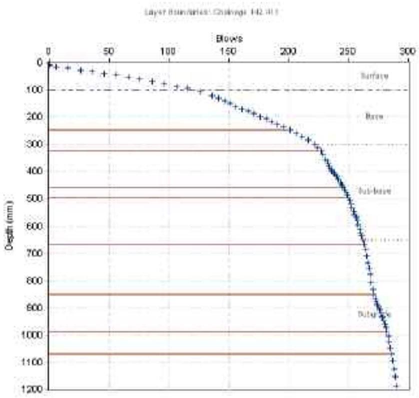
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

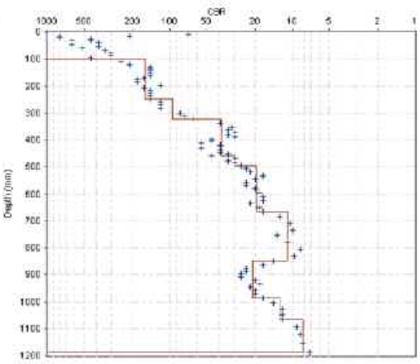
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.411	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.82	160	147	247	Base	0.14
2	2.96	96	77	324	Base	0.14
3	7.11	38	135	459	Sub-Base	0.10
4	9.00	30	36	495	Sub-Base	0.10
5	13.15	20	171	666	Sub-Base	0.09
6	23.00	11	184	850	Subgrade	0.00
7	12.45	21	137	987	Subgrade	0.00
8	20.25	13	81	1068	Subgrade	0.00
9	30.00	8	120	1188	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.25	1.25	1.25
Sub-Base	1.30	1.30	1.28
Subgrade	--	1.08	0.83
Pavement Strength	4.12	5.20	4.93

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

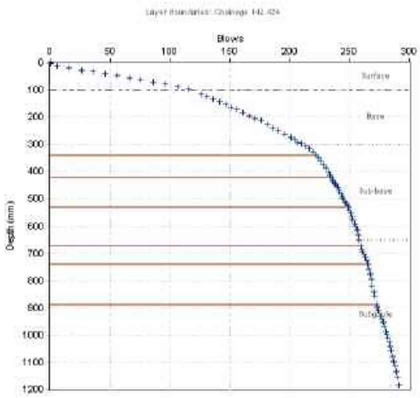
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

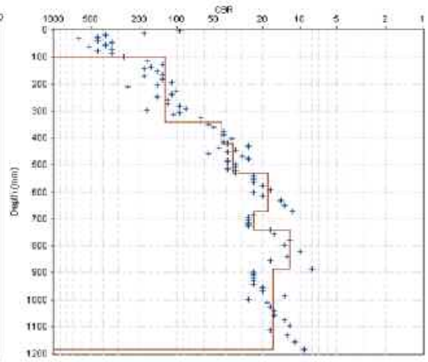
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.424	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	2.29	126	240	340	Base	0.14
2	6.31	43	82	422	Sub-Base	0.11
3	7.71	35	108	530	Sub-Base	0.10
4	14.40	18	144	674	Sub-Base	0.09
5	11.00	24	66	740	Subgrade	0.00
6	21.00	12	147	887	Subgrade	0.00
7	15.58	17	296	1183	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.35	1.35	1.35
Sub-Base	1.27	1.27	1.26
Subgrade	--	1.37	1.07
Pavement Strength	4.19	5.56	5.25

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

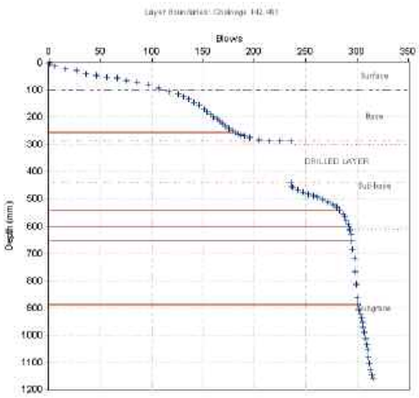
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

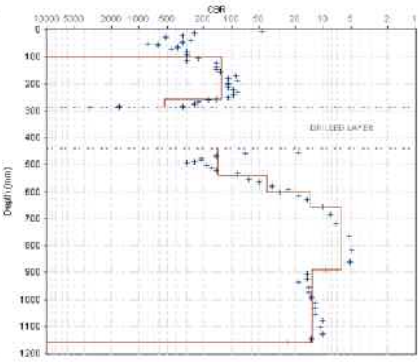
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.461	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	26/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	2.27	127	156	256	Base	0.14
2	0.59	525	32	288	Base	0.14
3	0.00	0	153	441	Sub-Base	0.12
4	2.08	139	100	541	Sub-Base	0.12
5	6.56	41	59	600	Sub-Base	0.11
6	18.67	14	56	656	Subgrade	0.00
7	38.83	6	233	889	Subgrade	0.00
8	19.29	13	270	1159	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.06	1.06	1.06
Sub-Base	1.42	1.42	1.42
Subgrade	--	0.83	0.68
Pavement Strength	4.05	4.88	4.73

CBR Relationship:
TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

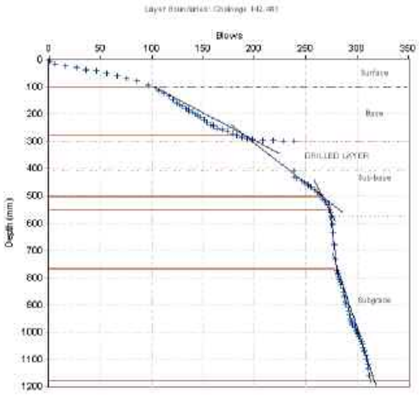
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

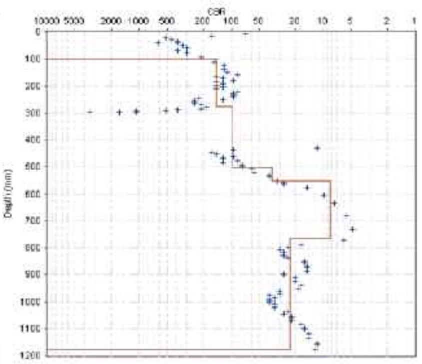
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.481	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.98	146	177	277	Base	0.14
2	2.96	96	226	503	Sub-Base	0.12
3	7.44	36	50	553	Sub-Base	0.10
4	29.74	8	214	767	Subgrade	0.00
5	11.37	23	412	1179	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.00	1.00	1.00
Sub-Base	1.23	1.23	1.25
Subgrade	--	1.08	0.94
Pavement Strength	3.80	4.88	4.76

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

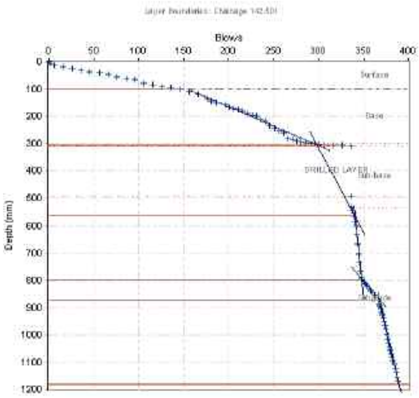
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

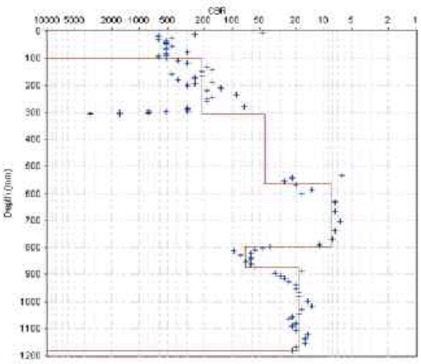
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.501	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.42	209	206	306	Base	0.14
2	6.16	44	258	564	Sub-Base	0.11
3	30.36	8	233	797	Subgrade	0.00
4	3.89	72	77	874	Subgrade	0.00
5	14.06	18	306	1180	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.17	1.17	1.17
Sub-Base	1.09	1.09	1.12
Subgrade	--	1.07	0.95
Pavement Strength	3.83	4.90	4.81

CBR Relationship:

TRL equation: log (CBR) = 2.48 - 1.057 x log (penetration rate)

APPENDIX C

DCP ANALYSIS

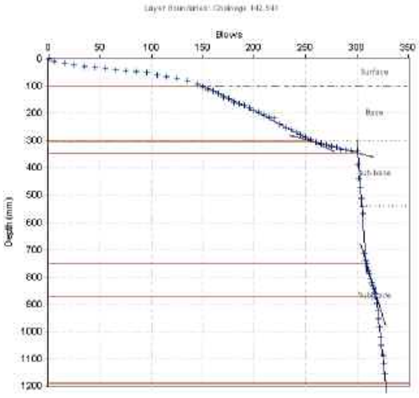
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

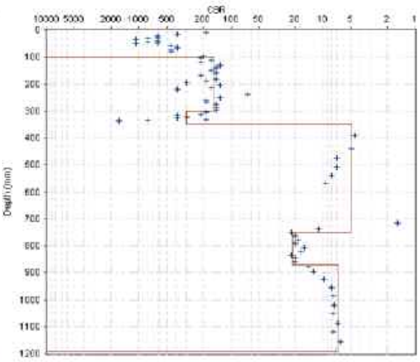
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.541	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.92	152	202	302	Base	0.14
2	1.00	301	44	346	Base	0.14
3	49.57	5	405	751	Sub-Base	0.03
4	12.27	21	121	872	Subgrade	0.00
5	36.32	7	319	1191	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.40	1.40	1.40
Sub-Base	0.49	0.49	0.46
Subgrade	--	0.90	0.61
Pavement Strength	3.46	4.36	4.04

CBR Relationship:
TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

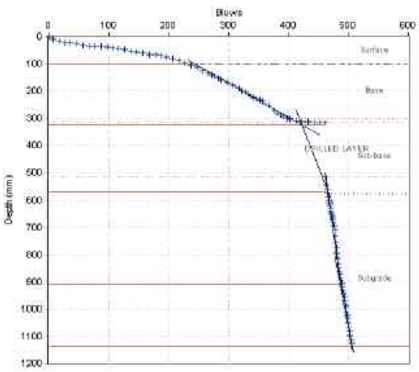
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

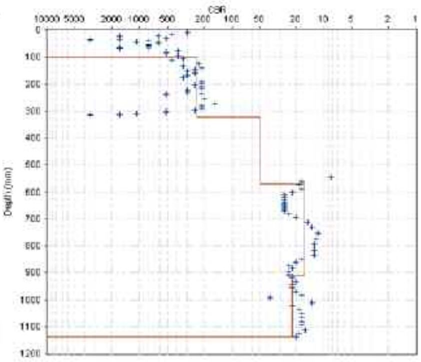
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.561	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.26	237	223	323	Base	0.14
2	5.62	49	248	571	Sub-Base	0.11
3	15.98	16	340	911	Subgrade	0.00
4	11.98	22	228	1139	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.27	1.27	1.27
Sub-Base	1.06	1.06	1.10
Subgrade	--	1.57	1.43
Pavement Strength	3.90	5.47	5.37

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

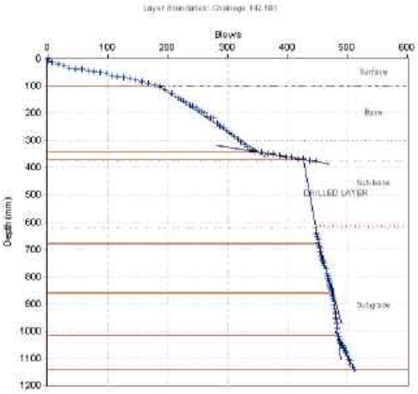
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

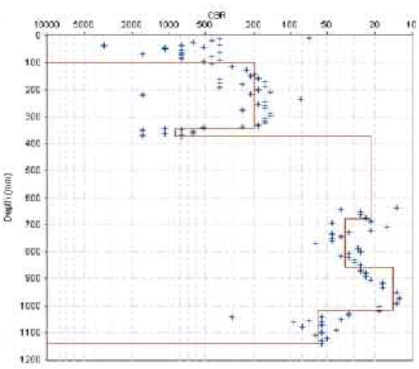
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.581	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.48	200	244	344	Base	0.14
2	0.36	892	27	371	Base	0.14
3	12.26	21	307	678	Sub-Base	0.09
4	7.63	35	182	860	Subgrade	0.00
5	17.94	14	157	1017	Subgrade	0.00
6	4.73	58	125	1142	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.54	1.54	1.54
Sub-Base	1.10	1.10	1.10
Subgrade	--	1.49	1.22
Pavement Strength	4.21	5.70	5.43

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

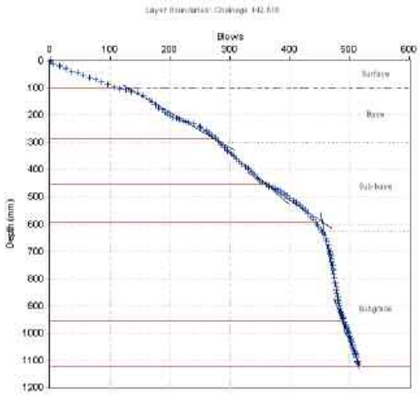
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

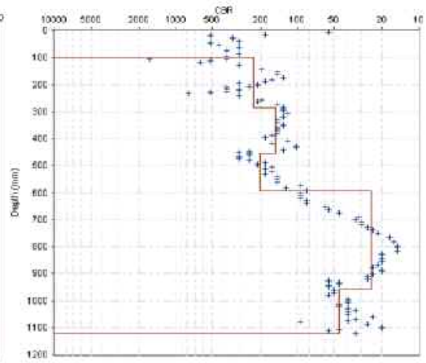
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.616	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.30	229	186	286	Base	0.14
2	1.93	151	170	456	Sub-Base	0.12
3	1.47	201	137	593	Sub-Base	0.12
4	10.55	25	365	958	Subgrade	0.00
5	6.06	45	163	1121	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.06	1.06	1.06
Sub-Base	1.39	1.39	1.39
Subgrade	--	1.82	1.49
Pavement Strength	4.02	5.84	5.51

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

Report produced by ....

Report Date: 05-Mar-2004

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APPENDIX C

DCP ANALYSIS

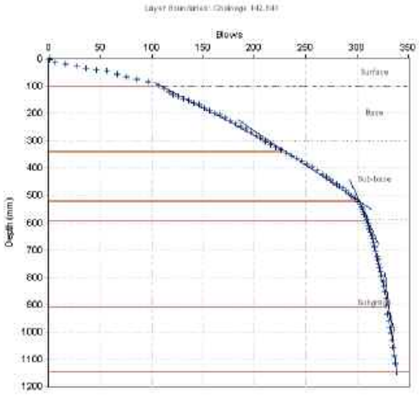
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

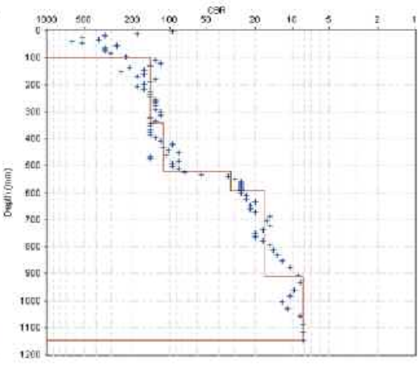
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.641	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	2.00	145	240	340	Base	0.14
2	2.52	114	181	521	Sub-Base	0.12
3	8.42	32	71	592	Sub-Base	0.10
4	15.53	17	319	911	Subgrade	0.00
5	30.24	8	236	1147	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.36	1.36	1.36
Sub-Base	1.11	1.11	1.14
Subgrade	--	1.07	0.97
Pavement Strength	4.04	5.11	5.04

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX C

DCP ANALYSIS

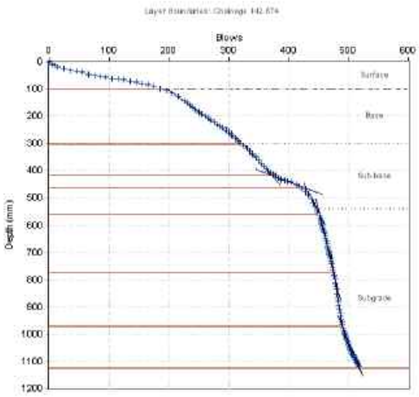
Note that the strength of any stabilized layer has not been assigned properly in these charts. The correct value of SNP is shown in Appendix D.

UK DCP V2.2

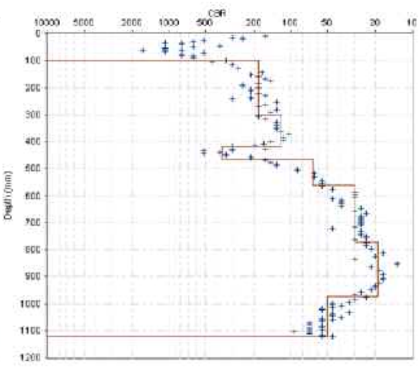
DCP Layer Strength Analysis Report

Project Name: Balayong 1

Chainage (km):	142.674	Surface Type:	Hot Mixed Asphalt
Direction:		Thickness (mm):	100
Location/Offset:	Carriageway/1.00m	Strength Coeff.:	0.40
Cone Angle:	60 degrees	Base Type:	
Zero Error (mm):	81	Thickness (mm):	
Test Date:	19/02/2004	Strength Coeff.:	



Layer Boundaries Chart



CBR Chart

Layer Properties

No.	Penetration Rate (mm/blow)	CBR (%)	Thickness (mm)	Depth (mm)	Position	Strength Coefficient
1	1.60	183	202	302	Base	0.14
2	2.38	121	116	418	Sub-Base	0.12
3	0.82	371	47	465	Sub-Base	0.12
4	4.25	65	96	561	Sub-Base	0.11
5	8.97	30	211	772	Subgrade	0.00
6	13.62	19	201	973	Subgrade	0.00
7	5.50	50	151	1124	Subgrade	0.00

Pavement Strength

	Layer Contribution		
Layer	**SN**	**SNC**	**SNP**
Surface	1.57	1.57	1.57
Base	1.15	1.15	1.15
Sub-Base	1.17	1.17	1.20
Subgrade	--	1.67	1.49
Pavement Strength	3.89	5.56	5.41

CBR Relationship:

TRL equation: $\log_{10}(CBR) = 2.48 - 1.057 \times \log_{10}(\text{penetration rate})$

APPENDIX D

STRUCTURAL NUMBERS

Section	Chainage	h1 mm	a1	h21 mm	CBR/UCS	a21	h22 mm	CBR/UCS	a22	h3 mm	layer 3 CBR	a3	h4 mm	layer 4 CBR	a4	h5 mm	layer 5 CBR	a5	h6 mm	layer 6 CBR	a6	Subgrade CBR	SN	SNG	ASN (SNP)
	142 +	SURFACING		ROADBASE 2			ROADBASE 3			SUB-BASE 1			SUB-BASE 2			SUB-BASE 3			SUB-BASE 4			%			
1	142.356	100	0.30	214	155	0.14				246	43	0.11	148	33	0.10							**14.5**	**4.03**	**1.50**	**4.99**
1	142.364	100	0.30	176	181	0.15				79	74	0.11	371	32	0.10							**11.5**	**4.03**	**1.34**	**4.71**
1	142.376	100	0.30	244	130	0.14				83	36	0.10	103	28	0.10	329	20	0.09				**9.0**	**4.45**	**1.15**	**4.85**
1	142.391	100	0.30	216	157	0.14				188	37	0.10	273	16	0.08	177	27	0.10				**9.7**	**4.74**	**1.21**	**4.79**
1	142.411	100	0.30	147	160	0.14	77	96	0.14	135	38	0.10	36	30	0.10	171	20	0.09	402	11.0	0.07	**5.5**	**4.82**	**0.70**	**4.53**
1	142.424	100	0.30	240	126	0.14				82	43	0.11	108	35	0.10	144	18	0.09	66	24.0	0.09	**8.7**	**4.05**	**1.12**	**4.75**
2	142.461	100	0.30	156	127	0.14				185	4.02	0.22	100	139	0.12	59	41	0.11	56	14.0	0.08	**4.0**	**4.51**	**0.38**	**4.73**
2	142.481	100	0.30	177	146	0.14				226	4.02	0.22	50	36	0.10							**5.6**	**4.31**	**0.72**	**5.00**
2	142.501	100	0.30	206	200	0.15				258	4.02	0.22										**8.0**	**4.61**	**1.05**	**5.60**
3	142.541	100	0.30	202	152	0.14	44	150	0.14	405	5.33	0.26	121	21	0.09							**4.3**	**7.09**	**0.45**	**6.91**
3	142.561	100	0.30	223	200	0.15				248	5.33	0.26										**12.3**	**5.02**	**1.39**	**6.35**
3	142.581	100	0.30	244	200	0.15				334	5.33	0.26	182	35	0.10							**10.7**	**6.75**	**1.28**	**7.14**
4	142.616	100	0.30	186	200	0.15	170	151	0.14	137	200	0.11										**21.0**	**3.87**	**1.72**	**5.73**
4	142.641	100	0.30	240	145	0.14				181	114	0.12	71	32	0.10	319	17	0.08				**5.5**	**4.71**	**0.70**	**4.75**
4	142.674	100	0.30	202	183	0.15				116	121	0.12	47	250	0.11	96	65	0.11	211	30.0	0.10	**15.2**	**4.34**	**1.53**	**5.07**

APPENDIX E

TEMPERATURE CORRECTIONS TO FWD DEFLECTIONS

For a variety of reasons it is convenient if all deflections are normalised to a standard temperature so that comparisons of different sections of road can be made more precisely. To develop a method for converting deflections measured at different road temperatures to deflections made at a standard temperature, deflections and temperatures were measured at four specific locations continuously throughout the day. Two temperatures were recorded namely the temperature at the surface of the road (Tsurf) and the temperature at a depth of 40 mm (T40). The results are shown in Figures E1 to E9 for each the central geophone.

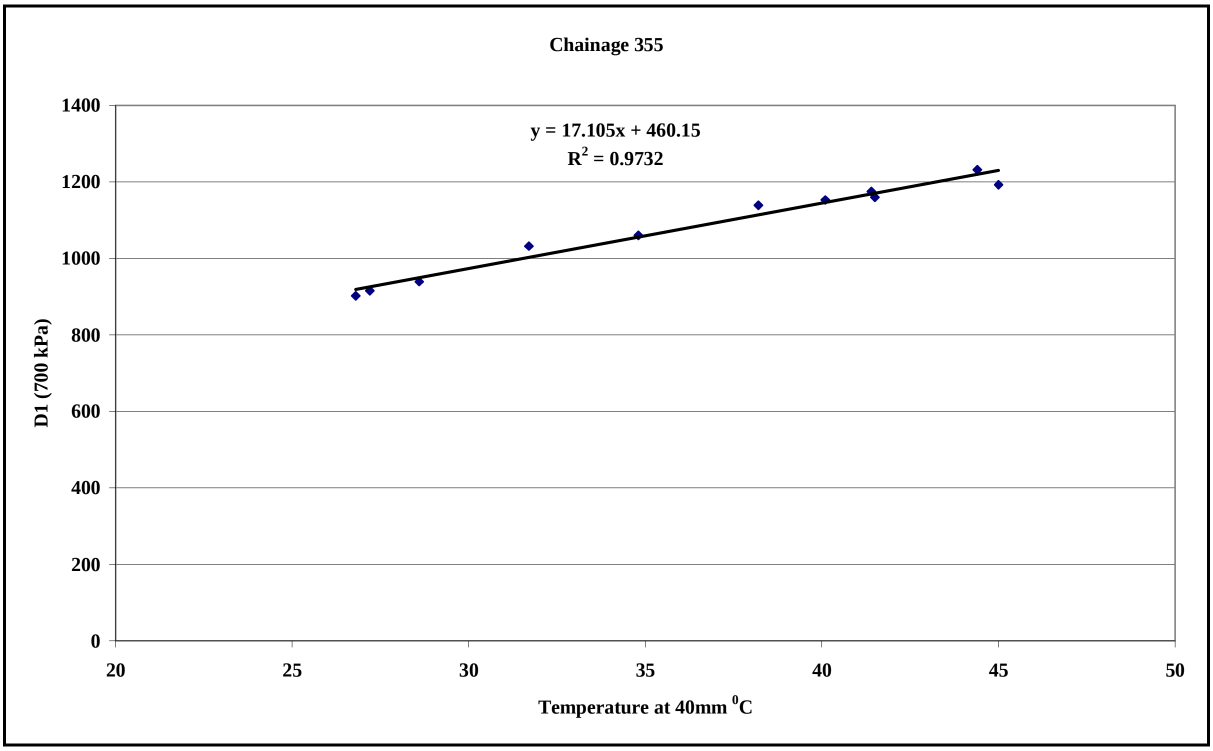


Figure E1 Central deflection at chainage 141+355

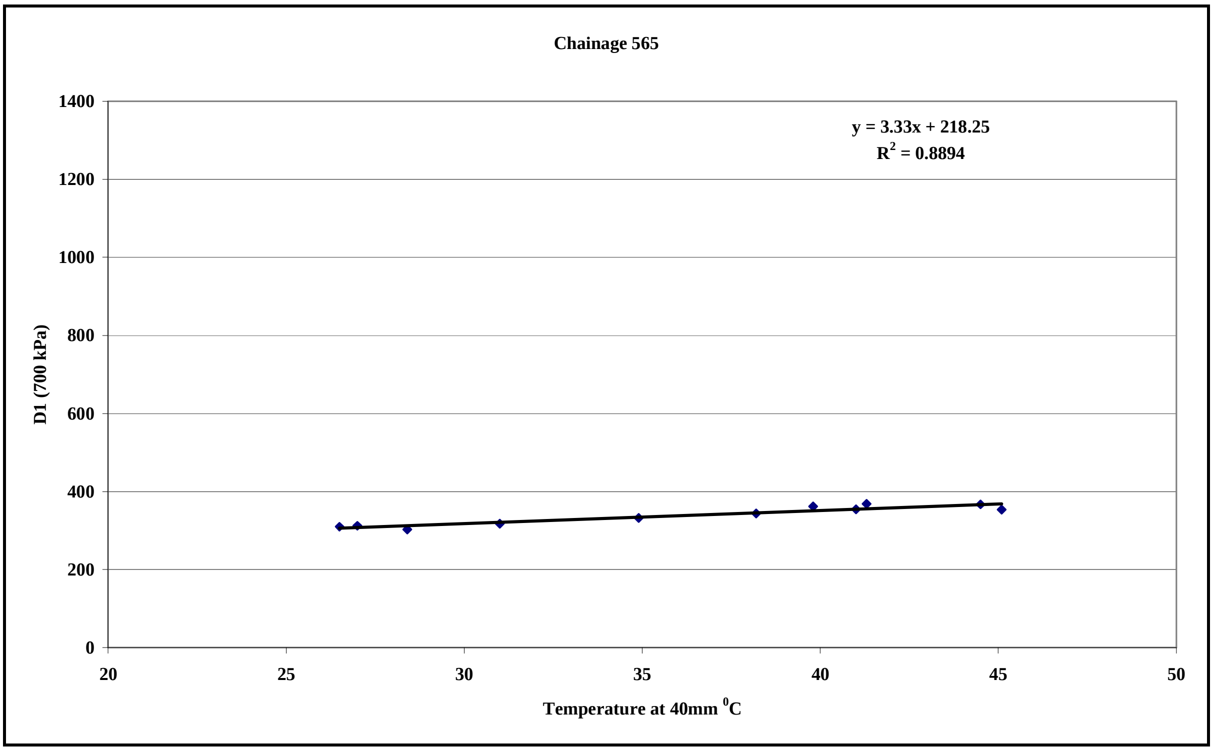


Figure E2 Central deflection at chainage 141+565

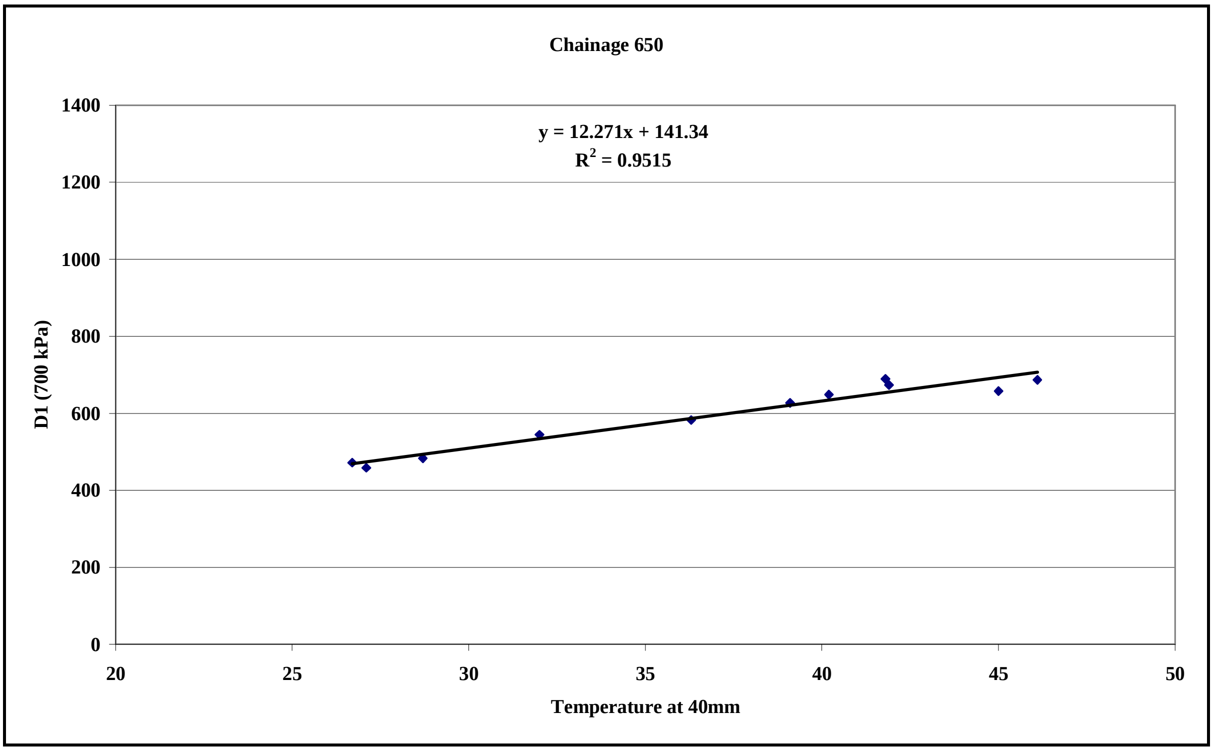


Figure E3 Central deflection at chainage 141+650

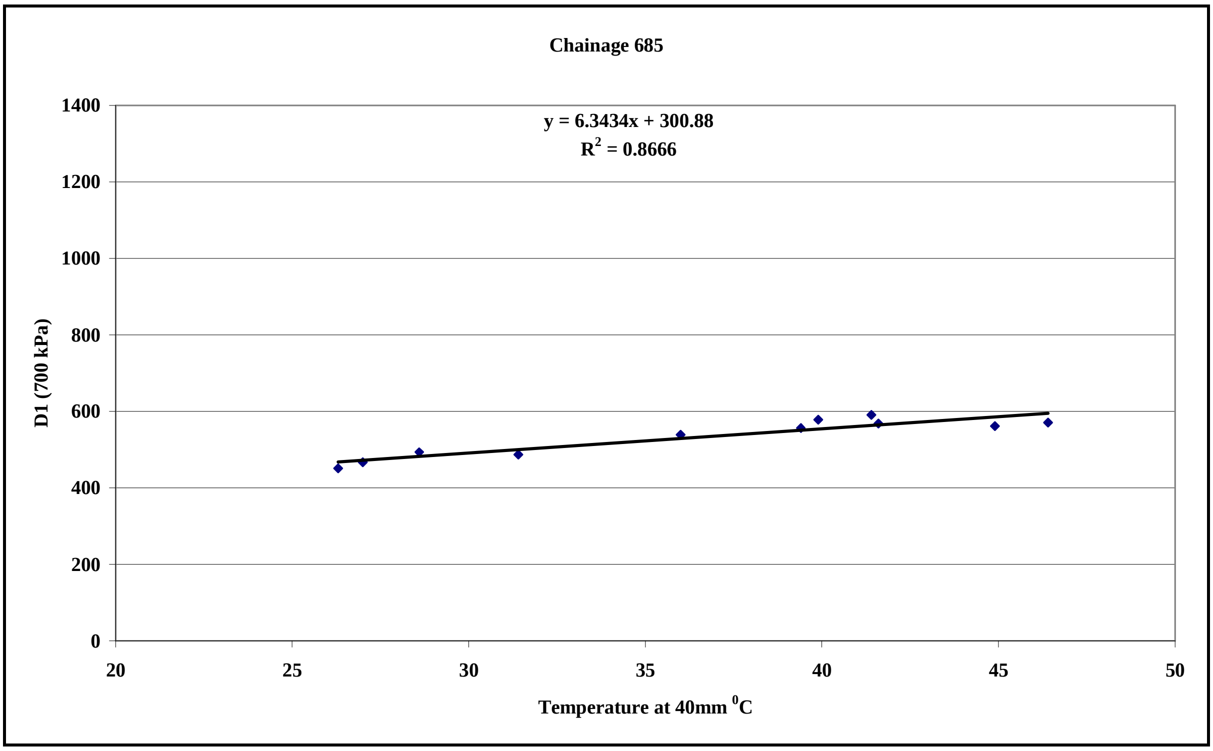


Figure E4 Central deflection at chainage 141+685

The results for chainages 141+650 and 141+685 were very similar and were combined.

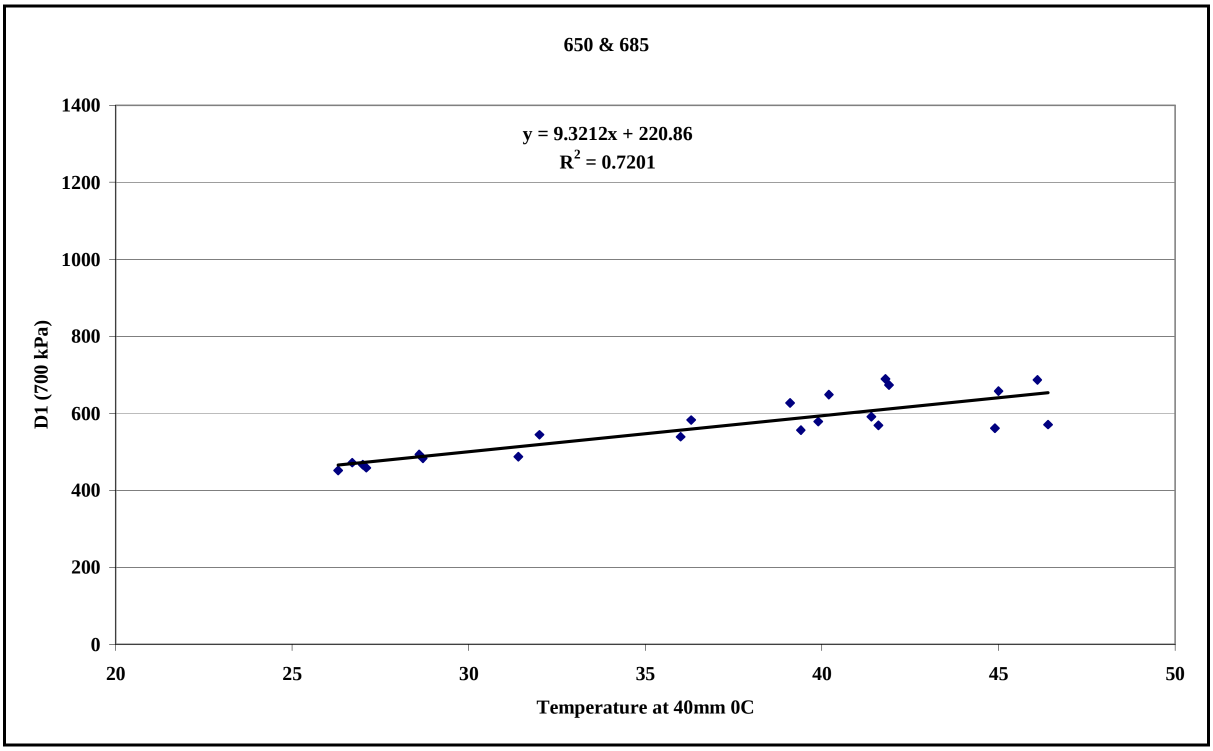


Figure E5 Central deflection at chainage 141+650 plus 141+ 685

Similar relationships were derived for each of the geophones. Table E1 summarises all the relationships.

Table E1 Deflection versus temperature

Geophone	Chainage	Equation
D1	141+355	D1 = 460.2 + 17.11*(T40)
	141+565	D1 = 218.3 + 3.33*(T40)
	141+650 and 685	D1 = 220.9 + 9.32*(T40)
D2	141+355	D2 = 449.1 + 11.32*(T40)
	141+565	D2 = 210.8 + 1.85*(T40)
	141+650 and 685	D2 = 253.4 + 5.35*(T40)
D3	141+355	D3 = 414.6 + 6.76*(T40)
	141+565	D3 = 178.7 + 1.45*(T40)
	141+650 and 685	D3 = 242.4 + 3.14*(T40)
D4	141+355	D4 = 359.0 + 2.54*(T40)
	141+565	D4 = 147.8 + 0.85*(T40)
	141+650 and 685	D4 = 219.2 + 1.02*(T40)
D5	141+355	D5 = 220.2 + 0.68*(T40)
	141+565	D5 = 91.6 + 0.83*(T40)
	141+650 and 685	D5 = 139.1 + 0.53*(T40)
D6	141+355	D6 = 102.7 + 0.56*(T40)
	141+565	D6 = 51.2 + 0.59*(T40)
	141+650 and 685	D6 = 75.6 + 0.42*(T40)
D7	141+355	D7 = 76.9 + 0.44*(T40)
	141+565	D7 = 42.5 + 0.33*(T40)
	141+650 and 685	D7 = 60.6 + 0.22*(T40)
D8	141+355	D8 = 61.4 + 0.52*(T40)
	141+565	D8 = 33.3 + 0.32*(T40)
	141+650 and 685	D8 = 47.7 + 0.24*(T40)
D9	141+355	D9 = 45.8 + 0.53*(T40)
	141+565	D9 = 23.8 + 0.24*(T40)
	141+650 and 685	D9 = 32.8 + 0.25*(T40)

These relationships were used to estimate deflections at T40 = 30, 35, 40 and 45 ^{0}C.

The important parameter is the gradient of the deflection/temperature relationship. Assuming a linear dependence between defection and temperature, all that is required to compute the deflection at a standard temperature is the deflection at the measured temperature (T) and knowledge of the gradient.

The gradient (G) in Table E1 depends upon deflection itself and the temperature at 40mm depth (T40) The relationships were found to be,

For D1 G1 = 4.7 – 0.1723*(T40) + 0.0174*(D1)

For D2 G2 = 1.45 – 0.0958*(T40) + 0.0155*(D2)

For D3 G3 = 0.42 – 0.0457*(T40) + 0.0121*(D3)

For D4 G4 = -0.09 – 0.0095*(T40) + 0.0065*(D4)

For the outer geophones the gradients D5 to D9 were very close to zero i.e. there was no temperature dependence.

In the 'results' tables all deflections have been 'normalized' to a T40 of 30 ^{0}C.

The FWD records the temperature at the road surface. T40 is measured at a depth of 40mm and requires a small hole to be made in the AC. T40 is much more representative of the temperature of the AC layer and therefore it would be very convenient if T40 could be predicted from Tsurf. The relationship between T40 and time of day is shown in Figure E6.

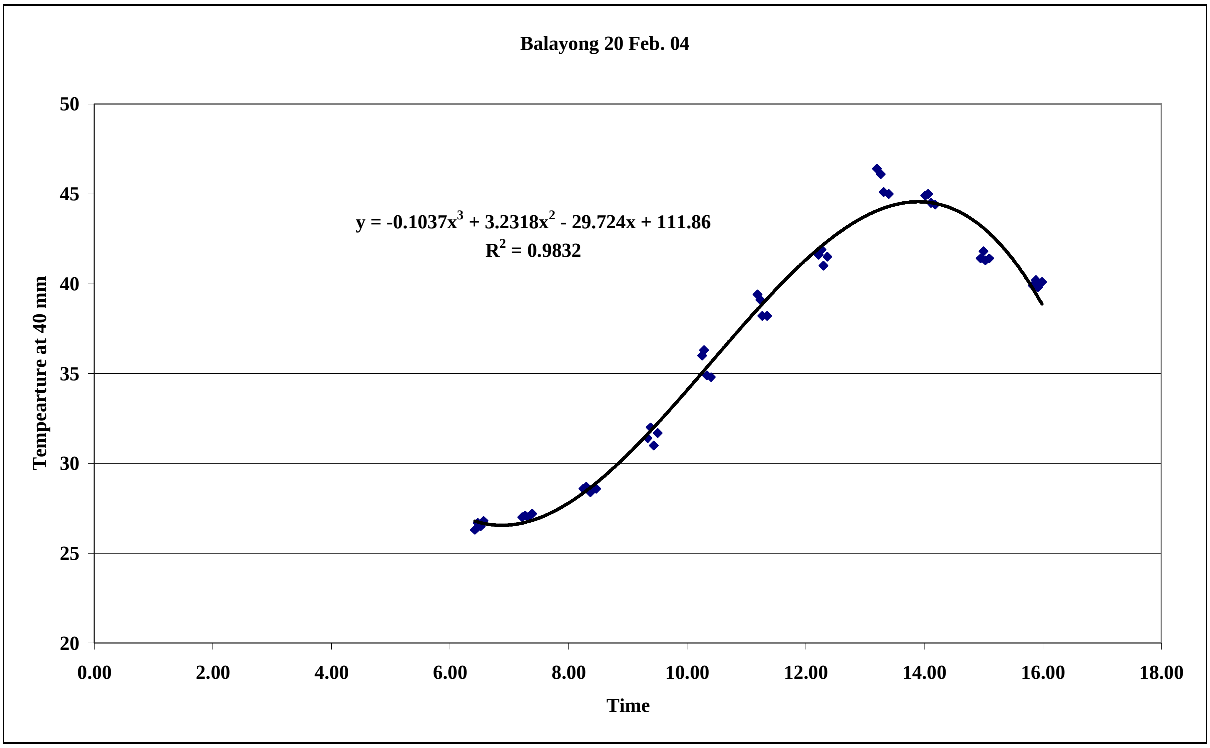


Figure E6 Temperature versus time of day

The highest temperature occurs at about 14.00hrs after which it is likely that temperature reversals will occur in the AC layer. Under these conditions T40, which will have lagged Tsurf during the morning will now be higher than Tsurf. The following relationship was developed to estimate T40 from Tsurf and the time of day, t in hours at which the temperature was measured.

$$T40 = 0.7345*(Tsurf) + 41.65*\sin[7.5*(t-2)] - 25$$

Where [7.5*(t-2)] is in degrees. Thus at 14.00 hours 7.5*(t-2) = 90. In Excel the sine function SIN assumes that the angle is specified in radians therefore the relationship in the data spreadsheet is

$$T40 = 0.7345*(Tsurf) + 41.65*\sin[\pi*(t-2)/24] - 25$$

Where $\pi*(t-2)/24$ is now in radians.

Figure E7 illustrates how the model fits the data.

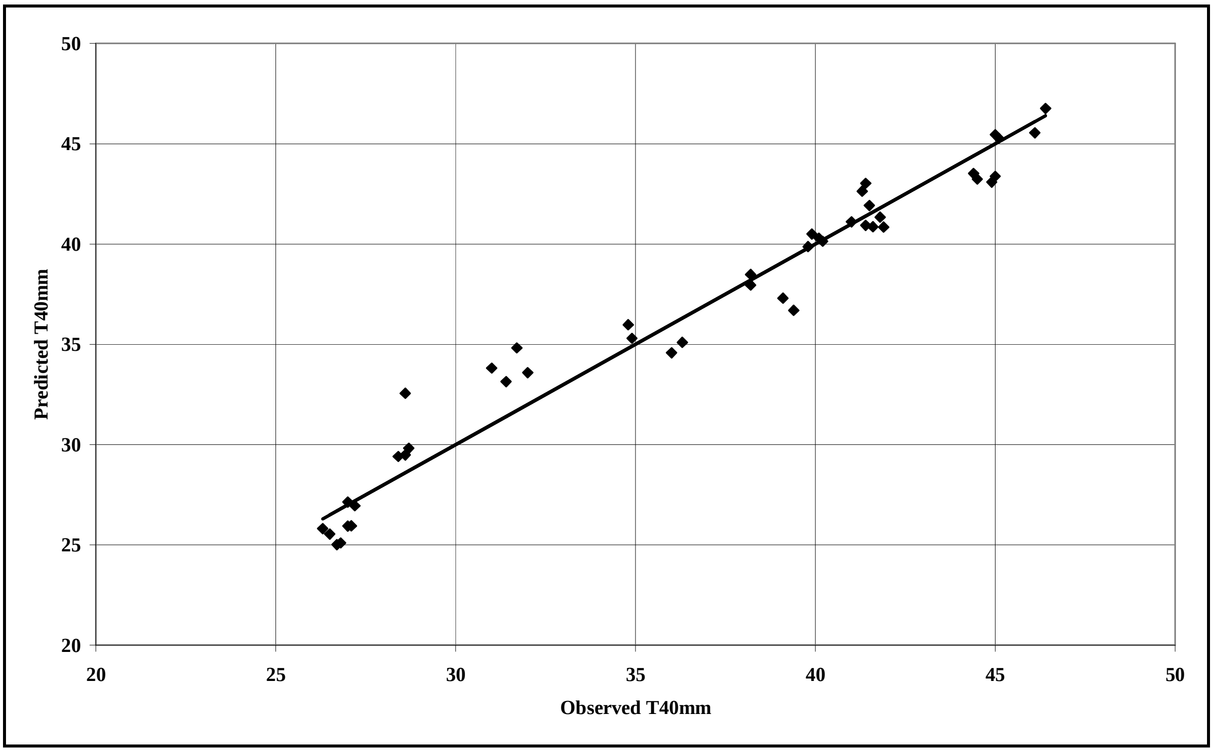


Figure E7 Observed versus predicted T40

APPENDIX F
INTERPRETING DCP TESTS

The DCP was originally developed for use in fine-grained soils but has been used extensively in recent years to measure properties of sands, gravels and other materials with maximum particle sizes up to or exceeding 37.5mm. When using the DCP for such tests the user should be aware of the likely problems that might arise in interpreting the results.

1 Relatively well-compacted pavements showing reasonable elastic behaviour but with large stones in the base or sub-base.

Such pavements may display normal deflection behaviour but with either high or low SNPs as measured by DCP. This arises when the cone of the DCP strikes a large stone in different ways or misses large stones altogether as shown in Figure F1. In test (a) the cone cannot penetrate at all and so the test is ignored. In test (b) the cone breaks the stone but penetration is uncharacteristically hard and hence the calculated SNC is high. Similarly SNC is high in situation (c) where the cone tries to push the stone aside. The result of test (c) is also probably high because of side friction that is likely to be generated on the DCP shaft. Conversely situation (d) provides a more normal result in many cases but could also provide a low SNP if there is insufficient granular material to fill the gaps between the larger stones properly. In this latter case the deflection values could be affected adversely (ie deflections high also). However, it must be remembered that a layer as thin as a single stone (albeit a large one) may be insufficiently thick to affect the deflections significantly whereas if penetration is difficult, the DCP reading will be affected much more.

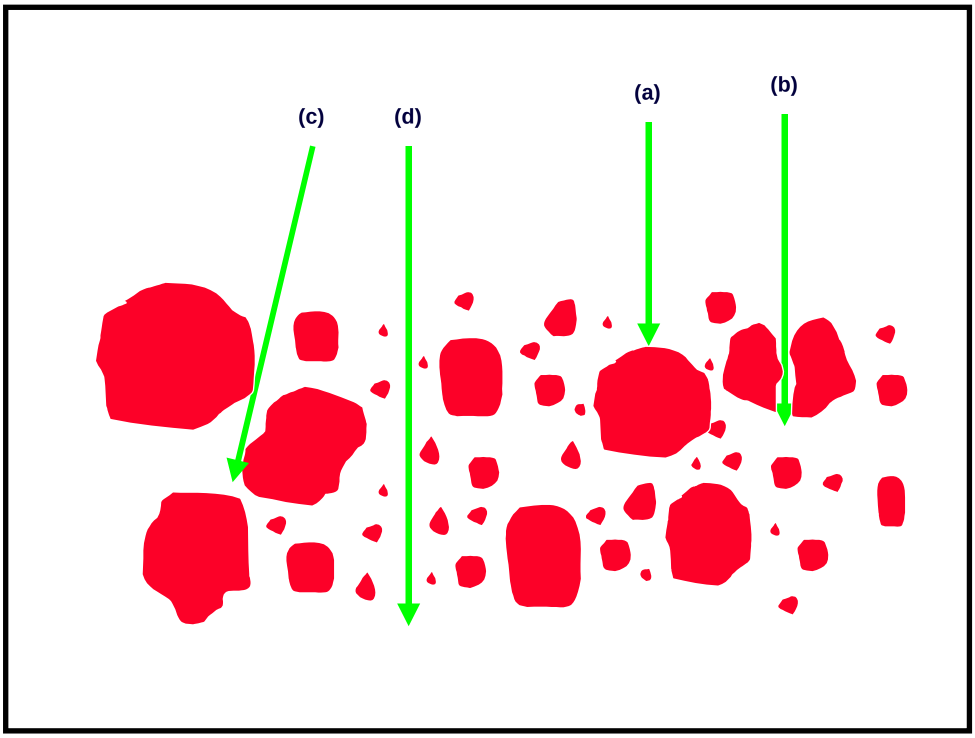


Figure F1. Typical DCP effects with large stones

2 Poorly compacted layers with large stones in the base or subbase

In this situation the DCP readings can suffer from the same problems as in example 1 above but the deflections are likely to be high immediately under the load and uncharacteristically low at high radial offsets. This is because the badly compacted layers are not behaving elastically to spread the load, with the result that their effective modulus – if such a property can be defined - is very low.

3 Badly cracked asphalt layers hidden beneath a relatively sound surfacing.

In this case the central deflections can be high because of lack of elastic layer behaviour but the SNPs can also be high (Figure F2). A convenient analogy is that of piles of bricks stacked side by side. A DCP test through one pile will give a very high value but the deflections will also be high because there is no interlock between the brick piles and so the effective elastic modulus is very low.

4 Strong pavement layers on a very weak underlying soil at depth.

This situation is slightly different in that both the SNP measured to a normal depth of 800-900mm and the deflections can appear to be satisfactory but a weakness at greater depth can result in gross subgrade movements (landslides). Traffic is not usually the primary cause of these failures. Their prevention is a matter of controlling the movement of water and providing suitable geotechnical solutions rather than through pavement thickness design.

Thus there are a variety of models that provide logical explanations of the observed behaviour and the scatter of the results apparent in the relationship between SNP and deflection shown in Figure 3.1 in the main text.

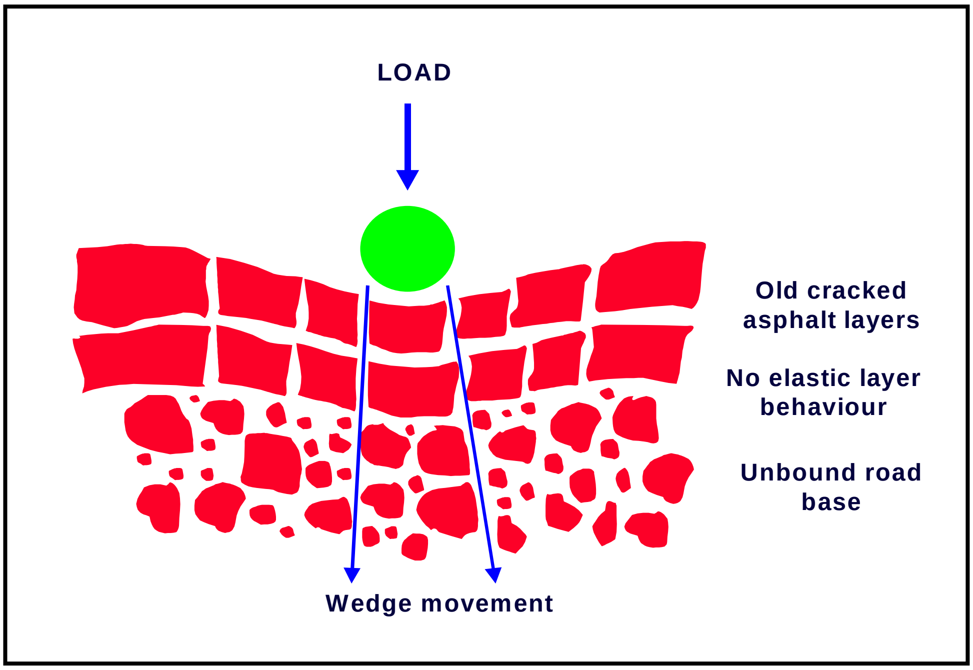


Figure F2 Layers of cracked asphalt

Thus it is concluded that a really good correlation between SNC and d_0 cannot be expected and that unless DCP and deflection tests either coincide or are separated by no more than one metre or so at most, the relationship will be even less robust.

8.1 Conclusions

In conclusion, neither the deflection values alone nor the DCP-measured structural numbers can give a true picture of the structural characteristics of the layers of a road as far as likely performance as a road pavements is concerned. High values of SNC derived from DCP data can give a misleading evaluation of apparent strength because high strength at a point does not necessarily translate into good load spreading characteristics (i.e. a ‘strong’ layer in the pavement sense). Also low values of deflection at high radial offsets from the point of load does not necessarily mean high subgrade strengths for the reasons discussed above.

In evaluating the properties of the layers in terms of their function as pavement layers i.e. their ability to spread the traffic loads, the central deflection is by far the most important. Information regarding the effective elastic modulus of each separate layer would be equally valuable but cannot be obtained from the data for the reasons outlined above. The DCP tests provide appropriate data much of the time but fail to do so sufficiently reliably at all times. The correlation between deflection and DCP data provides a means of separating 'good’ DCP data representing layers that are behaving in an adequate structural manner from the anomalies described above.