REPORT ON THE TRIALS AT KRIEL TO ASSESS THE EFFECTIVENESS OF IMPACT COMPACTION AND TO ESTABLISH APPROPRIATE METHODS OF INTEGRITY TESTING

1. INTRODUCTION

Africon was appointed by Landpac to assist them with compaction trials at a site on the outskirts of the town of Kriel. The compaction trials were done as part of Landpac's ongoing technology development program. The trials comprised the treatment of the in situ soil with various types of compaction equipment and varying the number of compaction passes within each treated section. *The broad objectives were to demonstrate the effectiveness of impact rolling in absolute terms, but also relative to conventional vibro compaction.*

In discussions, which preceded the trials, there was consensus that the tests performed on the trial sections had to aim at achieving to following:

- **Characterization** of the **site** to reflect the type of materials present and their consistencies.
- **Degree** and **depth** of improvement against various compaction efforts.
- The most appropriate **parameters** to be measured in respect of determining the degree and depth of improvement.
- The most appropriate **way of measuring** the required parameters to be in line with norms in the industry
- Attempt establishing **possible trends** between the results of the different test methods
- Comment on the **potential use** and **limitations** of the various test methods

Consequently a range of tests were selected to satisfy the above requirements. This involved the following:

- Establishing the characteristics of the soils across the site in terms of material type by employing indicator tests and moisture content tests to evaluate the field compaction results against these important parameters. To allow comparison of results between various test sections, it was necessary to establish the degree of uniformity of soil conditions across the site. Maximum dry density, optimum moisture content and CBR tests were conducted in the laboratory to assess compaction characteristics.

- Determining the stiffness of the in situ material both in the virgin and compacted state. This was done via plate bearing tests to obtain the static moduli and via falling weight deflectometer (FWD) tests to obtain the dynamic moduli. A limited number of consolidation tests were also done to assess the compressibility characteristics of the in situ materials.

- Testing the consistency (strength) of the soil via hand DCP equipment to limited depths and via a heavy mechanical DCP apparatus to larger depths. These results were correlated with the abovementioned measured stiffnesses to establish whether DCP soundings yielded a sensible indication of stiffness and its variation.

- Measurement of settlement in the compacted areas and to assess its significance as a measure of degree of compaction.

The results of the abovementioned tests and the conclusions drawn from these results are the subject of this report and covered in the sections below. The way in which the report is structured is that all technical discussions are contained in **Volume 1**, as well as summaries of test results. The detail test results are contained in **Volume 2** under various appendices.

In **Volume 1** a description of the site is given in **Section 2** and some basic details on how the compaction trials were performed (**Section 3**). A summary is then given of the tests that were done in **Section 4**, followed by a review of the site characterization tests in **Section 5**. Discussions of the results of each type of test are conducted in **Section 6**, except that the FWD results are discussed separately in **Section 7**. This was done since the application of the FWD directly on a subgrade is a fairly novel application, not only within the realm of pavement diagnostic testing, but also within the context of testing the effectiveness of compaction.

In **Section 8** the findings of the trials are summarized and comments made on:

- The appropriateness of the respective tests and/or their value.
- Any limitations or problems associated with the various tests
- Future work or research that should be done.

2. SITE DESCRIPTION

The site is situated on the outskirts of the town of Kriel in the Thubelihle township in Mpumalanga. The topography comprises mild slopes and vegetation consists only of veld grass.

Geologically the site is underlain by sandstone of the Vryheid Formation of the Ecca Group of Karoo Sequence sedimentary rocks. Bedrock depth varies from 2 m in places to more than 6 m but generally speaking bedrock depth is in excess of 3 m.

The soil profile is fairly simple and uniform. It consists almost entirely of a redbrown clayey fine sandy silt aeolian deposit. On inspection the material is moist and distinctly pinholed and reworked, i.e. highly compressible and voided. It is generally soft or soft to firm but stiff close to surface due to desiccation. Indicator and compaction properties of the soil is discussed in more detail in **Sections 5.2 and 5.4**. Where bedrock was encountered towards the western end of the site, there was very little development of residual soil and consequently the transition to bedrock was rapid. Minor seepage was encountered at bedrock level, indicating that temporary perched water tables are likely to develop on top of the relatively impervious bedrock in wet rainy seasons.

Figure 1: Typical view of in situ soil at a test pit

3. COMPACTION TRIALS

The compaction trials involved treatment of 5 designated sections of the site with a specific compactor each.

The compaction equipment used, were the following:

- a 12-ton vibro compactor
- a 4-sided single drum compactor of which the compaction energy is ± 10 kJ
- two 5-sided dual drum Landpac Impact Compactors of respective energies 10 kJ and 15 kJ
- a 3-sided dual drum Landpac Impact Compactor of which the energy is rated at 25 kJ.

It should be noted that the 4-sided compactor requires a 400 mm strip on either side of the compacted zone for the wheels of the towing tractor as standard practice. Consequently this strip is not compacted and hence this compactor does not give continuous areal compaction coverage as is the case of the Landpac Compactors, unless the compacted zones are aligned to exclude the 400 mm strip.

Each section comprised 4 lanes of which the degree of compaction was varied. The first lane was not compacted and represented virgin soil, while the other 3 lanes were subjected to 20, 40 and 60 passes of the compactors respectively.

The compaction trials were conducted between 8 and 17 September 1997. During the compaction work itself, certain tests were conducted, ie after every 10 passes of the compactor, levels were taken at 20 positions in each lane, as well as 3 hand DCP tests.

Figure 2: Compaction work in progress with the 3-sided 25 kJ compactor

4. SUMMARY OF TESTS CONDUCTED

A summary of the tests that had been conducted appears in **Table 1**. The tests were selected to characterize the site properly and to measure the compaction effects over a fairly wide range of tests. The motivation for conducting the various tests is described under the sections where the results of the tests are reported.

Table 1: Summary of tests performed per compactor type at various levels of treatment (0, 20, 40 and 60 passes)

TEST	VIBRO				10 kJ				15 kJ				25 kJ				4 SIDED			
	0	**20**	**40**	**60**	**0**	**20**	**40**	**60**	**0**	**20**	**40**	**60**	**0**	**20**	**40**	**60**	**0**	**20**	**40**	**60**
Level surveys	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**
Visual settlement																1				
Indicator	1	1		2		1		1		1		1	1			2			1	1
CBR				1						1										1
Moisture content		5	1	5		5	1	5		5	1	5	5	1	1	5		1	5	5
Hand DCP	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	6	6	6	6
Heavy DCP	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Plate bearing	1			3			1	2	1		1	2	3	1	1	7	1			2
In situ density	3			3									4			11				
Consolidation													3			3				
Deflection measurements	10	10	10	10		10	10	10		10	10	10	10	10	10	10		10	10	10

A layout of the test sections and the positions where test pits were made and samples taken are shown on **Drawing 50444/G1/1** in **Appendix A** in **Volume 2** of this report.

Test pits were dug at representative positions across the site to depths of about 3 m to allow inspection and recording of the soil profile, as well as sampling for moisture content, indicator, compaction and consolidation tests. In addition, in situ tests sand replacement tests were also done.

5. MATERIALS CHARACTERISATION

5.1. Soil Profiles

Site characterization is necessary since the results obtained have to be viewed against the background of the type of materials present and their basic engineering properties. It stands to reason that certain observations are not universally applicable on all materials, or that different materials may reflect results differently.

In each test pit the soil profile was recorded and although the soil profile is fairly uniform in terms of material type, the consistencies differ in accordance with the degree of compaction that was attained. Although only a qualitative assessment, the soil profiles reflected the effects of the compaction very well in terms of observed consistencies of the in situ soil. Lack of pinholding was specifically noted in those zones of the soil profile that had been well compacted. The attenuation of consistency with depth was clearly noted in the compacted areas and these results are in good agreement with the results of the hand DCP tests and mechanical DCP tests that were conducted.

5.2. Indicator Test Results

Samples were taken at various positions across the site and submitted for foundation indicator tests. Representative soil samples of each of the five test sections were taken and a total of 13 indicator tests were done. **Table 2** shows the number of indicator tests performed in each of the tests sections.

Table 2: Distribution of indicator tests

NUMBER OF PASSES	VIBRO	10kJ	15kJ	25kJ	4 SIDED
0	1			1	
20	1	1	1		
40					1
60	2	1	1	2	1

These tests were done to determine the type of soils present and the degree of uniformity of the soil properties across the entire site. It is important when comparing the results of the different compactors at the various compaction sections that they were used on the same type of material. In other words for the compactors to be compared to one another in a meaningful manner, it is necessary that the trials have been conducted on the same type of materials under the same conditions.

The results of these tests can be summarised as follows:

Table 3: Indicator test results

Hole no	Depth (m)	Material type	Soil composition			GM	Atterberg Limits		LS	AASHO classification
			Clay & Silt (%)	Sand (%)	Gravel (%)		LL (%)	PI (%)	(%)	
TH 1	0 – 2	Hillwash	51	49		0,56	26	13	7,0	A-6(5)
TH 2	0 – 2	Hillwash	51	49		0,55	25	14	6,5	A-6(5)
TH 3	0 – 2	Hillwash	44	55	1	0,66	22	10	5,5	A-6(2)

TH 4	0 – 2	Hillwash	55	45		0,53	25	12	6,5	A-6(5)
TH 5	0 – 2	Hillwash	47	50	3	0,66	23	12	5,5	A-6(3)
TH 6	0 – 2	Hillwash	44	56		0,65	23	12	5,5	A-6(3)
TH 7	0 – 2	Hillwash	50	50		0,58	26	13	6,0	A-6(5)
TH 8	0 – 2	Hillwash	51	49		0,57	23	12	6,5	A-6(5)
TH 9	0 – 2	Hillwash	44	56		0,65	22	10	5,5	A-6(2)
TH 10	0 – 2	Hillwash	51	49		0,57	24	11	6,0	A-6(4)
POS B	0 - 0.,74	Hillwash	37	63		0,74	17	4	2,0	A-4(0)
POS C	0 – 0,74	Hillwash	51	49		0,59	19	8	3,5	A-4(3)
POS D	0 – 0,74	Hillwash	35	65		0,76	18	8	3,5	A-2-4(0)
Mean			47	53		0,62	23	11	5,4	
Standard deviation			5,9	6		0,07	3	2,6	1,5	

Legend LL = Liquid limit; LS = Linear shrinkage
PI = Weighted plasticity index; GM = Grading modulus

The results in **Table 3** indicate that:

- Generally the hillwash material across the site comprises fine sandy clayey silt with low grading moduli varying between 0,53 and 0,76. The linear shrinkages are low to moderate and vary between 2,0 and 7,0 %. The weighted plasticity indices of the samples tested varied between 4 and 14 %, which indicates a low to moderately plastic material. The tests indicate that this material displays a low potential expansiveness.

- There is a high degree of uniformity of the soil indicator properties over the entire site as all the indicators show very much the same characteristics. The different compaction trials were performed on the same type of material.

The detail indicator test results appear in **Appendix B** in **Volume 2** of this report.

5.3. Moisture Content Results

The moisture content of the in situ soils is very important in view of the fact that it can influence the stiffness of a material, especially in the case of the finer grained soils that are generally slightly or moderately plastic. A soil at a low moisture content can, for example, have a much higher stiffness as the same soil at higher moisture content. Rain did occur between the date of the initial batch of tests and the dates on which the FWD and plate bearing tests were performed. However, hardly any infiltration has occurred and this matter is discussed in more detail shortly.

Soil samples were taken from all five test sections from each of the 20, 40 and 60-pass lanes as well as the 0-pass lane in the 25 kJ test section. These samples were used to determine the moisture content of the soil. The moisture content at 56 different positions on site was determined. These results must be read in conjunction with the plate load and DCP test results.

The envelope of moisture content tests with depth is shown in **Figure 3** below, while the test results are contained in **Appendix C** in **Volume 2** of this report.

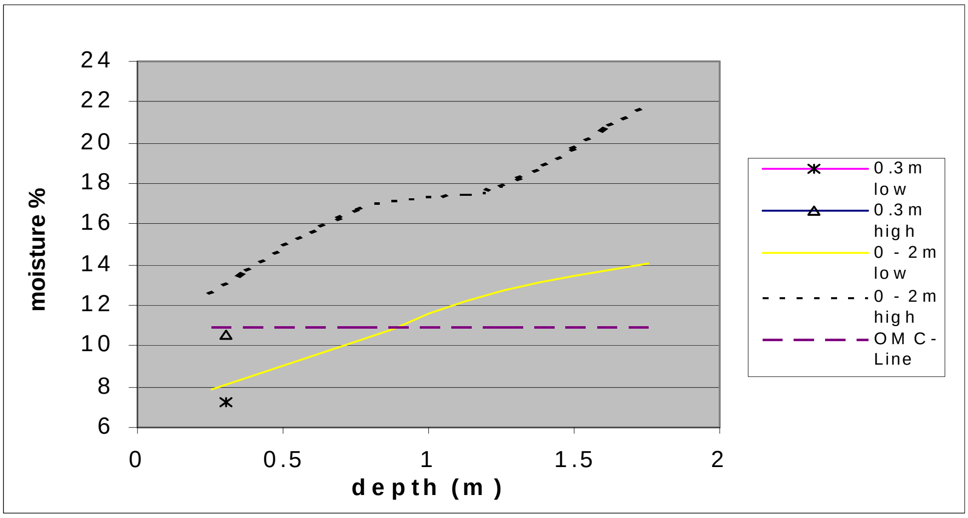


Figure 3: Moisture content envelope

The moisture content samples at 300 mm depth were taken the day after a rainstorm as the deflection measurements were done on that day. It would appear as if the upper layer of the compacted areas has been transformed into a relatively stiff and impervious crust that would not allow easy infiltration of water unless ponding takes place for a considerable period of time. The moisture content samples taken after the rain are slightly lower than those taken earlier, thereby indicating that for all intents and purposes the moisture contents in the test sections remained the same. Consequently it did not affect the tests conducted afterwards.

From the above figure it is evident that the optimum compaction moisture content increases with depth but not drastically. This has the effect that the material in the top horizon of the test sections has a greater stiffness, not only as a result of compaction, but also due to limited desiccation and acts like a capping or a crust.

It is also evident from the above figure that the optimum compaction moisture content (OMC) of the in situ soils, which is between 10 and 11 % according to Section 5.4 below, is generally lower than the prevailing moisture content of the in situ soils except over the upper ± 0,5 m of the soil profile. The significance of this phenomenon is discussed in Section 6.5.

5.4. Compaction Test Results

Compaction tests were done to establish the basic compaction parameters of the soil against which some of the tests that have been conducted, can be gauged by. In addition, the compaction tests were done to establish whether the compaction properties of the soils across the site were fairly similar.

Three samples were taken from different positions to assess the compaction characteristics of the material and to establish whether these characteristics are similar across the entire site. One sample was taken from each of the following test sections:

- Vibro compactor section: 60 passes
- 15 kJ compactor section: 20 passes
- 4-sided compactor section: 60 passes

The compaction test results can be summarised as follows:

Table 4: Compaction test results

Hole No	Depth (m)	Material type	OMC (%)	MDD (kg/m^3)	Swell (%)	Soaked CBR at various Densities			
						90%	93%	95%	97%
TH 2	0 – 2	Hillwash	11,1	1 958	0,5	5	11	18	23
TH 5	0 – 2	Hillwash	9,9	2 020	0,3	19	25	30	33
TH 10	0 - 2	Hillwash	9,7	2 024	0,1	7	15	23	31

Legend
OMC = Optimum moisture content
MDD = Maximum dry density (Mod AASHTO)
Swell = Soaked at 100% Mod AASHTO compaction

The results in **Table 4** indicate that:

- The silty hillwash (in all three test holes) has a moderately high maximum dry density and a moderate optimum moisture content. The CBR swell values are low and the tests yielded moderate CBR values at densities typically specified in the field (93 % to 95 %).
- The properties of the materials from the three test holes vary little and indicate that the compaction characteristics are similar across all the test sections.

The compaction test results, together with the indicator test results, show that there is a high degree of uniformity across the entire site. This implies that compaction results measured in situ should be comparable.

The compaction test results appear in **Appendix D** in **Volume 2** of this report.

6. RESULTS OF TESTS ON COMPACTED SECTIONS

6.1. **Settlement Results**

6.1.1 Level survey

During the course of the compaction trials levels were taken with leveling equipment in all the lanes at 20 positions and the levels averaged to obtain a weighted average settlement against the number of passes. **Figure 4** depicts the magnitude of settlement in the 60-pass lane of the 25 kJ compactor.

From the levels that have been taken during the compaction trials, significant conclusions can be drawn. These results are summarized in **Figure 5** as the cumulative settlement versus number of passes.

When one considers that the settlement must in some way be related to compaction of a certain degree and to a certain depth, the following trends are evident from the above figure:

- The 12-ton vibro compactor has only achieved a total settlement of 150 mm as opposed to 470 to 500 mm of the other impact compactors, except the 560 mm that was achieved by the 25 kJ impact compactor. The 10 kJ, 15kJ and 4-sided compactor have yielded remarkably similar results.
- At a lower number of passes (20), the vibro compactor achieves only about 38 % and 34 % of the settlement of the other compactors and the 25 kJ compactor respectively. After 60 passes the corresponding values are only 30 % and 27 %.

Figure 4: Settlement measurement in 60-pass lane of 25 kJ section

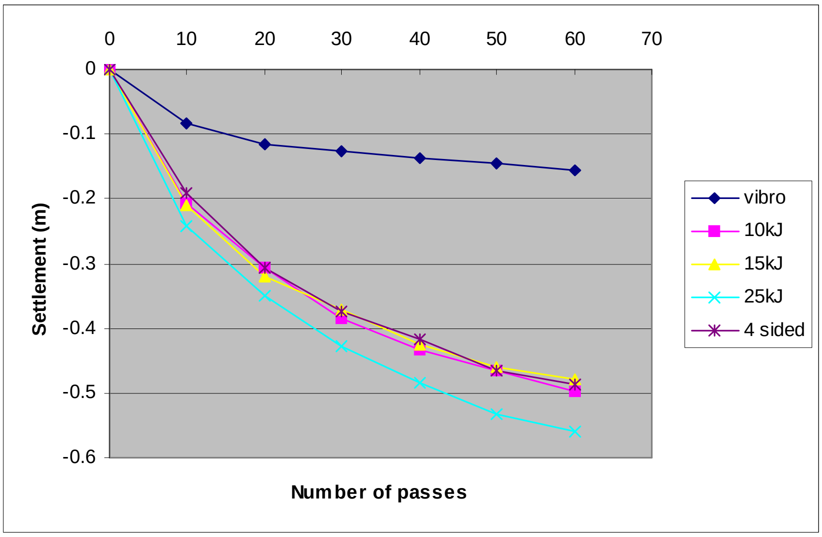


Figure 5: Settlement vs number of passes

- After 60 passes the settlement induced by the impact compactors does not seem to have leveled of entirely. While this may be said of the vibro compactor as well, the increase in settlement beyond 20 passes achieved by this compactor is almost insignificant. The impact compactors, however, still had a significant effect after 20 passes, but increase in settlement leveled of beyond 40 passes. At 20 and 40 passes the respective settlement of all the impact compactors were on average 63% and 84% of the settlement at 60 passes. In the case of the vibro compactor the corresponding values are 77% and 90%.
- Although the settlement induced by the 25 kJ compactor is only about 10 % more than that of the other compactors, it represents significant additional induced stiffness to achieve the additional settlement.

When considering the above mentioned comments, it should be kept in mind that:

- the 4-sided compactor has achieved virtually no settlement in the 400 mm strips adjacent to the compacted lane
- in the compaction process differential stiffnesses are virtually eliminated and a subsoil of uniform stiffness is created.

6.1.2 Visual settlement indicator

A visual indication of the depth of soil that was influenced by the compaction was obtained in the 60-pass lane of the 25 kJ compactor. A narrow strip of soil (0,6 m) was excavated across the 60-pass lane to about 3 m depth before compaction was conducted. The soil was subsequently backfilled but in 300 mm layers, except the top layer which was 500 mm thick and nominally compacted to a density of about 85 to 90%, deemed to be the density of the in situ material. Each layer was separated from the one above by a thin horizontal marker layer of lime. Lime was chosen simply because of its color contrast with the redbrown in situ soil.

After compaction was completed, the strip of soil was exhumed to reveal an elevational view of the marker layers. A view of the exhumed section appears in **Figure 6**.

The marker strip of soil is volume-wise small in relation to the surrounding virgin soil and compacted to a density close to that of the surrounding soil. It would therefore be fair to assume that the behavior of the marker strip and the surrounding soil is very similar.

From the above figure it is evident that:

- Intense shearing of the soil has taken place in the transition zone between the virgin soil and the compacted soil. This is evident from the intense curvature of the marker lines in this area.

- Beyond about 1,5 m depth the influence diminishes rapidly and beyond 2 m depth the marker lines are almost horizontal, indicating little compaction influence beyond this depth.
- The settlement of each marker layer can roughly be measured, and hence the cumulative settlement below that depth. This allows an estimate of the settlement within each marker layer.

Figure 6: A view of the visual settlement section indicating degree of settlement with depth

The estimated settlement within each marker layer is summarized below.

0 – 500 mm:	75 mm
500 – 800 mm:	100 mm
800 – 1100 mm:	125 mm
1100 – 1400 mm:	100 mm
1400 – 1700 mm:	75 mm
1700 – 2000 mm:	50 mm
2000 – 2300 mm:	25 mm
Total	550 mm

The total settlement agrees well with the average 560 mm settlement obtained via the level survey.

It is also interesting to note that the maximum unit settlement is not at the surface. Even though the surface layer is 500 mm thick as opposed to the 300 mm of the lower marker layers, the settlement of the top layer is 75 mm as opposed 100, 125 and 100 mm of the next three layers. The implication is that the highest degree of compaction is between about 0,5 and 1,5 m below surface and the highest strain in the third marker layer is 42 % (125 mm settlement within a 300 mm layer).

6.2 **Hand DCP Tests Results**

Hand DCP soundings were conducted at three different positions on each test section and in each lane after 0, 10, 20, 30, 40, 50 and 60 passes of the various compactors. These tests were valuable in qualitatively comparing the stiffness of the soil with depth in the various sections. The penetration rate of the DCP apparatus is an indication of the consistency of the material. Since the driven cone causes shear failure of the soil, the DCP blow count is not a direct measure of stiffness, but it stands to reason that the penetration rate should be in some way related to stiffness. One should however be mindful that a relationship between the blow count and stiffness is not unique for all soil types.

The moisture content also influences the stiffness of especially finer grained plastic soils and therefor also the penetration rate. From **Section 5.3** it is evident that the moisture content of all the test sections is within a fairly narrow envelope and that the DCP test results will therefore be comparable. The detail DCP results appear in **Appendix E (Volume 2)** and the comparative results for the various sections are shown in **Figure 7a and 7b**.

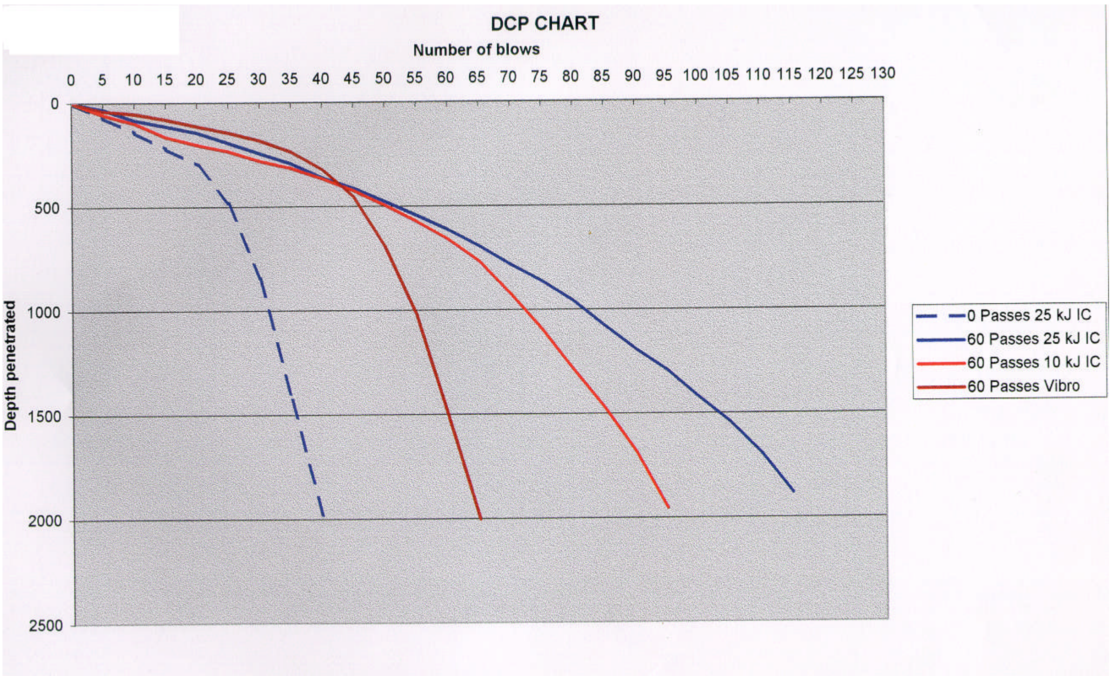


Figure 7a: Comparative DCP results: Number of blows versus depth after 60 passes

From the above figure it is evident that:

- The 25 kJ compactor had the deepest influence on the soil with the 15 kJ, 4-sided and 10 kJ compactors following closely.
- The vibro compactor had more or less the same effect as the other compactors over the top ± 500 mm but its influence on the soil below that depth reduced dramatically in comparison with example the 25 kJ compactor.

The results of DCP soundings are known to be sensitive to variations of moisture content of the soil. Although the moisture content increases somewhat with depth (see Section 5.3) this trend is the same across the site. This implies that over the same depth range the soil moisture content does not vary by more than 4 – 5 % across the site. In addition over the depth range (± 2 m) that the DCP tests were done, the moisture content increases on average by only 7 % This implies a slight increase in blow count with depth attributable to higher soil moisture. Yet the higher blow count deeper down is most likely negated by the higher overburden (confining) stresses which tend to give rise to a higher blow count. These effects have little effect on the *absolute* values of the blow counts and is more of academical interest. For *comparative* purposes it is more irrelevant.

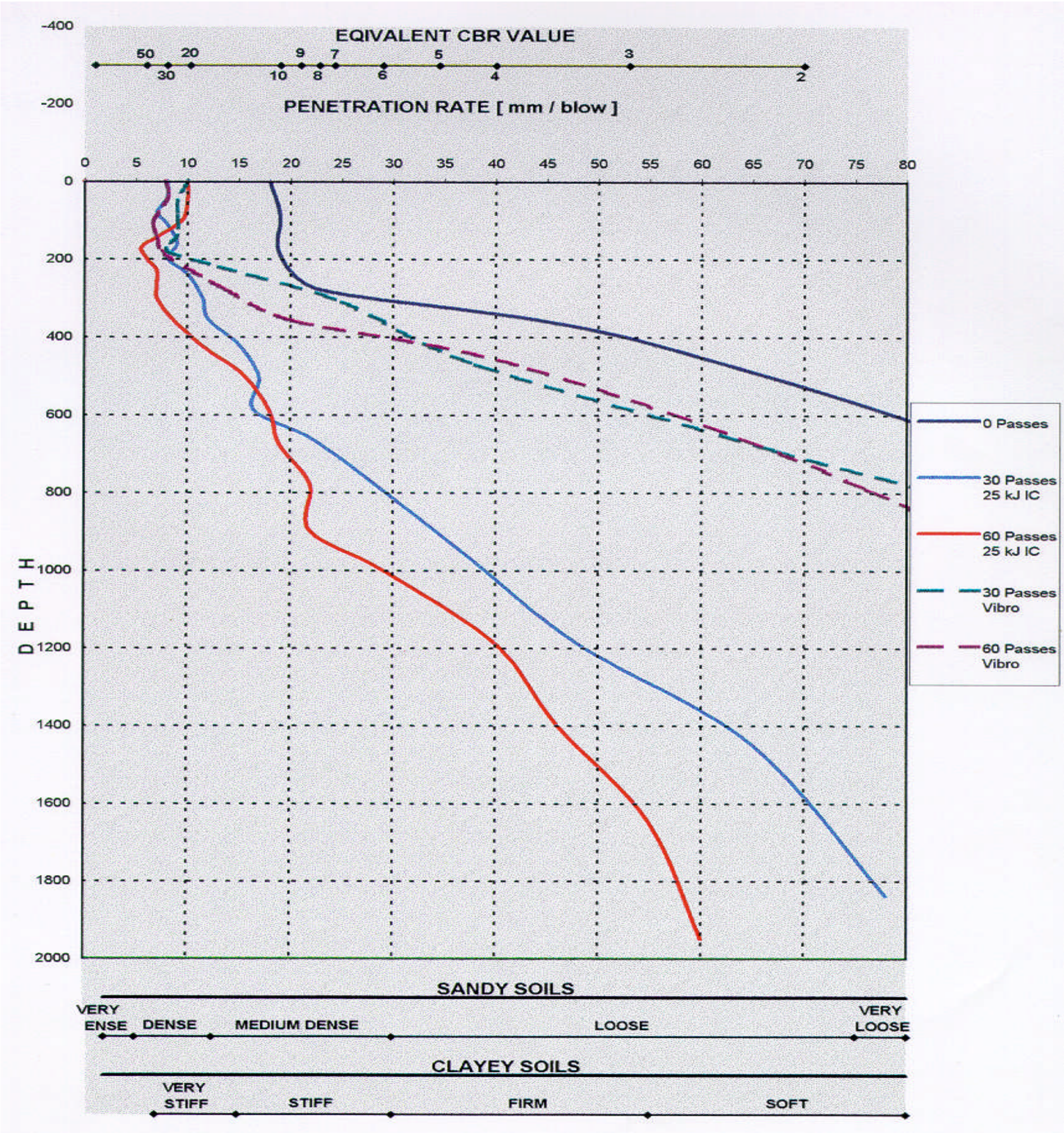


Figure 7b: Variation in DCP blow count with depth: Vibro compactor versus 25 kJ compactor

It was also noted from the DCP tests that the increase in blow count over the upper ± 0,6 m of the profile tends to level off as the number of compaction passes increase. However, this is accompanied by an increase in the blow count below this depth. It would therefore appear as if once the upper part of the profile has reached a certain consistency, it acts as a stiff medium to transfer the compaction energy to larger depths. Consequently the DCP blow counts increase at depth. It is also important to note that the DCP blow counts in the 400 mm wheel track strip of the 4-sided compactor were considerably higher than in the compacted zone itself over the entire depth tested. This refutes the claim, at least for this type of material, that compaction extends effectively into the wheel track strip.

6.2 Heavy mechanical DCP Tests

The heavy mechanical DCP was used on all the test sections in all the different lanes (0 to 60 passes). This equipment utilises a 60 mm diameter disposable cone that is driven into the ground using an SPT trip hammer and the blow count over successive 300 mm depths recorded according to SPT practice. By adding additional driving rods the tests can be conducted to depths of about 15 m. The advantage of this apparatus is that it can test soils to depths far beyond what is attainable with the 1 or 2 m hand operated DCP. The disadvantage of this equipment is that if employed according to SPT practice, the variation in blow count within a 300 mm depth is not reflected. However the user of the equipment is free to record the blow count over lesser depths or simply obtaining the settlement per blow in zones of soil where better definition is required. The same principles mentioned in **Section 6.2** for the hand DCP apply to the heavy DCP.

Three DCP’s were done in each test lane to determine the depth of influence of compaction for each compactor and to compare the results with those obtained from the hand DCP tests. The heavy DCP test results appear in **Appendix F (Volume 2)**. Typical DCP results for the 0 and 60-pass lanes of the vibro compactor and the 25kJ impact compactor, appear in **Figures 8 and 9**.

From the heavy DCP test results it is also evident that the 25 kJ compactor had the deepest influence on the soil it compacted (± 2 m) while the vibro compactor only compacted the top ± 500 mm. The test results indicate that the 25 kJ compactor causes some disturbance of the profile over the top 300 mm, hence the penetration rate of the DCP is higher for the vibro compactor only over this depth.

The DCP tests also clearly reflected consistencies of the soils to the 5 to 6 m depth that the tests were conducted. It is interesting to note the gradual increase in consistency of the soil in the 25 kJ test section and the abrupt refusal at depths of 1,8 m and 2,4 m in the vibro compactor section. This is due to sandstone bedrock present at these levels.

6.4 **Plate Bearing Tests**

Vertical plate bearing tests were conducted because, although they are somewhat time-consuming to conduct, they are simple to execute and provides very useful results on the static moduli of the soil. Since stiffness is the intrinsic and most important engineering parameter in terms of assessing the effectiveness of compaction, it is no doubt a very appropriate test.

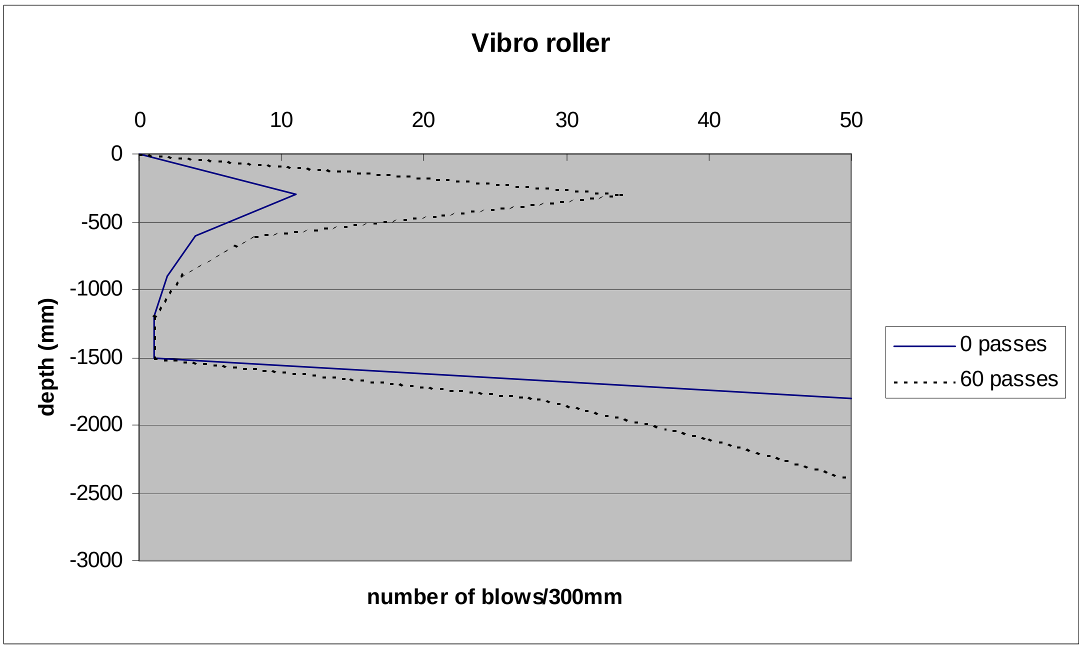


Figure 8: Typical DCP results in vibro compactor section

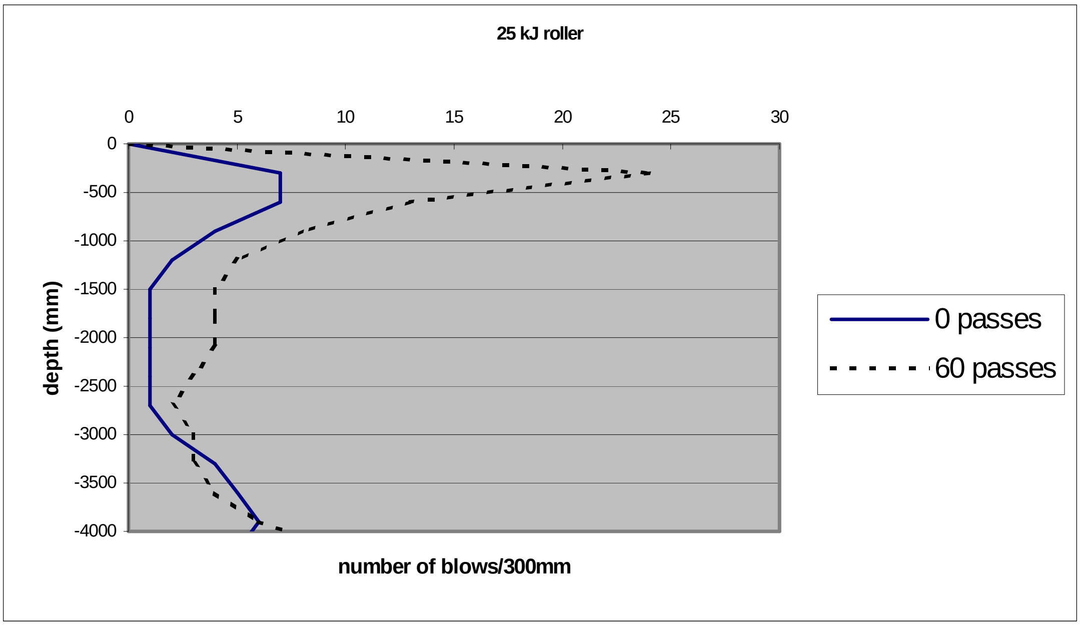


Figure 9: Typical DCP results in 25 kJ compactor section

24 Tests were conducted mainly employing plates of 400 and 600 mm diameter, while in two tests 250 mm plates were used. Reaction consisted of a filled water bowser and depending on the plate size and the 10-tonne capacity of the hydraulic jack, plate stresses of between 350 kPa (600 mm plate) and 780 kPa (400 mm plate) could be mobilized. The first series of plate bearing tests were conducted at surface in the compacted areas and a typical test setup is shown in **Figure 10a**.

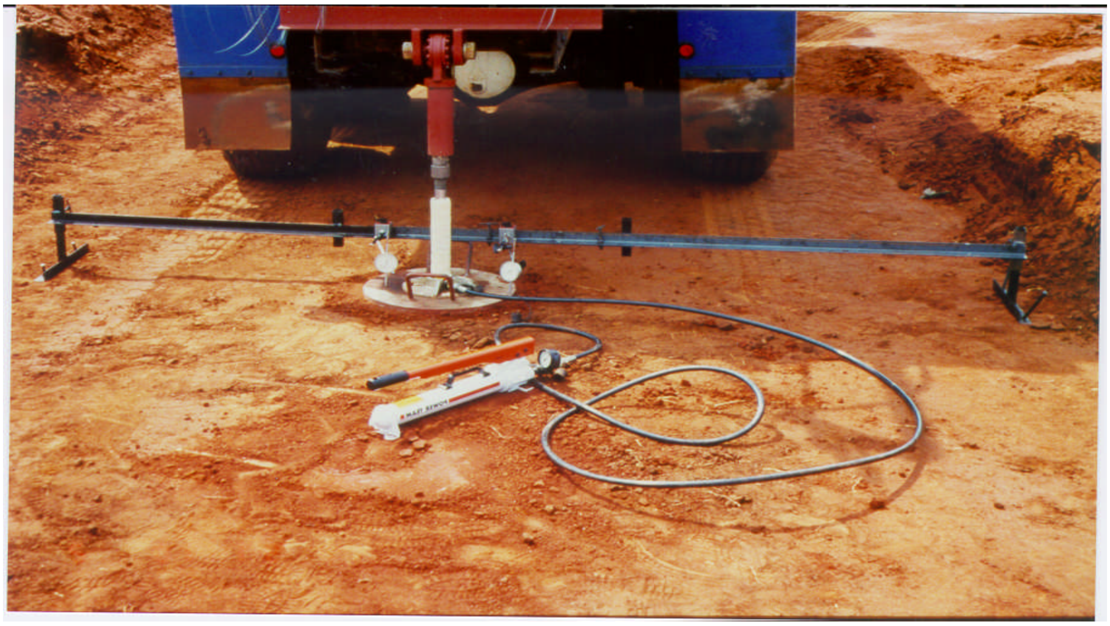
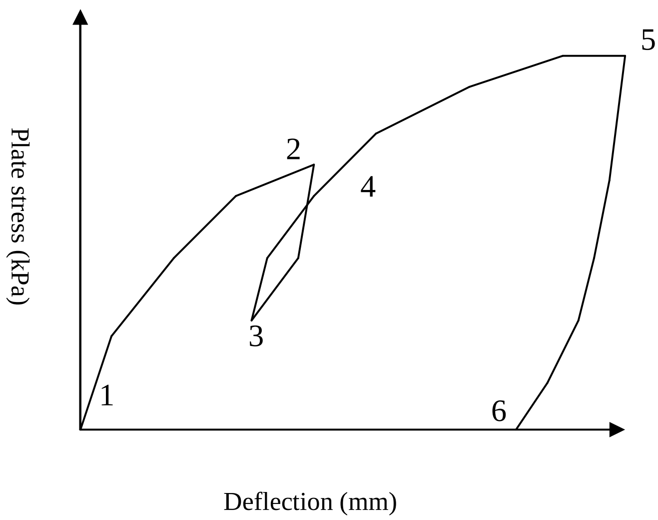


Figure 10a: Plate bearing test in progress

Although both the above stresses are generally well above the stresses imposed on a subgrade as a result of trafficking, the crust (which acts more like a slab) would not allow accurate reflection of the stiffness of the materials at larger depth. Even though the theoretical depth of influence of the abovementioned plates are about 900 and 600 mm respectively, the crust shield the underlying materials fairly effectively from the imposed plate stresses.

Tests included an unloading-reloading loop at about halfway of the stress range and the results of the tests at surface are summarized in **Table 5**. When considering the results it should be noted that the modulus is strictly speaking not the modulus at surface, ie the level at which the test has been conducted. It rather represents the *average* modulus of the top 600 mm and top 900 mm for the 400 mm and 600 mm plates respectively. This implies that at any given depth at which the test is conducted, the average modulus applies to depths of 300 mm and 450 mm respectively, below the test (plate) position on the assumption of a linear attenuation of modulus over the theoretical depths of influence mentioned above.

The format of the results is such that E1 represents the secant modulus of the test from initial loading to the beginning of the unloading-reloading loop; E2 the secant modulus after the unload-reloading loop to the end of the test and E-AVG the *average* secant modulus over the entire stress range. Where sensible unloading reloading moduli could be obtained during the test and unloading moduli at the end of the test, these are represented by E-R1 and E-R2 respectively. Definition of the above parameters are illustrated by way of **Figure 10b.**



E1 - Secant modulus between 1 and 2
E2 - Secant modulus between 4 and 5
E-AVG - Secant modulus between 1 and 5
E-R1 - Average modulus on the rebound curve between 2 and 3
E-R2 - Average modulus on the rebound curve between 5 and 6

Figure 10b: Definition of plate bearing moduli

Table 5: Results of plate bearing tests at surface

COMPACTOR	SECTION	DEPTH (m)	E1 (MPa)	E2 (MPa)	E-AVG (MPa)	E-R1 (MPa)	E-R2 (MPa)	PLATE DIAM (mm)
10 kJ	40-pass	0	17	24	20	606	667	400
	60-pass	0	35	36	35	455	556	400
	60-pass	0	27	24	25	135	137	400
15 kJ	0-pass	0	26	24	24	296	334	250
	40-pass	0	25	30	27	514	718	400
	60-pass	0	45	35	37	553	688	400
	60-pass	0	22	31	25	NA*	854	250
25 kJ	0-pass	0	30	20	25	538	600	600
	0-pass	0	18	8	14	327	NA**	400
	20-pass	0	31	21	24	799	192	400
	40-pass	0	35	38	35	1028	512	400
	60-pass	0	13	14	16	202	NA*	600
	60-pass	0	72	44	49	197	139	400
	60-pass	0	51	38	41	899	NA***	400
4 sided	0-pass	0	36	22	27	600	515	400
	60-pass	0	19	20	19	538	806	600
	60-pass	0	62	32	40	959	NA****	400
Vibro	60-pass	0	21	13	15	227	115	400

* Negative value
** Failure occured
*** Too steep slope
**** Second rebound curve has not been created

It should be noted that the magnitude of unloading-reloading (rebound) moduli are not highly dependant on the stress level at which unloading takes place. It stands to reason that the values of E1, and E2 are dependent on the stress range over which these moduli are measured, except of course if the curve approximates a straight line and in which case E1, E2 and E-AVG are essentially the same.

Table 5: Results of plate bearing tests at surface

COMPACTOR	SECTION	DEPTH (m)	E1 (MPa)	E2 (MPa)	E-AVG (MPa)	PLATE DIAM (mm)
10 kJ	40-pass	0	17	24	20	400
	60-pass	0	35	36	35	400
15 kJ	0-pass	0	26	24	24	250
	40-pass	0	25	30	27	400
	60-pass	0	45	35	37	400
	60-pass	0	22	31	25	250
25 kJ	0-pass	0	30	20	25	600
	0-pass	0	18	8	14	400
	20-pass	0	31	21	24	400
	40-pass	0	35	38	35	400
	60-pass	0	13	14	16	600
	60-pass	0	51	38	41	400
4 sided	0-pass	0	36	22	27	400
	60-pass	0	19	20	19	600
	60-pass	0	62	32	40	400
Vibro	60-pass	0	21	13	15	400

These test results are significant because they represent the actual stiffness measured at surface and therefor represent to some degree the support that the subgrade offers a pavement to be constructed on top of it. It is apparent that the stiffnesses are fairly substantial. The E1 values, for example, vary between 17 and 62 MPa but there is no distinct trend of increasing modulus against increasing number of passes. In places the 0-pass modulus exceeds the 60-pass modulus for a given type of equipment.

However, under the dynamic loading conditions which are characteristic of pavement conditions, it is necessary to establish what the effects of compaction are at depth not influenced by the crusting effect. The plate bearing tests did not reflect trends between the various zones well. The reason for this, in hindsight, was fairly obvious. The DCP results (both hand and mechanical) reflected a crust or capping material in all the lanes of the test sections. This crust was even present in the 0-pass lane mainly due to some degree of desiccation but also the 1-pass vibro compactor compaction mentioned earlier and strengthening of the soil by grass roots over the top 300 mm. The latter is not reflected by the DCP results.

Consequently, 8 tests were conducted at various depths below surface employing only the 400 mm plate to establish what stiffnesses were obtainable if the crustal effect could be bypassed. The results are summarized in **Table 6**. As mentioned above, it should be noted that when using a 400 mm plate, the measured stiffness represents the average stiffness at a position 300 mm below the plate.

Table 6: Results of plate bearing tests below ground surface

COMPACTOR	SECTION	DEPTH (m)	E1 (MPa)	E2 (MPa)	E-AVG	E-R1	E-R2	PLATE DIAM (mm)
10 kJ	60-pass	0,58	27	24	25	135	137	400
25 kJ	0-pass	0,44	11,4	0,6	-	-	-	400
	60-pass	0,45	72	44	49	197	139	400
	60-pass	1	15	8	2	98	90	400
	60-pass	1,5	2,4	1,4	1,7	57	103	400
Vibro	0-pass	0,52	-	0,3	0,6	-	-	400
	60-pass	0,55	9,1	1,1	-	-	-	400
	60-pass	1	6	1,2	-	-	-	400

From **Table 6** it is evident that:

- The E1-stiffness in the 0-pass lanes are 11,4 for the 25 kJ compactor and <1MPa for the vibro compactor at respective test depths of 0,44 and 0,52 m. This clearly indicates that crusting has no effect here. It is interesting that the E2 and E-avg values are <1MPa.
- The E1-stiffnesses in the 60-pass lanes of the 10 and 25 kJ compactors at ± 0,5 m depth are 27 and 72 MPa respectively. While the former value is moderate, the latter is very high within the context of a compacted material which is primarily of a silty nature. The E2 and E-avg stiffnesses are somewhat lower as can be expected of a silty material which softens on straining.
- The stiffness at about the same test depth (0,55 m) in the case of the vibro compactor is only 9,1 MPa and the E2 and E-avg values deteriorate rapidly.
- At a test dept of 1 m, the E1 value is still 15 MPa in the case of the 25 kJ compactor but only 6 MPa in the case of the vibro compactor. In the latter case the modulus is deemed to be that of the in situ soil, as the other tests indicate no effect of the vibro compactor beyond 0,6 m depth.

Comparison of the plate bearing moduli with those derived from the consolidometer and FWD tests are discussed in **Sections 6.5** and **7.3.2** respectively.

Figure 11 shows a typical depth vs modulus plot for the results of the 25 kJ and vibro compactors.

The plate bearing test results appear in **Appendix G** in **Volume 2** of this report.

6.5 Consolidation Tests

Six consolidation tests were done on undisturbed samples of material from the 25 kJ test section. The undisturbed samples were taken at depths of 0,75 m, 1,5 m and 2,25 m in each of the 0-pass and 60-pass lanes. The detail consolidation test results appear in **Appendix H (Volume 2)** and a summary of the results appear in **Table 7**.

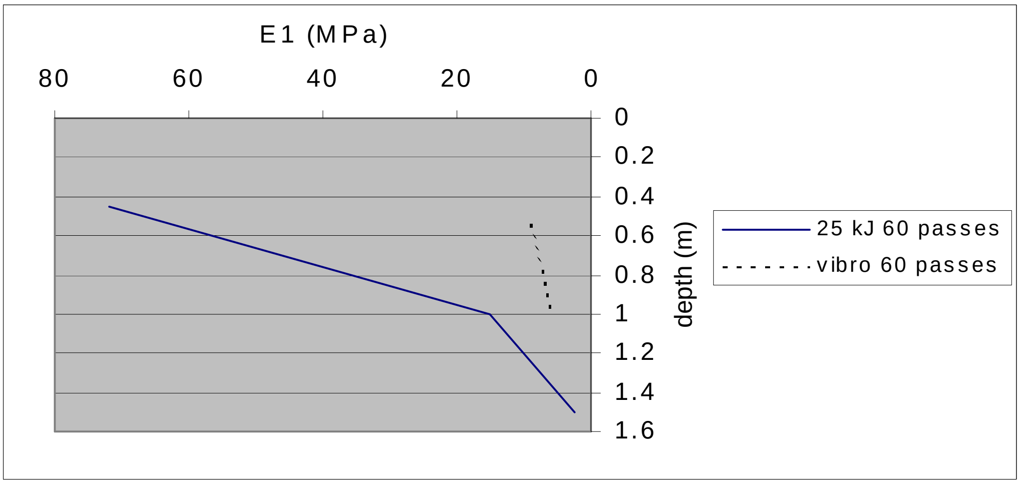


Figure 11: Typical depth vs modulus results

Table 7: Summary of consolidation test results.

	0-PASS LANE			**60 –PASS LANE**		
DEPTH (m)	0,75	1,5	2,25	0,75	1,5	2,25
E* (MPa) **(0-100 kPa stress range)**	1,8	1	1,6	11,8	11,1	5,3
E* (MPa) **(100-200 kPa stress range)**	1,8	1,8	1,8	13	12,2	2,9
DRY DENSITY (kg/m^3)	1494	1409	1479	1712	1677	1479
Initial void ratios	0,800	0,906	0,826	0,570	0,604	0,760

*Constrained moduli over the stress ranges indicated

The test results show that:

- The material has a much higher initial dry density (and lower void ratio) in the 60-pass lane than in the 0-pass lane and decreases with depth in the 60-pass lane. In the 0-pass lane the densities are fairly consistent and relatively low, i.e. between about 1 400 and 1 500 kg/m^3.
- The constrained moduli (E-values) follow similar trends as the densities and decrease dramatically below 1,5–2 m which means that the compactor did not have much effect below these depths.

According to **Section 5.3** the moisture content over the upper 1,5 m of the profile ranges typically between 12 and 18 %. At these moisture contents and assuming an average void ratio of 0,84 for the virgin soil, the degree of saturation varies between 38 % and 58 %. This implies that although the moisture content is above OMC it is still well below saturation (zero air voids) and hence would not affect the compaction process adversely as a result of excess pore pressure as the soil densifies. However one would expect that as the soil reaches a high degree of densification over about the top 1 m of the profile, even a slightly higher density and stiffness could be achievable, had the soil been at OMC.

Figure 12 and 13 reflect the abovementioned trends.

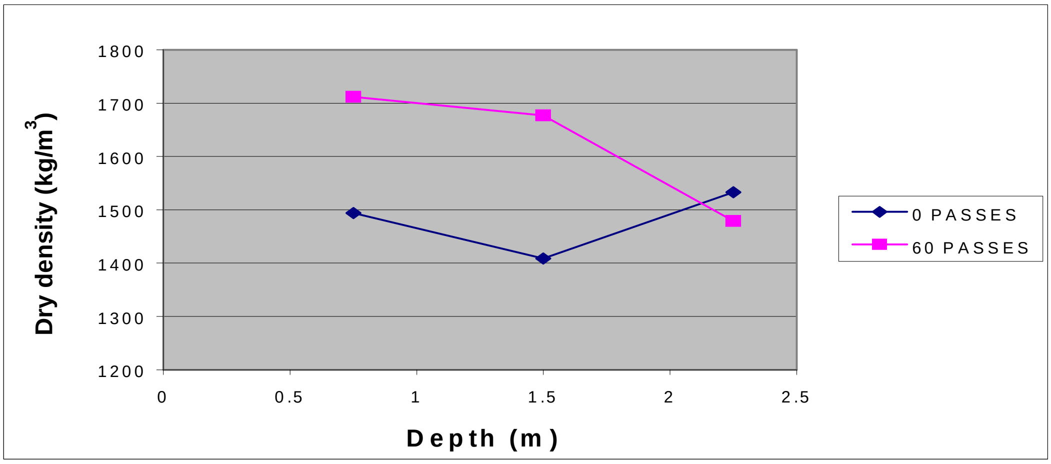


Figure 12: Consolidometer dry density versus depth and compaction effort, 25 kJ compactor

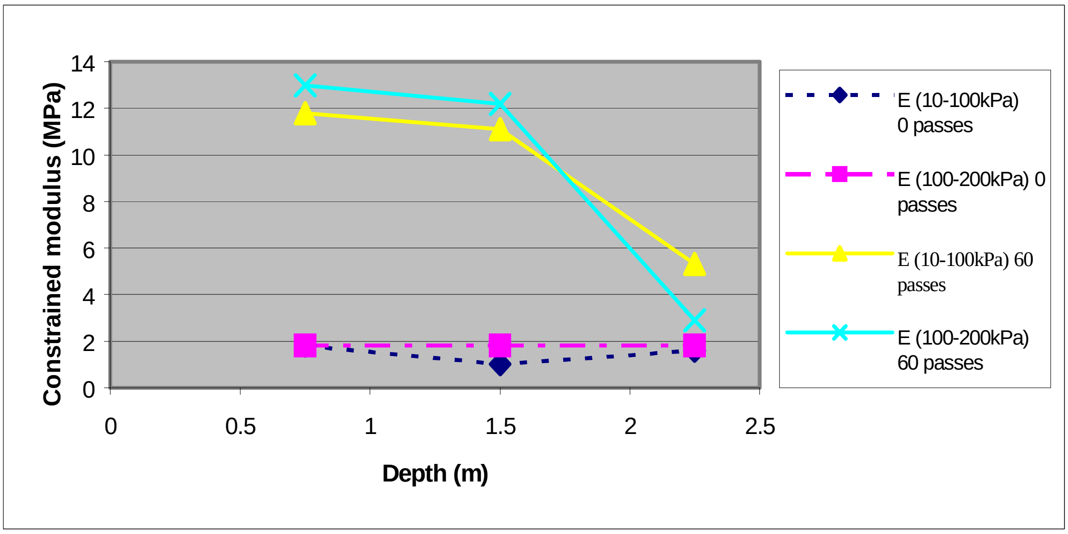


Figure 13: Consolidometer constrained modulus versus depth and compaction effort, 25 kJ compactor

Although not strictly comparable since they measure moduli in a somewhat different manner, it makes sense to compare plate bearing moduli with consolidometer moduli. In terms of depth and the fact that the plate bearing apparatus measures the stiffness below the test level, it makes sense to compare plate bearing moduli at depths of 0,45 and 1 m with consolidometer moduli at 0,75 and 1,5 m respectively. The plate bearing moduli **(Table 6)** are 72 and 15 MPa respectively. The corresponding consolidation moduli are 12 and 11 MPa. The 72 MPa value is exceptionally high and the 27 MPa stiffness obtained from the 10 kJ compactor at 0,58 m depth seems more representative. It should be noted that due to lack of confining stress in the consolidometer one would expect that the consolidometer moduli should be somewhat lower than the plate bearing moduli.

6.6 In Situ Density Tests

In situ density tests via the sand replacement method were done at various depths in the 0 and 60-pass lanes of each of the vibro and 25 kJ compactor sections. The results of these tests appear in **Appendix I (Volume 2)**.

Table 8 is a summary of the in situ density as a percentage of the average maximum dry density as determined from the compaction test results in **Section 5.4**.

Table 8: Summary of measured in situ densities

SECTION	LANE	TEST HOLE	DEPTH (m)	FIELD DENSITY (kg/m^3)	% OF AVERAGE MAXIMUM DRY DENSITY
25 kJ	0 passes	3	0,855	1526	76
	0 passes	3	1,455	1487	74
	0 passes	3	2,105	1411	71
	0 passes	3	2,805	1381	69
25 kJ	60 passes	4	0,725	1800	90
	60 passes	4	1,375	1573	79
	60 passes	4	2,175	1552	78
	60 passes	4	2,875	1632	82
25 kJ	60 passes	A	0,675	1833	92
	60 passes	A	1,225	1750	87
	60 passes	A	1,875	1625	81
	60 passes	A	2,475	1485	74
Vibro	0 passes	B	0,335	1659	83
	0 passes	B	0,575	1481	74
	0 passes	B	0,815	1400	70
Vibro	60 passes	C	0,335	1639	82
	60 passes	C	0,575	1547	77
	60 passes	C	0,815	1493	75

From **Table 8** it is evident that:

- in the 0-pass lanes, except where some crusting is present over the top 0,5 m, the in situ densities are between 70 and 75 %.
- tests have not been conducted at shallow depth (0 – 0,5 m) in the compacted lanes. Between about 0,7 and 1,5 m the densities are between 92 and 79 % indicating a considerable improvement in the case of the 25 kJ compactor.

This together with **Figure 14,** which is a typical comparison of in situ density vs depth for the 25 kJ and vibro compactor, is an indication that the 25 kJ compactor has a much greater effect on the density of the material and the depth of compaction than the vibro compactor.

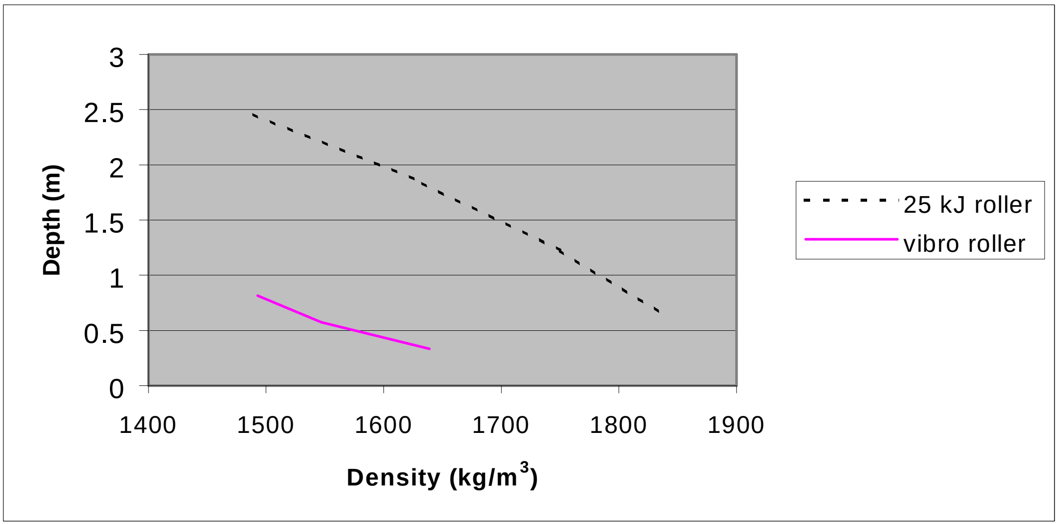


Figure 14: Typical results of in situ density vs depth

7. FALLING WEIGHT DEFLECTOMETER (FWD) TESTS

7.1. INTRODUCTION AND BACKGROUND

Non-destructive deflection testing is commonly used as part of the structural evaluation of road pavements. Dynamic loading devices have become popular because their field operation is relatively simple, fast and economical. The Falling Weight Deflectometer (FWD) has evolved as one of the favourite and most suitable devices for pavement evaluation. One of the objectives of this investigation was to determine the feasibility of using the FWD to verify the compaction achieved with different compaction equipment on in situ material i.e. typical subgrade.

Strips of 10 m length were prepared in the centre of each of the three lanes (20, 40 and 60 passes) within the five test sections. Only one of the 0-pass lanes (25 kJ section) was tested and deemed representative of the virgin material across the site.

The objectives of this section are:

- to present and discuss possible trends with regard to number of passes and type of machine, determined through analysis of deflection data;
- to discuss the correlation of these trends with trends from other tests used during this investigation;

- to comment on the significance of material improvement through practical pavement design application;
- to comment on the applicability of using the FWD to verify and establish altered material characteristics.

This section commences with an introduction of the Falling Weight Deflectometer (FWD). The concept of deflection basin parameters is introduced as a means to investigate possible trends, and results are presented. This section includes results from further analysis of deflection data, which were carried out in an attempt to verify the influence of compaction with depth, and to quantify elastic moduli of the compacted material. The significance of the results is discussed and comments made on the applicability of using the FWD for testing of compacted in situ material.

7.2 Falling weight deflectometer (FWD)

Figure 15 is a schematic representation of the FWD. This lightweight trailer-mounted device consists of three primary parts, i.e. the trailer (with hydraulic unit, weights and transducers) , the Dynatest 8600 System Processor and a portable laptop microcomputer. It can be towed by a passenger car, van or truck, which is also needed to supply 12V/DC power to the complete system.

An impulse load is applied to the pavement through a circular loading plate. The FWD is equipped with a standard 300 mm diameter rigid loading plate, with a rubberised pad to help distribute the load evenly. A 450 mm plate and rubberised pad are also supplied. Uneven pavement surfaces are accommodated by the plate which is split into two halves. The applied load, measured by a precision heavy duty load cell above the loading plate, results in a deflection of the pavement surface. The deflection is measured by high speed velocity transducers (deflectors) which are factory calibrated. The whole procedure is controlled from the inside of the towing vehicle through the laptop microcomputer. The system processor scans and processes the FWD measured load deflection signals, and have separate channels which can be mounted on the system at any one time. This system temporarily stores the complete digitised output of each channel at each drop during the FWD loading sequence. This simultaneous multi-channel output contains the deflection basin data.

The applied load can range between 7 and 120 kN. The FWD produces a single impact load, which is essentially half-sinusoidal and 25 to 30 milliseconds in duration, closely approximating the effect of a wheel moving at 60 to 80 km/h. The FWD trailer has an integrated raise/lower sensor bar which is remote controlled. There are seven seismic deflection transducers in movable brackets mounted along the 2,25 m bar (two optional extra ones may be added). All components of the FWD equipment are weather resistant and can operate in the temperature range of -20ºC to +40ºC.

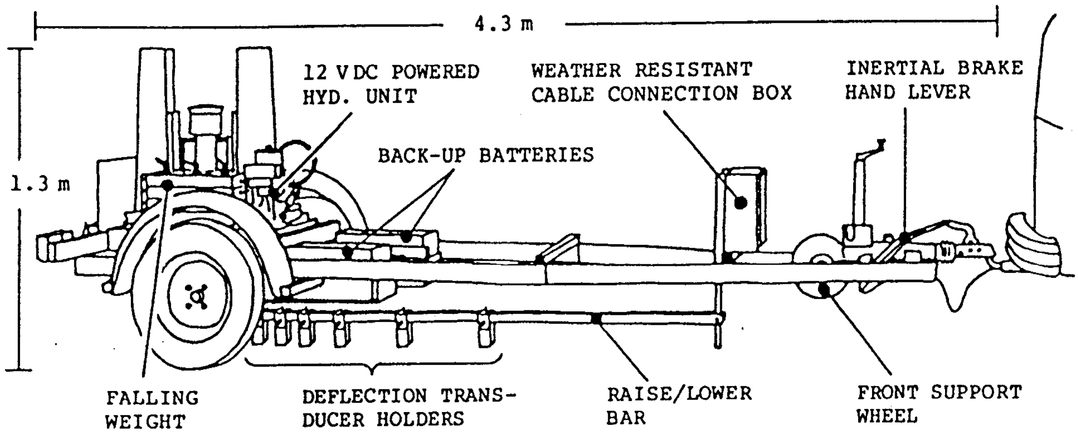


Figure 15: Schematic representation of the Dynatest 8002 Falling Weight Deflectometer

The deflection range is 2,0 mm, and deflections are measured with an absolute accuracy of better than 2% $\pm$ 2 microns. The resolution of the equipment, in terms of deflection is one micron (0,001 mm). The load is measured with an accuracy of better than 2% $\pm$ 0,07 kN. The resolution of the equipment (in terms of load) is 0,07 kN (= 1 kPa mean stress over a 30 cm diameter loading plate).

A test sequence is identified and programmed from the computer keyboard, i.e. site identification, height and number of drops per test point etc. The sequence involves the lowering of the FWD loading plate and raise/lower bar carrying the deflectors to the test surface, drop of weights (usually 3 times) and raising the bar again. The latter lasts approximately 35 seconds, depending on the magnitude of the load. The set of measured data is displayed on the computer for direct visual inspection. (Dynatest, 1995).

7.3. Analysis of measured deflection basins

For the purpose of pavement evaluation, loads are usually applied at a magnitude of 40 kN, simulating the wheel of a truck with an 80 kN axle load moving at a speed of 60 to 80 km/hr. On-site inspection of the deflection basin data showed maximum deflections more than 2,0 mm. It was decided to perform tests at both 25 kN and 40 kN since the latter load application was preferred and some measurements were within the stated range. The first phase of the analysis of data was therefor to compare deflection basins at the two different load applications. This comparison revealed that the variation in data is position rather than load associated, and that similar basin shapes exist for the two load cases. Both processed data sets were used in the initial analysis (deflection bowl parameter analysis) which confirmed the latter. The 40 kN data set was used for further analysis.

Two methods commonly used to analyse deflection basins were identified to give an indication of possible trends, i.e. deflection basin parameters and the back calculation of elastic moduli. The latter could further be used to give an indication of the compaction influence with depth and to quantify the possible contribution of the compacted material as part of a pavement structure.

7.3.1 Deflection basin parameters

A number of deflection basin parameters are used to analyse measured FWD deflections. The parameters commonly used are listed in **Table 9** and defined in **Figure 16.**

Table 9: Deflection basin parameters

PARAMETER	DESCRIPTION	FORMULA
Y_{max}	Maximum Deflection	$\delta 0$
BLI	Base Layer Index	$\delta 0 - \delta 300$
MLI	Middle Layer Index	$\delta 300 - \delta 600$
LLI	Lower Layer Index	$\delta 600 - \delta 900$
D7	Deflection at Outer Sensor	$\delta 1\,500$
δ - Deflection (mm) at offset *r* from the load.		

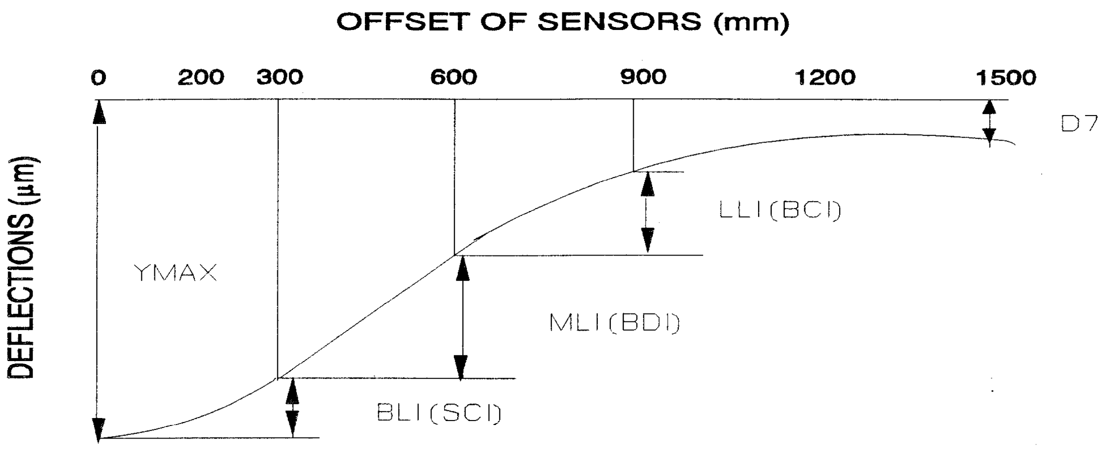


Figure 16: Deflection basin parameters defined (Maree and Jooste, 1992)

Figure 16 shows that the deflection basin parameters describe the sub-surface condition in terms of zones, rather than layers. The use of these parameters in this particular investigation, which differs from conventional pavement investigations, is appropriate in view of the fact that no defined layers exist. The parameter Y_{max}, the peak deflection measured under the FWD is, theoretically influenced by the stiffness of the total area influenced by the load. Experience however revealed that this parameter is particularly sensitive to changes in the stiffness of the top 300 mm. Typically, the magnitude of BLI strongly correlates with the stiffness of the top 200 mm, while the Middle Layer Index (MLI) is typically influenced by the stiffness of material at a depth of between 200 and 400 mm. In terms of pavement structures, the Lower Layer Index (LLI) normally gives an indication of the stiffness of the top of the subgrade (in situ material or foundation of pavement structures), while the D7 parameter is purely a function of the subgrade stiffness as well as shallow (±1,8 m) but stiff rigid layers (actual or apparent) underlying the subgrade.

Results obtained from deflection basin parameters in terms of influence of various compaction equipment with number of passes and relative performance of different compaction equipment are graphically presented in **Appendices J1** and **J2** respectively.

All graphs are expressed in terms of percentage improvement relative to the 0-pass lane. Although soil tests indicated a fair uniformity, it was found that with depth, some features such as natural stiff layers and moisture may influence the results. The improvement relative to this uncompacted lane thus serves as an indication of possible trends, and can not directly be interpreted as the final and true improvement achieved with any compactor. The results are discussed in the following sections. It should be noted that the depth of influence does not form part of this discussion. The next phase of analysis concentrated on determining elastic moduli to complement the findings based on deflection basin parameters.

A general indication of the influence of different compaction equipment on maximum deflection, is given in **Table 10**. A coefficient of variation of 30 % was the measure used to evaluate the acceptance of a data set for analysis purposes. The 4-sided (40 passes) and 15 kJ (60-pass) results have high coefficients of variation which can be ascribed to material variables, which include moisture content variation at the surface, however not necessarily further down. Detailed discussion on soil moisture content is covered in section 5.3. Absolute values of Y_{max} are shown to give an indication of the magnitudes of deflection which can be encountered after compaction of such a material. The maximum deflections obtained for the two sections outlined above, are unrealistically high and should be considered during interpretation and processing of data. The effect achieved with different compaction equipment, based on maximum deflection, is dealt with in **Section 7.4.1.**

Table 10: Influence of equipment and passes on average maximum deflection (µm)

EQUIPMENT	20 PASSES	40 PASSES	60 PASSES
Vibrating	2670 (14, 0)*	1880 (12, 0)	1857 (12, 0)
4-Sided	1585 (26, 0)	1911 (27, 5)	1648 (20, 1)
10 kJ	1868 (9, 0)	1795 (18, 0)	1478 (20, 0)
15 kJ	1236 (8, 0)	1162 (25, 1)	1868 (23, 3)
25 kJ	1603 (21, 0)	1266 (25, 0)	1462 (17, 0)

* [Coefficient of variation (%), number of outliers removed]

Results, graphically presented in the Appendix, indicate that the pronounced effect of all the impact compactors is concentrated in zones reflected by MLI, LLI and D7. Sections compacted with the 15 kJ and 25 kJ compactors tend to show a decrease in the BLI parameter from 40 to 60 passes. A more detailed discussion follows.

The ***vibro compactor*** appears to have no significant influence after 40 passes. The maximum deflection supports this statement firmly, whereas only a small improvement can be discerned from the other parameters after 20 passes. The BLI, MLI and Y_{max} parameters can be used to assess the improved material characteristics due to the influence of this compactor. This is the only section with a ‘shallow’ natural stiff layer at an approximate depth of 1,8 to 2,4 m which could contribute to the apparently good performance of this device. The improvement indicated by LLI and D7 parameters should therefor not be accepted as reliable trends.

The 40-pass lane of the section compacted with the ***4-sided impact compactor*** showed a high variability in data (Table 10). The graphically presented results indicate that all the parameters are influenced with the emphasis on the MLI and LLI. The Y_{max} parameter remains constant after 20 passes which relates to a decrease in the BLI after 60 passes and increase in MLI, LLI and D7. The former indicates that compaction after 20 passes may be destructive to the upper zones.

Similar trends were observed from results obtained with the ***10 kJ impact compactor*** . Raw data tends to show low variability on all compacted lanes. Consequently, these trends can be accepted with greater confidence. A positive improvement in all parameters occurred with a concentrated influence on the MLI and LLI and D7 parameters. The BLI remains constant from 20 to 40 passes and improves from 40 to 60 passes.

The 40 and 60-pass lanes of the ***15 kJ impact compactor*** indicated a relatively high variability of unprocessed data. Significant improvements in all parameters were obtained after 20 passes with the emphasis on the MLI, LLI and D7 parameters. The Y_{max} remains constant from 20 to 40 passes where after its initial improvement deteriorates, owing to possible destruction of the upper soil zone, represented by BLI and MLI. However the LLI and D7 parameters were positively influenced up to 60 passes.

Similar trends were established from results obtained from the section compacted with the ***25 kJ impact compactor*** . The Y_{max} and BLI parameters decreased from 40 to 60 passes, while MLI, LLI and D7 were still improving from 40 to 60 passes.

The relative performance of different compaction equipment is shown in **Appendix J2.** The presentation of this performance, in terms of improvement of the subsurface conditions, is simplified by separation into influence on the upper and lower soil zones. This comparison is given in **Table 11** where the rating of the various equipment decreases from 1 to 5. **Table 10** indicated that a high degree of variation occurred on the 4-sided 40-pass section which implies that an improvement after 40 passes is most likely to occur, hence a 40 % maximum improvement is therefor conservative. The relative maximum improvement between the different devices does not appear significant in the lower zones. However the relative effect between the machines becomes less prominent with increased number of passes as shown in **Appendix J2**. It should be borne in mind that the difference in performance of the rated equipment is small and that no significant conclusions can be drawn from **Table 11**.

Table 11: Comparison of equipment based on improvement deflection basin parameters relative to no compaction

UPPER ZONE (BLI)			LOWER ZONE (D7)		
Equipment ranking	**Passes**	**Improvement**	**Equipment ranking**	**Passes**	**Improvement**
15 kJ (best)	40	56% (77%)#	15 kJ (best)	60	66% (90%)*
25 kJ	40	52% (75%)	25 kJ	60	65% (84%)
4-Sided	20	40% (71%)	10 kJ	60	62% (85%)
10 kJ	60	40% (81%)	4-Sided	60	58% (83%)
Vibro	40	35% (48%)	Vibrating	-	-

Improvement in MLI given in brackets,* Improvement in LLI given in brackets

7.3.2 ELASTIC MODULI BACK CALCULATED FROM FWD DEFLECTION BASINS

This part of the analysis was carried out to verify the findings of the deflection basin parameters, to give an indication of the influence depth of various compaction equipment, and to obtain approximate values of elastic moduli for the compacted material.

Nowadays, in-depth deflections (Multi-Depth Deflectometer) or deflection basins on the surface (Falling Weight Deflectometer) are used to determine pavement layer moduli (with back calculation), and the moduli to compute stresses or strains (forward calculation) which can be used to evaluate structural capacity. The back calculation of elastic moduli basically involves the calculation of the displacements at the surface with a combination of selected E-values. The calculated and measured deflection basins are then compared and the percentage error at each sensor calculated. The E-values are changed to obtain the closest fit. An average absolute error per sensor of 2% indicates a good fit and errors greater than 8% are high. This is however a tedious process and many semi-automated and fully-automated back calculation programs exist. Most back calculation programs can accommodate only up to a maximum of 5 layers, and are equipped with the facility to determine an apparent rigid layer in order to compensate for natural stiff layers and stress sensitivity of subgrade materials.

These commercially available back calculation programs would not be applicable for calculation of E-values during this investigation, since the depth of influence was of importance. The multi-layered elastic program BISAR (forward calculation program) was used to calculate the displacements at the desired sensor offsets. A total of ten layers could be accommodated. As mentioned before, this investigation differs in that no specific layers are defined. Deflection basins were initially back calculated for each lane at predefined layer thicknesses of 300 mm up to a depth of 3 000 mm. These basins were then used to calculate the E-values for a 150 mm interval up to a depth of 1 500 mm. Interpretation of results and comparison with Heavy Dynamic Cone Penetrometer (HDCP) results as well as deflection basin parameters indicated that a different approach is needed to obtain more realistic values. An optimum combination of layer thicknesses were obtained which indicated the sensitivity of this analysis to this additional variable, i.e. layer thickness. One representative basin was analysed for each lane.

The results from this analysis, according to which the elastic moduli with depth and the influence of the various compaction equipment against number of passes are considered, are contained in **Appendix J3**. Comparison between different compaction equipment in terms of elastic moduli with depth appear **in Appendix J4**:

The 25 kJ section 0-pass lane has a fairly uniform and soft soil profile relative to other sections according to HDCP results. However this was the only 0-pass lane tested, and these conditions can not be used without cognisance of possible variation in sub-surface conditions. Graphical inspection of trends was used to identify possible original soil conditions in order to create a representative envelope. The lines identified are presented on the graphs as dotted lines and served as an aid to establish ranges of depth of influence for different compaction equipment. **Table 12** summarises the findings and is complemented by Dynamic Cone Penetrometer (DCP) established ranges. It should be appreciated that the calculation of moduli from surface deflections at such depths is not common practice and that these back calculation methods are primarily for use at shallow depths. In general these values appears to be high and should not be considered as absolute answers.

Table 12: Depth of influence of different compaction equipment

EQUIPMENT	DEPTH (m) OBTAINED FROM E-VALUES	DEPTH (m) OBTAINED FROM DCP RESULTS
Vibrating compactor	500 - 700	500 - 600
4-Sided Impact	2100 - 2500*	1600 - 1800
10 kJ Impact	2100 - 2500	800 - 1600
15 kJ Impact	2200 - 2500	1200 – >1600
25 kJ Impact	2200 - 2500*	> 2000

* Possible higher maximum value

Typical back calculated E-values are presented in **Table 13**. The findings based on deflection basin parameters are generally confirmed, and more detailed information presented with regards to material characteristics. These calculations showed that it is imperative to identify possible “layer” thicknesses by utilising deflection basin parameters as well as DCP results to assist during the back calculation procedure. During this process, layers were reviewed to obtain the best basin fit. Only one representative basin was analysed per lane. This provided general trends and typical E-values.

A zone of low stiffness occurs typically within the upper 160 to 250 mm, which is representative of the high BLI values obtained. This analysis indicated that stiffness values reach a maximum at depths between 250 and 550 mm which is deeper than the upper 300 mm, suggested by results obtained with the Heavy Dynamic Cone Penetrometer (HDCP). The HDCP therefore appears to be sensitive to stiffer characteristics of soils over a certain depth.

Table 13: Typical back calculated elastic moduli (in MPa) with depth

DEPTH (m)	VIBRO	4-SIDED	10 kJ	15 kJ	25 kJ
20 PASSES					
0 - 160	60	90	70	130	130
160 - 400	65	85	80	100	85
400 - 550	70	205	170	230	180
550 - 850	70	160	150	180	150
850 - 1150	-	150	145	160	140
1150 - 1650	-	120	140	140	120
1650 - 2150	-	120	120	120	120
DEPTH (m)	**VIBRO**	**4-SIDED ***	**10 kJ**	**15 kJ**	**25 kJ**
40 PASSES					

0 - 160	80	95	60	100	140
160 - 400	70	95	140	185	115
400 - 550	70	180	280	280	250
550 - 850	70	160	260	270	215
850 - 1150	-	120	250	200	210
1150 - 1650	-	110	200	170	180
1650 - 2150	-	110	150	130	160
DEPTH (m)	**VIBRO**	**4-SIDED**	**10 kJ**	**15 kJ**	**25 kJ**
60 PASSES					
0 - 160	80	60	75	110	85
160 - 400	70	210	150	100	205
400 - 550	70	260	250	350	280
550 - 850	70	240	220	340	260
850 - 1150	-	220	200	320	250
1150 - 1650	-	200	150	260	240
1650 - 2150	-	160	130	200	220

* Relatively low values due to material variables

7.4 Practical implications of results

The effect of compaction on pavements required for different traffic levels was calculated using two pavement design procedures. The one, AASHTO, is based on the maximum deflection and the other, the TRH4 method, based on the strength of the subgrade. This exercise showed that the AASHTO design method is more sensitive to variation in compacted subgrade materials than the TRH4 design method.

7.4.1 Asphalt overlay design based on maximum deflection

The 1986 AASHTO pavement design guide proposed a relationship between pavement layer thickness, stiffness and design traffic (in terms of standard axles). This method is commonly used in pavement engineering and details are published elsewhere (AASHTO, 1986).

The average maximum deflection on each section was used to determine the corresponding elastic modulus of the soil support. These values were then used as an input to the AASHTO design procedure. The exercise was simplified by only assuming one strengthening layer of asphalt. The asphalt overlay thickness to accommodate 1 Million Standard Axles (MSA), 3 MSA or 10 MSA was then determined.

Table 14 summarises the cost of the required asphalt overlay to accommodate 3 MSA. It also shows the ranking of the different compaction equipment. The ranking does not differ significantly with an increase in number of passes. It should be noted that the 4-sided section was tested in the compacted lane and not in the wheel tracks. The relative effect is reflected by the hand DCP results presented in **Appendix E**.

Table 14: Equipment ranking based on cost of asphalt layer required for structural design capacity of 3 MSA

20 PASSES		40 PASSES		60 PASSES	
Equipment ranking*	**Cost ($R\times10^3$/km)**	**Equipment ranking***	**Cost ($R\times10^3$/km)**	**Equipment ranking***	**Cost ($R\times10^3$/km)**
15 kJ (Best)	585	15 kJ (Best)	574	15 kJ (Best)	619 #
25 kJ	630	25 kJ	585	25 kJ	619
4-Sided	630	4-Sided #	630	10 kJ	619
10 kJ	664	10 kJ	664	4-Sided	641
Vibrating	754	Vibrating	664	Vibrating	666
No passes	776	No passes	776	No passes	776

* Ranking not applicable where costs are equal, # Anticipated realistic deflections used in calculations

Equipment is compared in **Table 15** based on the minimum cost achieved over 60 passes. The total cost was calculated by adding the cost of rolling. The general trend as depicted in **Table 14** remains unchanged. This ranking is however purely based on cost and the practical significance thereof not taken into consideration.

Table 15: Comparison of maximum effect achieved with different compaction equipment

Equipment	Number of passes	Cost of overlay ($R\times10^3$/km)	Total cost* ($R\times10^3$/km)
15 kJ	40	574	594
25 kJ	40	585	605
4-Sided	20	630	639
10 kJ	60	619	646
Vibrating	40	664	674
In situ material	0	776	777

***** Includes cost of compaction

7.4.2 Structural design based on material classification according to the TRH4 method

Preparation of pavement foundations normally include rip and compact of the in situ subgrade soil to a depth of 150 mm. One or two selected layers will then be added on top of this prepared layer. The California Bearing Ratio (CBR) of the subgrade will determine the type and number of selected layers required. **Table 16** gives the required preparation of the in situ subgrade and required selected layers for different subgrade design CBR's according to the TRH4 (CSRA, 1996) manual which is commonly used in South Africa. This classification was used to classify the material before and after compaction to give an indication of the sensitivity of the improved soil characteristics for application in practical pavement engineering.

California Bearing Ratio (CBR) values were obtained from elastic moduli using the relationship proposed by the Shell pavement design manual (1978). Back calculated moduli to an approximate depth of 500 mm were used in this exercise. Subgrade CBR classes were derived and differently compacted lanes classified **(Table 17)**. Although the classification simplifies the interpretation, borderline cases exist. Derived CBR values are therefor presented to show the extent to which a certain section comply with the class definitions.

Table 16: Definition of subgrade CBR class and required actions for preparation of pavement foundation (CSRA, 1996)

Subgrade CBR class	SG4	SG3	SG2	SG1
Design CBR of subgrade (%)	< 3 *(93% Modified AASHTO density)	3 - 7 (93 % Modified AASHTO density)	7 - 15 (93 % Modified AASHTO density)	➢ 15 (up to CBR of 25: 95 % Mod. AASHTO density)
Add selsected layers: Upper Lower	Not applicable	 150 mm G7# 150 mm G9	 150 mm G7 -	 - -
Treatment of in situ subgrade	Special treatment required	Rip and recompact to 150 mm G10	Rip and recompact to 150 mm G9	Rip and recompact to 150 mm G7
Typical differential cost	Not applicable	R 36 700 / km	R 20 900 / km	R 0 / km

* Field density to be achieved (related to upper limit CBR values), # Details on material classification can be obtained from TRH4 (CSRA, 1996)

Table 17: Classification of compacted in situ materials on Kriel trials according to TRH4 (1996)

Number of passes	**No equipment**	**Vibrating**	**4-Sided**	**10 kJ**	**15 kJ**	**25 kJ**
20	SG3 (5)	SG3 (6)	SG2 (12)	SG2 (10)	SG2 (14)	SG2 (13)
40	SG3 (5)	SG2 (7)	SG2 (10)	SG1 (15)	SG1 (19)	SG1 (16)
60	SG3 (5)	SG2 (7)	SG1 (18)	SG1 (16)	SG1 (17)	SG1 (20)

* Values in brackets: soaked CBR derived from elastic Moduli.

The following conclusions can be drawn from the TRH4 classified materials **(Table 17)**:

- In general no significant improvement takes place after 40 passes. The 4-Sided (40-pass) lane is over conservative due to variability in material conditions, hence it is inferred that a SG1 can be achieved. The results for the 4-sided sections were obtained from testing in the compacted lane and not in the wheel tracks. Comparative results are presented in **Appendix E**.
- No practical difference therefor exists between sections compacted with the 4-Sided (anticipated better results), 10 kJ, 15 kJ and 25 kJ at 40 passes. The 15 kJ however, can be classified with higher confidence.
- The Impact compactors compare well at 60 passes with the 10 kJ closer to the lower limit of the SG1 classification. It is of interest that the decrease in the stiffness of the upper 160 to 250 mm, which occurred on some of these sections after 40 passes, is not reflected by the material classes. This differs from the designs based on maximum deflection which is more sensitive to these variations.

Table 18 summarises the total costs for a bituminous base pavement with a stabilised subbase, and design bearing capacity of 3 Million Standard Axles according to the TRH4 design method. The costs are influenced by the subgrade class achieved with the different compactors, as well as cost of rolling to achieve the desired class. The table shows that due to similar subgrade classes and number of passes to achieve the stated class, the 4-Sided and 10 kJ compactors yield lower cost. This is different from the rating suggested by the design method based on maximum deflection, and confirms that design methods vary in sensitivity to the achieved compaction.

Table 18: Comparison of equipment based on cost of pavement structure with design capacity of 3 MSA

Equipment	Subgrade Class	Number of passes	Cost of subgrade preparation ($R\times10^3$/km)	Total cost* ($R\times10^3$/km)
4- Sided	SG1	40	18,0	278
10 kJ	SG1	40	18,0	278
15 kJ	SG1	40	20,0	280
25 kJ	SG1	40	20,0	280
Vibro	SG2	40	25,9	285
In situ	SG3	0	41,7	301

* Total cost include: Constant cost of pavement structure with design bearing capacity of 3 MSA

7.4.3 Conclusions

The analyses show that the use of impact rolling results in a stiffer subgrade and thus a thinner (and less expensive) pavement to accommodate a certain volume of traffic. The size of saving may vary depending on the pavement design method used. This is illustrated in **Figure 17**. The AASHTO design method is more sensitive to variation in subgrade conditions induced by different compaction equipment, while the TRH4 method shows lower variations in cost. General guidelines for pavement designs can not be developed based on this one trial only.

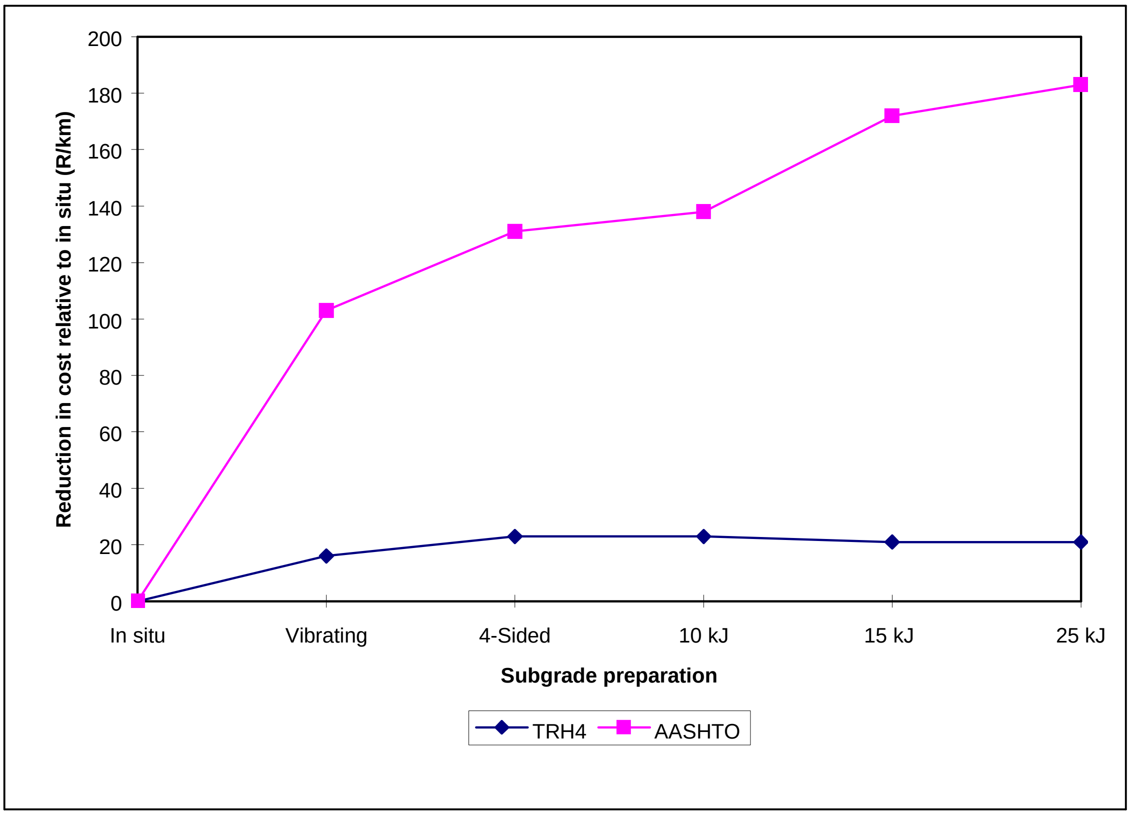


Figure 17: Influence of compaction on cost depends on the design method used

7.5 Applicability of the fwd in testing of compacted in situ material

The main advantage of using a device such as the FWD in testing compacted in situ materials lies in the rapid, economical execution of tests, and the availability of test data on site. Such raw data can be valuable once experience on different material types have been gained in this regard. This investigation showed that the Falling Weight Deflectometer or similar device can be used to compare different compaction equipment and to evaluate the compacted material characteristics. A number of important aspects should however be highlighted.

- **PRACTICAL CONSIDERATIONS**

Deflection measurements on compacted in situ materials can reach values higher than 2 mm at normal test loads, which is not within the deflection range considered to be accurate by the FWD manufacturers. However, a comparison between deflections at 25 kN and 40 kN during this investigation revealed that the variation in data is position (material associated) rather than load associated, and that similar basin shapes existed for the two load cases. Trial tests should be conducted to determine the magnitude of deflections on a particular material. Should the magnitude exceed 2,5 mm or a great variation in data sets with magnitudes higher than 2 mm occur, a decrease in applied energy should be considered. The position of the raise/ lower bar which contains the geophones should be investigated at each test position to avoid loose material or unacceptably uneven surfaces. The latter should also be considered in the preparation of test sections. The facility to adjust the applied load (7kN to 120 kN) creates the opportunity to investigate the stress sensitivity of different soils. This study revealed that a 40 kN load may easily produces a deflection in excess of 2 mm. A load range from 7 to 40 kN is therefore considered sufficient to determine the variation of material response at different energy levels.

- **DATA ANALYSIS**

The main difference between pavement analysis and the analysis of compacted in situ materials is the presence of defined layers in pavement systems. Analysis methods generally applied in pavement engineering were used to process deflection data during this investigation, i.e. the more simplified deflection basin parameter analysis, and more complex back calculation of elastic moduli. It should be noted that these methods were developed for use in pavement engineering which aimed at assessing material characteristics at depths seldom in excess of 1 m. These conventional methods can therefore be used in everyday practice within this depth range, however at greater depths this may become more problematic and results unreliable.

The deflection basin parameter gives a good indication of the trends of different compaction equipment as well as an indication of the relative material characteristics in zones with depth. However, a more detailed analysis should be carried out for determining usable material characteristics. A multi variable analysis approach should be followed for the most reliable characterisation of material properties. This is important in view of the fact that an additional unknown component is introduced in the back calculation of elastic moduli of compacted in situ materials, namely layer thickness. Analysis of deflection basins on these materials showed that postulated layer thicknesses may greatly influence moduli obtained, even though the material was fairly uniform before compaction and gradual changes with depth expected. Deflection basin parameters together with DCP results thus play an important role in the selection of initial thickness of layers for use in the back calculation process. The layers should however be reviewed throughout the process to obtain the best solution.

- **DEPTH OF INFLUENCE**

It was attempted to determine the depth of influence of various compaction equipment by using back calculation of elastic moduli and extrapolation of trends up to 2,6 metres. It was stated that the back calculation of elastic characteristics of materials at such depths is not in line with conventional use of these methods. The determination of depth of influence can not be considered as a routine exercise due to the limitation of models and software available, as well as time and experience involved in such an analysis. For future research purposes, a multi-depth deflectometer (MDD) (with possible alterations) may be employed to determine the depth of influence with greater confidence. The complexity and cost of such a testing programme are however high.

- **PAVEMENT DESIGN**

The significance of improvement of the stiffness of in situ materials by various compaction equipment in pavement design varies according to the pavement design method used. General guidelines for pavement designs can not be developed based on the results obtained from one trial. A data base of information is needed which includes a wide spectrum of material types. The latter can be used to developed a catalogue of designs, expressed in terms of simple measurable input parameters, such as maximum deflection.

- **DENSITY CORRELATION**

No direct correlation between density and compaction, derived from deflection measurements, can be established. Such a relationship would be dependant on CBR values derived from back calculated E-moduli which can then be used with a laboratory established correlation between density and CBR.

- **CONCLUSIONS**

The information presented in this document should be used with caution to classify the performance of compaction equipment on other types of materials. It is recommended that a database for different material types be established for development of more general conclusions towards these performance characteristics.

8. COMPARISON OF TESTS AND TRENDS ESTABLISHED

From the description of the relevant tests in the preceding sections it is evident that certain telling comments can be made and trends established. These are briefly outlined below.

- Characterization of the site, especially in terms of material type and moisture content is important since both can have a profound affect on the results if conditions vary significantly across the site. In this case conditions have been uniform in terms of material type, basic engineering properties, compaction characteristics and moisture content.
- The level survey (settlement) results indicate a gradual increase versus number of passes and indicates a significant reduction of effect beyond 20 passes for the vibro compactor and a lesser reduction beyond 40 passes for the other compactors. Level surveys remain a good qualitative and valuable measure of effect achieved and this is very well illustrated by 150 mm settlement achieved by the vibro compactor as opposed to the 560 mm achieved by the 25 kJ impact compactor after 60 passes. A level survey is important indicator of the optimum number of passes for a specific type of compaction equipment on a certain type of material. For a specific type of material it would be possible to obtain via trials a fairly good qualitative indication of the depth and degree of compaction if correlated with DCP tests taken after sets of say 10 passes.
- DCP soundings have also proved themselves as a valuable means to obtain qualitative and comparative results of consistency. They have the advantage of providing a continuous profile of consistency cheaply and quickly as opposed to in situ densities and plate bearing tests. They have indicated that after 60 passes the improvement achieved by the vibro compactor is limited to the upper 0,5 m of the profile, while the effect of the 25 kJ compactor extends to 2 m depth. Unfortunately DCP tests do not provide a direct measure of soil stiffness and in addition the results are sensitive to changes in soil moisture. Yet they have also provided useful guidance in the estimation of layer thicknesses during the back calculation of stiffnesses from the FWD results.
- In situ density tests have limited value, despite having been in good agreement with the results of specifically the DCP and consolidation tests. In addition they have to be conducted as sand replacement tests if conducted below ground level, since nuclear density tests are not reliable in excavations due to backscatter. Sand replacement tests are time consuming and cumbersome to do in test holes, cause large disturbance of the compacted area and do not provide a direct measure of stiffness, nor do they provide continuous results with depth, unless conducted at close depth intervals (which is costly and impractical)
- Plate bearing tests are simple but time consuming to conduct and provide a static stiffness which is of more value from a compressibility/settlement point of view in the case of building structures. When conducting plate bearing tests, account should be taken of crusting effects during testing at the surface. The plate bearing test reflects the average stiffness of a volume of soil to a depth of about 1,5 times the

plate diameter. Hence to obtain good definition of stiffness with depth a fair number of tests have to be conducted and the test depths judiciously chosen. For pavements the dynamic modulus, as derived from FWD testing, is more appropriate. Yet plate bearing tests measure stiffness which is a more fundamental engineering parameter than density or DCP blow count and hence a test with merit. The fact that it is widely used in the Dynamic Compaction (DC) industry for integrity testing, attests to the appropriateness of this test method.

- Constrained moduli obtained from the consolidation tests are in fair agreement with the plate bearing moduli. Calculations taking into account the void ratio of the virgin soil and the prevailing soil moisture content indicate a degree of saturation of between 38 and 58 %. Although the in situ soil moisture is above compaction optimum it is well below full saturation and did therefore not affect the compaction process adversely. However, if the in situ moisture content was at optimum one would expect somewhat higher stiffnesses and densities achievable over the upper 1 m of the profile.
- FWD tests are appropriate provided that limitations of the device are taken into consideration during testing and interpretation of data at each test site. The maximum deflection is useful in assessing the overall stiffness of the subgrade, while the deflection bowl allows back calculation of elastic moduli. The latter however, requires an indication of the “layering” in terms of stiffness. This can be obtained by inspection of deflection bowl parameters, assessment of errors during the back calculation process and DCP results. Obtaining sensible back calculated moduli at greater depth was not satisfactory using a program employing a 5 layer system and a program employing a 10 layer system had to be resorted to.
- Analysis of FWD data indicated that the maximum effect achieved with impact compactors, in general, is concentrated in a zone 400 – 850 mm. Significant improvement was also discerned from 850 – 1600 mm. Deflection analysis in general showed improvements in deeper zones ranging from 1600 up to 2500 mm. The sections compacted with the 15 kJ and 25 kJ compactors indicated a decrease in stiffness in the upper 160 mm after 40 passes to 60 passes although the stiffness after 60 passes compared favourable with that obtained by other equipment .
- The use of surface deflections in determining relatively deep material characteristics as a routine exercise, is not recommended. More advanced devices, such as the MDD, may be used in future research with regards to depth of influence of various compaction equipment. Deflection measurements are valuable in evaluating pavement capacity and strengthening requirements. This was demonstrated by employing two different design methods. A data base of information should be compiled to develop a catalogue of designs based on deflection parameters.
- Although not many plate bearing tests were done at depth, comparison with the FWD test results indicate that:
  - compared to plate bearing tests done at surface the FWD moduli were about 10 times as high as the virgin plate bearing (secant) moduli and about 0,6 to 1 times the plate bearing rebound moduli;

- compared to plate bearing tests conducted below surface the FWD moduli were 3 – 5 times higher than the virgin plate bearing moduli and about 1,5 times the rebound moduli.

9. CONCLUSIONS AND RECOMMENDATIONS

From the discussions in the preceding sections certain conclusions can be drawn and recommendations made. Also implied are indications of what future work could be undertaken. These are highlighted below.

- Characterization of a site in terms of type of material and engineering properties are essential to be able to correlate effectiveness of compaction with site characteristics but also to ensure validity of results under trial conditions when comparing different types of compaction equipment.
- Level surveys remain an important and simple site exercise, which should be employed in conjunction with other tests to tailor and optimize the compaction process.
- DCP testing by hand or mechanical remains a valuable qualitative assessment tool that compliments level surveying and plate bearing tests and has proved important in producing sensible back calculated moduli from FWD tests. Thought should be given to employ the 2 m hand DCP in conjunction with the mechanical DCP but where the DCP is used in a different way to obtain better definition of near surface changes in soil consistency. This simply implies recording the penetration after every blow, 2 blows or 3 blows (depending on soil consistency). This has particular advantages in the case of the mechanical DCP where the standard procedure is to record the blow count over consecutive 300 mm penetration depths.
- Vertical plate bearing tests, despite its drawbacks of being time consuming and cumbersome to perform below ground level, remains a valuable and important test. It requires no laboratory work and test results can be processed very quickly using available spreadsheets. It is for good reason that it is still primarily and widely used in dynamic compaction to establish the effectiveness of that process. Aspects which should be take into account are:
  - A formalized test and analysis procedure defining amongst others appropriate plate sizes, stress increments, final stress levels, unloading-reloading cycles and definition of moduli.
  - Careful selection of test depths, taking into account crustal effects and the depth range below the plate over which the stiffness is reflected during the test.
  - A means of preparing tests positions to depths of about 1,5 m below ground with the least disturbance, yet in a simple manner. Some form augering process, which allows for trimming of the bedding surface, and extension of the deflection measurement system to be read at surface should be developed.
- Consolidation tests also have value since they provide both a measure of static stiffness and at the same time involve establishment of the in situ density. In other words, if consolidation samples can be extracted in a simple and effective manner, it entirely supersedes the need to do conventional in

situ density tests. The latter is cumbersome and time consuming to do at depth and results in excessive disturbance of the soil to allow work space for the testing staff. It should be possible in the case of sites overlain by finer grained soils to devise a way of extracting samples employing a motorized hand auger. Augering is conducted to the required depth and a "coring bucket" used to core out a sample. Attention will have to be given to the design of the coring bucket and how to prevent the sample from falling out of the bucket when raising it.

- The value of impact compaction to general pavement design has not been fully appreciated in the past. The latter can only materialise through further research. This investigation indicated that FWD tests can be used effectively in the case of pavements to evaluate dynamic stiffness of the subgrade and additional capacity required in view of the magnitude and frequency of the anticipated transit loads. A data base of information on different soil types should be compiled. Deflection data can then be utilised for the development of a catalogue of designs, incorporating the relative effect of subgrades subjected to various degrees of impact compaction. Another potential contribution of impact compaction is a possible decrease in the sensitivity of pavement structures to overloading, due to additional stiffening of the deeper subgrade zones.
- It should be noted that the discussions have largely focussed on the compaction of in situ soils. In the case of fills built in multi-layer fashion, settlement measurement in the sense discussed above can be applied on an individual layer. Yet in the case of high fills settlement may be induced in the lower layers due to the overburden pressure but reflected as settlement of the layer under compaction. As far as DCP tests are concerned they can be conducted on individual layers or several layers. Plate bearing and consolidation tests can be done at the surface of the individual layers, with the exception that consolidation can not be sensibly retrieved if the material is too granular.
- Where not available as yet, spreadsheets should be developed for easy and fast processing of results to ensure that the compaction process can be monitored closely. The above tests should form part of a quality management system which is actively followed and used both to guarantee the outcome of impact compaction projects, as well as marketing Landpac's impact compaction endeavours.

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J H STRYDOM   A HEFER

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REFERENCES

AASHTO. 1986. *Guide for Design of Pavement Structures.* Washington DC.

CSRA. 1996. *Draft TRH4 - Structural Design of Flexible Pavement for Interurban and Rural Roads.* Department of Transport, Pretoria.

Dynatest. 1995. *Dynatest 8000fwd test system: Owners manual.* Dynatest Engineering A/S. Vedbaek, Demark.

Maree, JH and Jooste, F. 1992. *Structural Classification of Pavements through the Use of IDM Deflection Basin Parameters.* PR 91/325. South African Road Board, Sandton.

Shell pavement design manual. 1978. *Asphalt pavements and overlays for road traffic*. Shell International Petroleum Company Limited, London.

APPENDIX J1

Influence of various compaction equipment with number of passes

APPENDIX J2

Relative performance of different compaction equipment

APPENDIX J3

Elastic moduli with depth: Influence of various compaction equipment with number of passes

APPENDIX J4

Comparison between different compaction equipment in terms of elastic moduli with depth

ABSTRACT

The report covers trials conducted at Kriel to assess the effectiveness of impact compaction on a highly compressible/collapsible windblown sand. Characterization of the site in terms of type of materials and their engineering properties was conducted. Five different compactors, viz a vibro compactor, a 4-sided impact compactor, two 5-sided impact compactors of varying energy and a 3-sided impact compactor were used in the trials. Each compacted section was subjected to treatment of 20,40 and 60 passes of the respective compactors.

Measurements during and after compaction involved level surveys; DCP tests (hand and mechanical); in situ densities, consolidation tests; plate bearing tests and Falling Weight Deflectometer (FWD) tests. A section with marker layers was prepared across the 60-pass path of the 3-sided compactor to allow visual evaluation of the influence of compaction afterwards.

The tests have indicated the value of the level surveys to reflect the overall compressibility of the soil and to optimize the number of compaction passes; DCP tests have proved very useful in reflecting the consistency of the profile in a qualitative and comparative manner, as well as determining "layer thicknesses" for back calculation from FWD results; plate bearing and consolidation tests yielded representative static stiffnesses and FWD tests have enabled the back calculation of dynamic moduli.

Further work involves performing DCP tests in such a manner to get better definition of consistency changes; ways to perform plate bearing tests at depths of up to 1,5 m in a more streamlined way and to extract consolidation samples to the above mentioned depths via auger drilling and bucket coring in finer grained soils; as well as performing FWD tests on other soils. In addition, the above mentioned tests should be incorporated in a quality management system.

Keywords

Kriel
Impact impaction
Landpac
Compressible
Trials
Stiffness
FWD tests
Level surveys
DCP tests
Plate bearing tests
Consolidation tests