

IMPACT COMPACTION OF SUBGRADES - EXPERIENCE ON THE TRANS-KALAHARI HIGHWAY INCLUDING CONTINUOUS IMPACT RESPONSE (CIR) AS A METHOD OF QUALITY CONTROL

I. Jumo¹ and J. Geldenhuys²

¹Stewart Scott International, Gaborone, Botswana.

²Stewart Scott International, Pretoria, Gauteng, South Africa.

ABSTRACT

Traditionally, compaction equipment for road building has consisted of static weight, kneading action or vibratory rolling of the soil to achieve the specified improvement in bearing strength. In some cases, however, such as the compaction of sand subgrades in arid regions, it was found that compaction of the in situ subgrades was problematic due to the inadequate energy output of traditional compaction equipment to compact the relatively dry sands in depth, or to break the inter-particle bonds of collapsible sands. This has led to the development of the Impact Compactors, namely the 25kJ three-sided Impact Compactor, based on the principle of high energy, high amplitude and low frequency. Compaction of subgrades is usually controlled by a density and method specification, e.g. 90% of mod AASHTO density and construction in layers of typically 150mm, or the number of roller passes. Impact compaction of subgrades is usually controlled by doing a trial section at the beginning of the project, and then specifying the number of passes for the entire length of the construction. The problem with this method is that the material properties, moisture conditions and subgrade support may vary resulting in some road sections being over compacted while others are inadequately compacted.

This paper describes the Impact Compaction technique and control method that was successfully implemented during the reconstruction of part of the Trans Kalahari Highway, 80km between Jwaneng and Sekoma, in the Republic of Botswana. The control method is based on the principle of measurement of the deceleration of the Compactor on impact with the subgrade. The measurement of these decelerations over time, is called the Continuous Impact Response (CIR). The results are then statistically analysed and the optimum number of passes for each compacted road section calculated. The CIR employs a GPS receiver to locate the position of the impact roller. Each of the peak decelerations is recorded relative to its position on site as determined by an integrated GPS receiver. It is thus possible to produce a "map" of the deceleration values over the entire site, which can be used, together with colour coding, to determine areas that would not have received adequate compaction.

It was found on the Jwaneng-Sekoma project that the control of the number of passes of the impact compactor after the analysis of the deceleration of the impacts works well, and is an excellent method to optimize cost vs. subgrade improvement.

The paper also looks at the results of the FWD back calculated stiffnesses, DCP calculated stiffnesses and other strength parameters, and compares them with the values that were assumed during the design of the road.

1. INTRODUCTION

The 80km road section between the diamond-mining town of Jwaneng and the village of Sekoma forms part of the Trans Kalahari Highway in the Republic of Botswana. The road was constructed in the late eighties and already showed signs of distress about 5 years after

construction. Originally the road was designed for a 10-year design life. During the rehabilitation investigation, it became apparent that the existing bitumen-surfaced road failed *inter alia* because of an unconsolidated sand subgrade. Impact Compaction, together with the Landpac CIR (Continuous Impact Response) system, was specified during the rehabilitation design to resolve the problems associated with a weak subgrade. It was envisaged that Impact Compaction will improve the life of the road at a comparatively cheaper cost than doing it the conventional way by excavating the sands and bringing it back in compacted layers of 150-300mm. This paper demonstrates that the pavement life after impact compaction, in terms of MESA (Million Equivalent Standard Axles), exceeds predictions of the as designed pavement life. The life after Impact Compaction was predicted after FWD measurements were done on top of the newly constructed road base. The results were also confirmed by utilizing the Rubicon1 pavement response model.

2. THE THEORY OF IMPACT COMPACTION

Impact compaction can be described as the process whereby high levels of energy are imparted into the ground, to achieve higher degrees of compaction at greater depth than would be possible with traditional compaction equipment. (The depth of influence of these machines may exceed 5m in certain materials). The energy transfer is obtained by using heavy masses with out-of-round profiles that are drawn across the area to be compacted with special dedicated power units at speeds greater than twice the speed of conventional compaction equipment.

Unlike conventional compaction, large parcels of energy are imparted at high amplitude and a low frequency (up to two blows per second). The largest of impact compactors imparts 25kJ of energy to the ground per blow.

Compaction using the high parcels of energy imparted by the impact compactors ensures that the maximum amount of void reduction occurs, if sufficient number of passes are applied. This increases the density and stiffness of the material. Impact compaction is thus a continuous process of dynamic loading similar to that found in dynamic compaction of structural foundations. The rolling mechanism is shown in Figure 1 below.

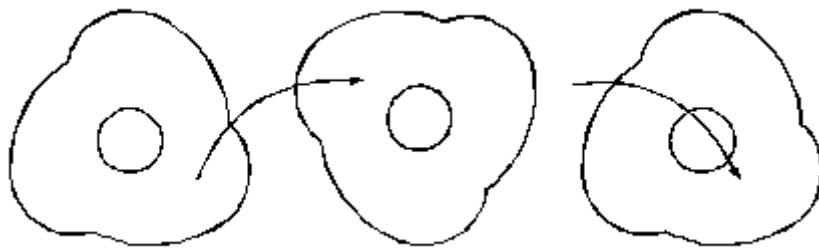


Figure 1. Rolling mechanisms of a 3-sided 25kJ impact roller.

According to Bousinesz the depth of influence is a function of the contact pressure and the geometry of the roller. The huge contact areas of the drum allows for improvement down to depths of 5.0m. The contact stress measured during impact range from 300kPa to 1.2Mpa depending on the soil type and moisture conditions. Decelerations during impact may measure up to 20g's, which equates to a dynamic load of 250 tonnes or an equivalent contact pressure of 1.2Mpa.

Traditional approaches to compaction specification and control have generally evolved to suit the capabilities of conventional compaction plant. For conventional plant, relatively thin-lift layered construction (typically 150 - 300mm) and a related stepped-density profile is normally specified. The enhanced compaction ability of impact rollers on deep unconsolidated sub-grades renders these traditional methods of specification and control inappropriate.

Specifying the relative density in terms of the Mod AASHTO test, controls current impact compaction. Another method is also to conduct a trial section of various passes with various tests at the start of construction. These tests are then related to the usual density specification. Based on the results of the tests, the Engineer specifies the number of passes for the whole project. At best, the trial section might be repeated midway through the contract but in most cases the number of passes is fixed at the start of the contract. This is accepted as the norm because of time and cost constraints during construction.

Clearly, the current method of establishing the optimum number of passes by trial section is not ideal, and with the development of the Continuous Impact Response (CIR) system for impact compaction, the opportunity presented itself to research the value of CIR as a method to control impact compaction. The investigation is based on and limited to an 80km road rehabilitation project from Jwaneng to Sekoma on the Trans Kalahari Highway in the Republic of Botswana. Stewart Scott was appointed by the Roads Department of Botswana to do the rehabilitation design and construction monitoring on the contract and the impact compaction was done by a firm called Landpac.

3. CONTINUOUS IMPACT RESPONSE

Landpac has developed and patented a Continuous Impact Response (CIR) measuring system with impact compaction equipment. With the CIR measuring system it is possible to use the impact compactor as a proof roller and simultaneously measure the subgrade's response to the dynamic loading of the impact compactor. The CIR system includes an accelerometer and a GPS, which records the deceleration and position of every impact of the roller (see figure 2.1).

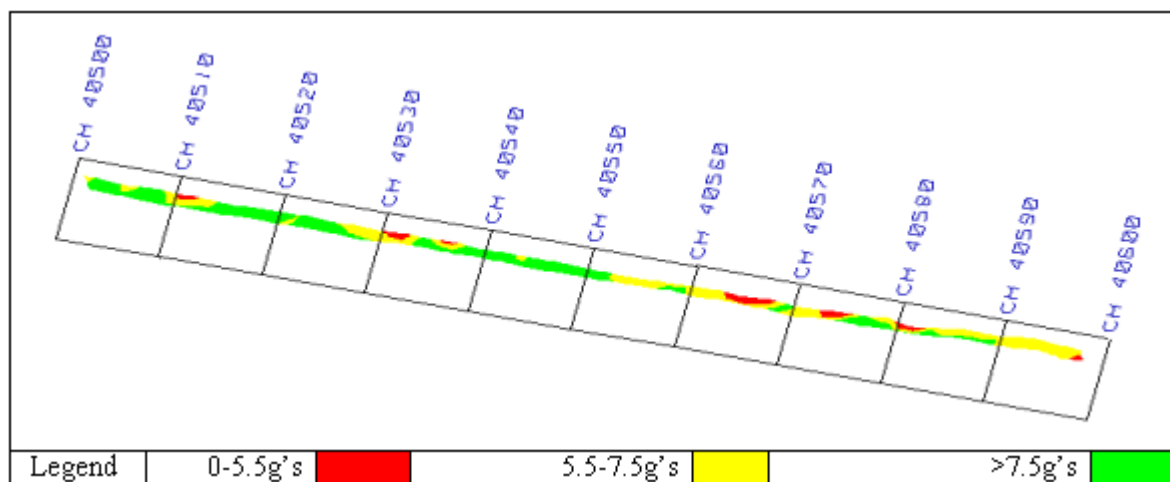


Figure 2.1. Map of decelerations during the trial section.

3.1 Response During Impact

The Continuous Impact Response system employs an accelerometer, which is fitted to the axle between the two masses. Deceleration is measured on a continuous basis, and peak deceleration with each impact is recorded. While the material is still in a loose state, most of the initial compaction energy will lead to plastic deformation of the soil. At this stage, the soil has a soft response to the load applied and relatively low decelerations of the compaction mass are measured. Decelerations in the order of three to five g's are normally measured. Further compaction of the subgrade results in densification of the material. Since the stiffness of the subgrade is a function of the material's density, a more rigid subgrade response is experienced as the material moves towards the elastic state. Higher decelerations (in the order of 6-10 g's) will therefore be measured as the soil reaches the elastic stage. Typical before and after deceleration graphs are shown in Figure 2 (Wilken, 2001).

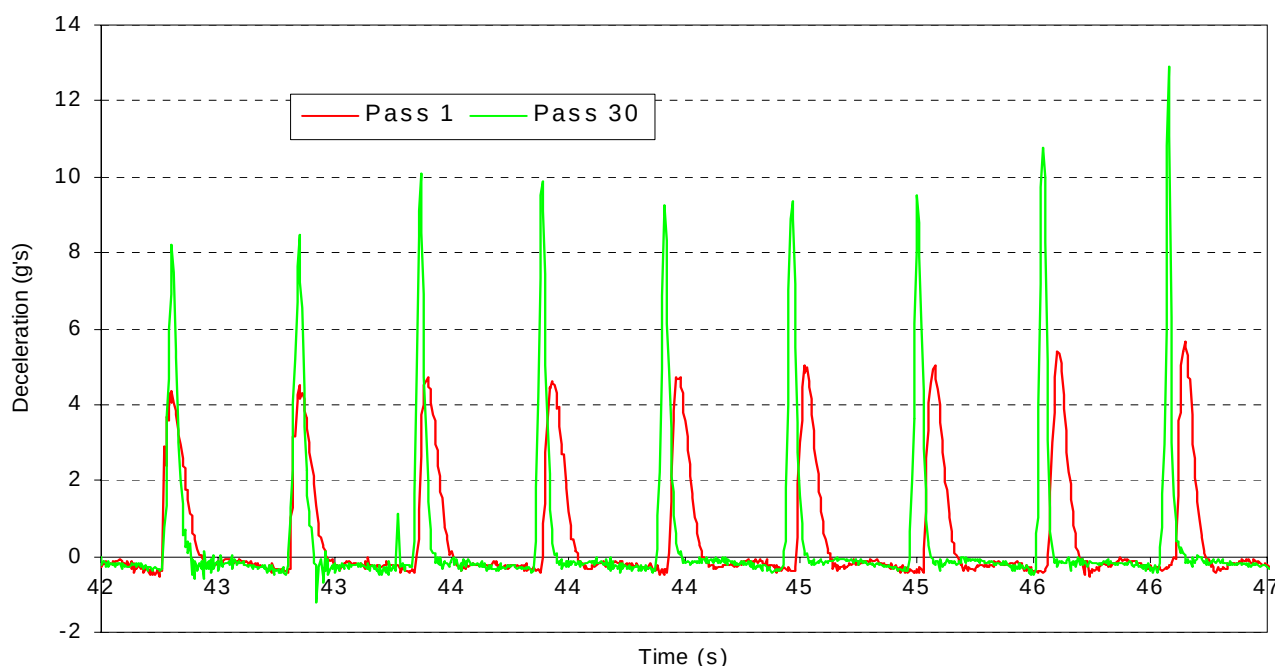


Figure 2. First and final pass decelerations comparison of a 25kJ Impact Compactor.

3.2 Use of GPS

The Global Positioning System (GPS) is a satellite-based navigation system made up of a network of satellites placed into orbit by the U. S. Department of Defense. The CIR employs a GPS receiver to locate the position of the impact roller. Each of the peak decelerations is recorded relative to its position on site as determined by an integrated GPS receiver. It is thus possible to produce a “map” of the deceleration values over the entire site.

4. INVESTIGATION AND DESIGN

The investigation into the premature failure of the road revealed that the road failed as a result of, among other things, the unconsolidated sand subgrade. Although collapse potential tests showed that the sand is not collapsible, it was found that the subgrade sand had not been adequately compacted and that the deeper subgrade might contain sections or lenses of loose to very loose Kalahari sand.

4.1 Existing and Proposed Pavement Design

The pavement structure of the existing road that had to be rehabilitated consisted of the following:

Surfacing	19/9,5mm double deal
150mm calcrete base	minimum CBR of 80% and compacted to 98% of mod AASHTO density
150mm sand subbase	minimum density of 95% of mod AASHTO density
150mm selected sand	minimum density of 93% of mod AASHTO density subgrade
Subgrade sand and fill	minimum density of 90% of mod AASHTO density

The rehabilitation design analysis using a response system (Rubicon 1) showed that this is a light pavement structure and that the existing subgrade provides very little support to the upper structural layers. It was therefore necessary that the subgrade be strengthened to provide adequate support, together with the addition of new pavement layers. The new pavement was designed for 3-4 million equivalent single axle loads (E80s).

The new pavement design is as follows:

Surfacing	19/9.5 double seal
150mm G2 rushed stone base (new)	minimum density of 86% of apparent relative density or 102% of mod AASHTO density
200mm calcrete subbase (new)	minimum CBR of 45% at 95% mod AASHTO density and minimum density of 98% of mod AASHTO density
150mm selected (old calcrete base – ripped and re-compacted after being brought to level)	minimum CBR of 15 at 93% mod AASHTO density compacted to 95% of mod AASHTO density
Subgrade sand	compacted to 95% of mod AASHTO density 600mm thick.

Subgrade sand samples were tested with a vibrating table and the K-mould to determine their maximum constructible density and their strength properties.

The following were found:

- Compaction to a maximum of 98% mod AASHTO density is practically possible.
- The sand is stress softening, i.e. the subgrade stiffness decreases with an increase of the vertical stress on top of it.
- Compressive stress must be less than 35 kPa for a 20mm rut depth at 6 million E80s.

4.2 Deep Compaction Field Investigation

The effect of deep compaction by impact rolling was investigated on site during the design process, before bringing the impact roller at high cost to the site. Landpac has developed a Mini Dynamic Compactor (MDC), shown in Figure 6, which consists of a truck-mounted 2-ton mass that is hydraulically lifted and dropped from a height of 1m onto a steel plate that is positioned on the ground to achieve deep compaction. The MDC also simulates the effects (e.g. settlement) of an impact roller on a pavement structure.



Figure 6. Landpac mini dynamic compactor on the Jwaneng – Sekoma site.

During the rehabilitation design stage of the project, the MDC was used at eight locations along the road to determine the following:

- Quantification of the expected typical improvement that would be achieved with the 25kJ impact roller
- An early indication of the optimum number of 25kJ impact roller number of passes that would be necessary during construction
- An indication of the expected settlement for geometric design and calculation of quantities and costs

The improvement in the shear strength was measured with a Dynamic Cone Penetrometer (DCP) at intervals of 10 MDC drops. The DCP consists of a steel rod that is hammered into the pavement with a specific amount of energy that is applied with each drop of the hammer (8kg hammer and a 575mm drop height). The rate of penetration through the various pavement layers is measured and used to define the shear strength of each layer in mm/blow. DSN800 value is defined as the number of drops from the surface to 800mm below the pavement. DSN300-1500 would then be the number of drops from 300mm to 1500mm below the surface.

The improvement between 30 to 40 drops and 50 to 60 drops of the MDC was small and it was therefore proposed to do, as a minimum 20 passes with the 25kJ impact roller (40 MDC drops = 20 x 25kJ impact roller passes).

Using the data from the MDC it was estimated that the settlements as presented in Table 1 would occur along the road with the proposed 20 passes of the 25kJ impact roller.

Table 1. Estimated settlements for 20 passes with the 25kJ impact roller.

Chainage (km)	2,5	10	18	27	40	48	62	79
Settlement (mm)	50	130	50	80	120	160	80	80

When the areas of higher settlement presented in Table 1 were compared with the uniform sections in a poor condition, as identified with the Falling Weight Deflectometer (FWD) during the rehabilitation investigation, there is a close correlation. Representative settlements were then identified for each uniform section and taken into account in the design of the final rehabilitated road level. From the real constructed settlements it was found that the settlements had been predicted quite accurately, although some sections received more than 20 passes. It should also be borne in mind that the settlement is moisture sensitive and that the moisture regime of the subgrade during the investigation might not be the same as when it is eventually impact rolled during the construction phase.

4.3 Specification

A typical specification for impact compaction should address the following aspects:

- The type of impact roller, dynamic energy and minimum speed
- Definition of a pass
- Trial section (length, width, tests and measurements of passes with testing, maximum number of passes)
- The surface finish and watering requirements during compaction considering the utilization of the subgrade's in situ water content
- The minimum number of passes and increments of passes after that

- The use of CIR as a method of compaction control, the increments of passes that should be measured, the process of submission and approval after each increment and the maximum number of passes that should be performed and measured when a road segment is ready for compaction
- The process after impact compaction. Roadbed preparation must still be done to compact the loose top layer after impact rolling.

5. CONSTRUCTION

A new approach was followed during the construction phase of the project in respect of the control of impact compaction. Standard practice is to do a trial section and then based on tests done at the trial to decide on a fixed number of passes for the entire project. However, during the trial done on this project, the tests on the trial section were used to establish the minimum number of passes and then, during construction, to vary the number of passes according to the optimum found with the 10th percentile value of the CIR decelerations.

5.1 Trial Section

At the start of construction, an impact compaction trial section was done on a 100m section of the road at km40.5. The trial section was done at one of the poorer identified uniform sections of the road.

The trial was done to:

- confirm the compaction process.
- develop a preliminary correlation between settlement, density and DCP sheer strength for compaction control.
- determine the minimum and required number of passes according to settlement, density and DCP sheer strength.

The first picture in Figure 7 shows the section during the compaction of the first 5 passes and the second picture shows the same section after 30 passes.



Figure 7. Trial section during the first 5 passes and after 30 passes.

5.1.1 Settlement

The settlement was measured by taking levels at 1m intervals over 20m of the test section. Levels were taken after every 5 passes of the roller. The settlement curve is shown in Figure 8. The graph shows a high rate of settlement (7mm/pass) up to 5 passes, and a fairly constant rate of settlement (3mm/pass) between 5 and 25 passes, after which the settlement begins to flatten out to (1.8mm/pass).

The shift in the curve between 25 and 30 passes is a result of the total disintegration of the seal surfacing between 25 and 30 passes. Before 25 passes, the seal forms loose and warped shingles. This effect, which gives a lower settlement reading, is included in the measurements between 0 and 25 passes.

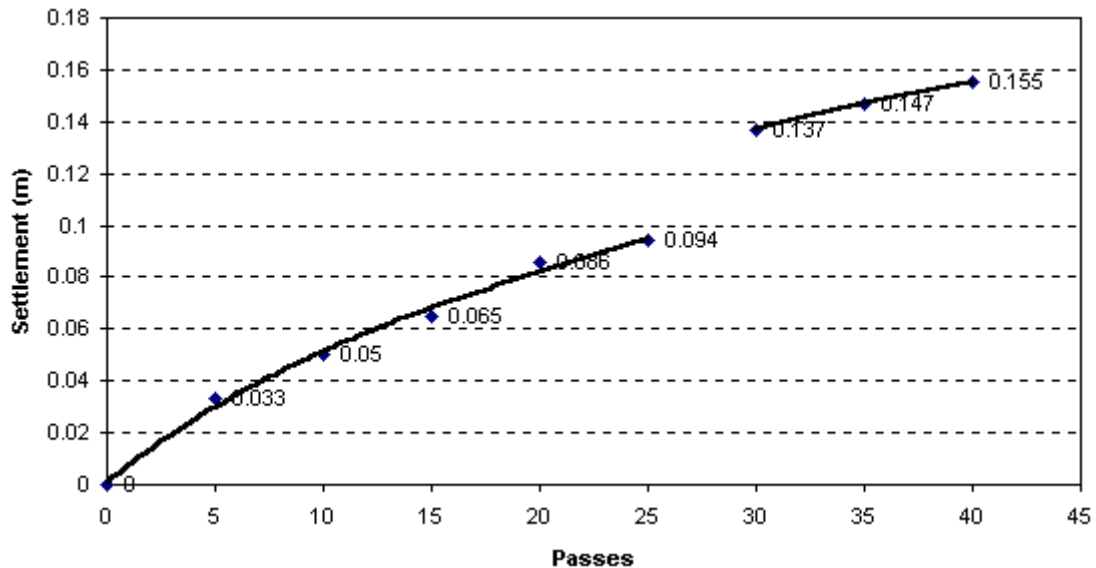


Figure 8. Average settlement measured at the trial section.

5.1.2 Density and continuous impact response (CIR)

Four density tests were done after every 10 passes. The tests were done at 300-600mm and 600-900mm below the surface. Initially, sand replacement densities were done at depth to confirm the results obtained with the nuclear gauge. The necessary adjustments were made to compensate for determining densities with the nuclear gauge at the various depths.

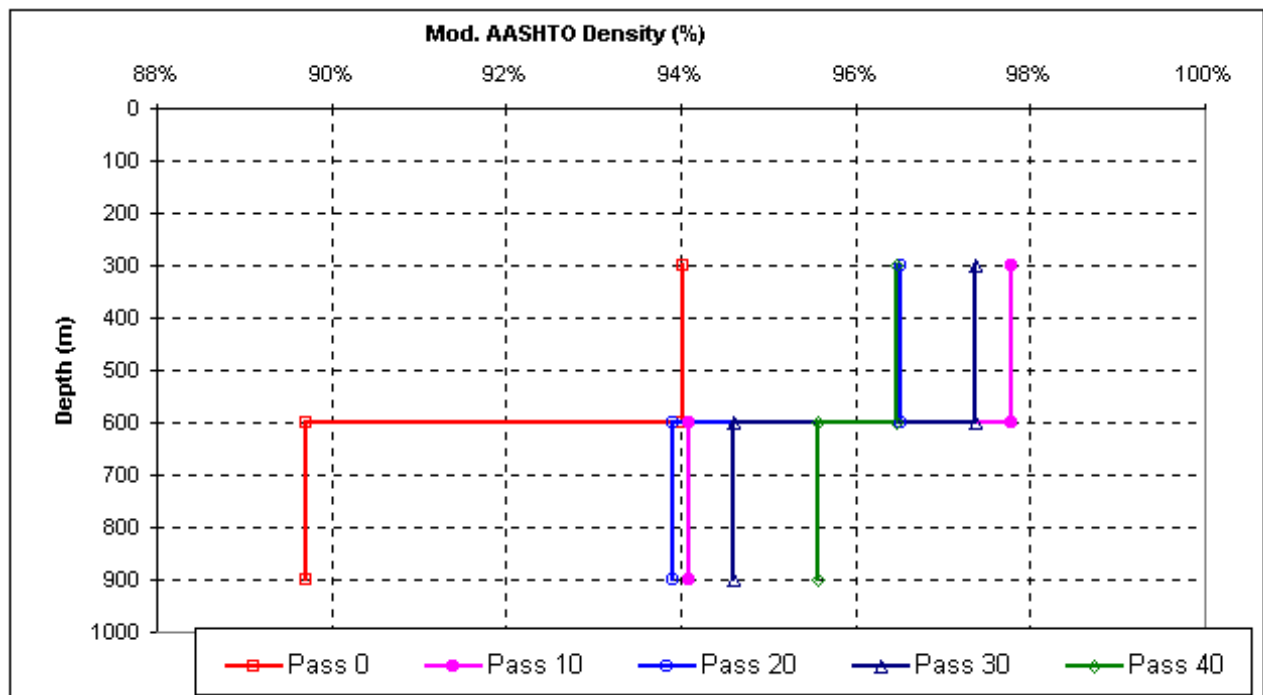


Figure 9. Density results achieved at the trial section.

The trial section results showed that it is possible to compact the 300-600mm subgrade sand to between 96-98% mod AASHTO density, and the 600-900mm subgrade sand to between 94-96% modified AASHTO density with a minimum of 10 passes of the 25kJ impact roller. A density of 95% modified AASHTO density was prescribed for 0-600mm of the sand subgrade.

The CIR was measured every 5 passes up to 40 passes. The 10th percentile g's of all the decelerations were calculated at each 5 pass interval respectively and have been shown in Figure 10. The graph shows that the increase in 10th percentile decelerations from 1 pass to 20 passes is about 0.18g/pass. However, after 20 passes this increase diminishes to only about 0.03g/pass. Therefore, the effectiveness per pass decreases about 6 times after 20 passes.

The relationships between the average measured modified AASHTO density at 300 – 900mm, the measured CIR, and the number of passes have also been shown in Figure 10. If the two curves are plotted against one other the following relationship is found:

$$\% \text{ mod AASHTO density at 300-900mm} = 0.8417g + 89.558$$

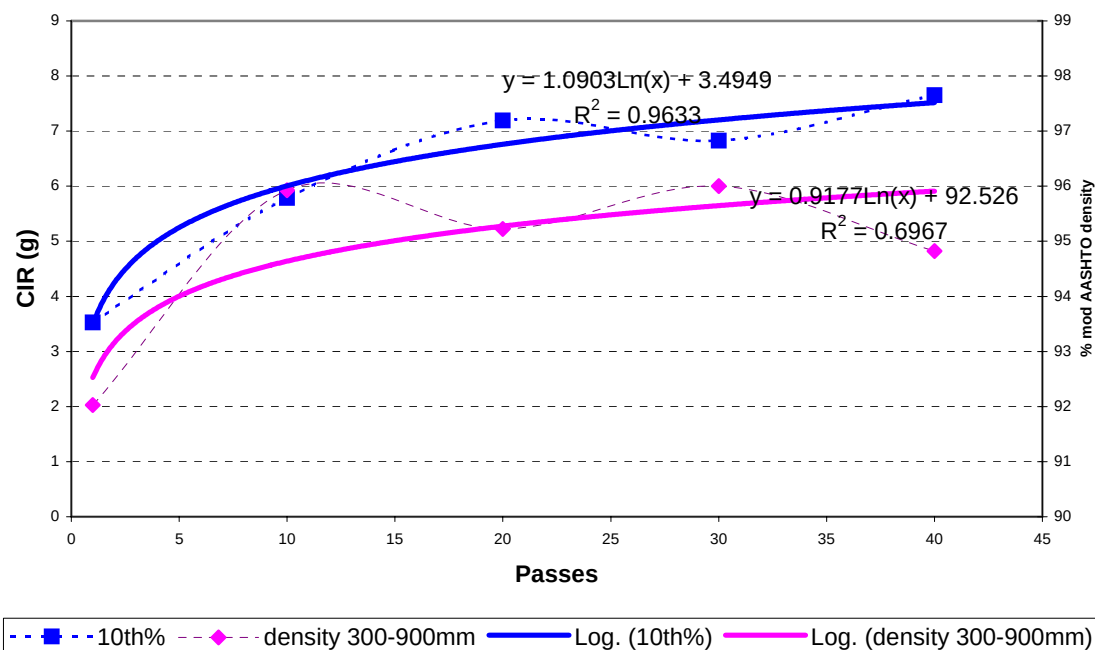


Figure 10. Relationship between measured layer density and number of passes.

This relationship between density and the 10th percentile deceleration (g) is only valid for the test section and has not been verified for the rest of the road. However, this gives an indication of what can be expected in the subgrade when it has been compacted by the 25kJ impact roller. The lower boundary value of the formula also confirms what was found at the test section. The formula shows that with a first pass 10th percentile deceleration measurement of about 3.5g the subgrade density is about 92.5% mod AASHTO density. This is close to the average measured in situ density of 92%.

5.1.3 Dynamic Cone Penetrometer (DCP)

Dynamic Cone Penetrometer (DCP) tests were done at the beginning of the trial and thereafter in 10 pass increments. The increases in shear strength, based on DCP measurements that extend to 2000mm below the surface, have been indicated in Figure 11. From the graphs it can be seen that the increase in shear strength between 800mm and 1900mm below the surface is very high during the first 10 passes. There is no marked increase in shear strength after 20 passes.

The DSN300-1500 was calculated from the 40 DCP tests that were done at the trial section. The DSN300-1500 is defined as the number of blows it takes to penetrate the DCP from 300 – 1500mm below surface. The DSN300 was not taken into account, as the impact roller disturbs

the top 300mm of the pavement and this cannot be compared to the original constructed 300mm of the road layer works. This disturbance can be clearly seen on the DCP curves (Figure 11).

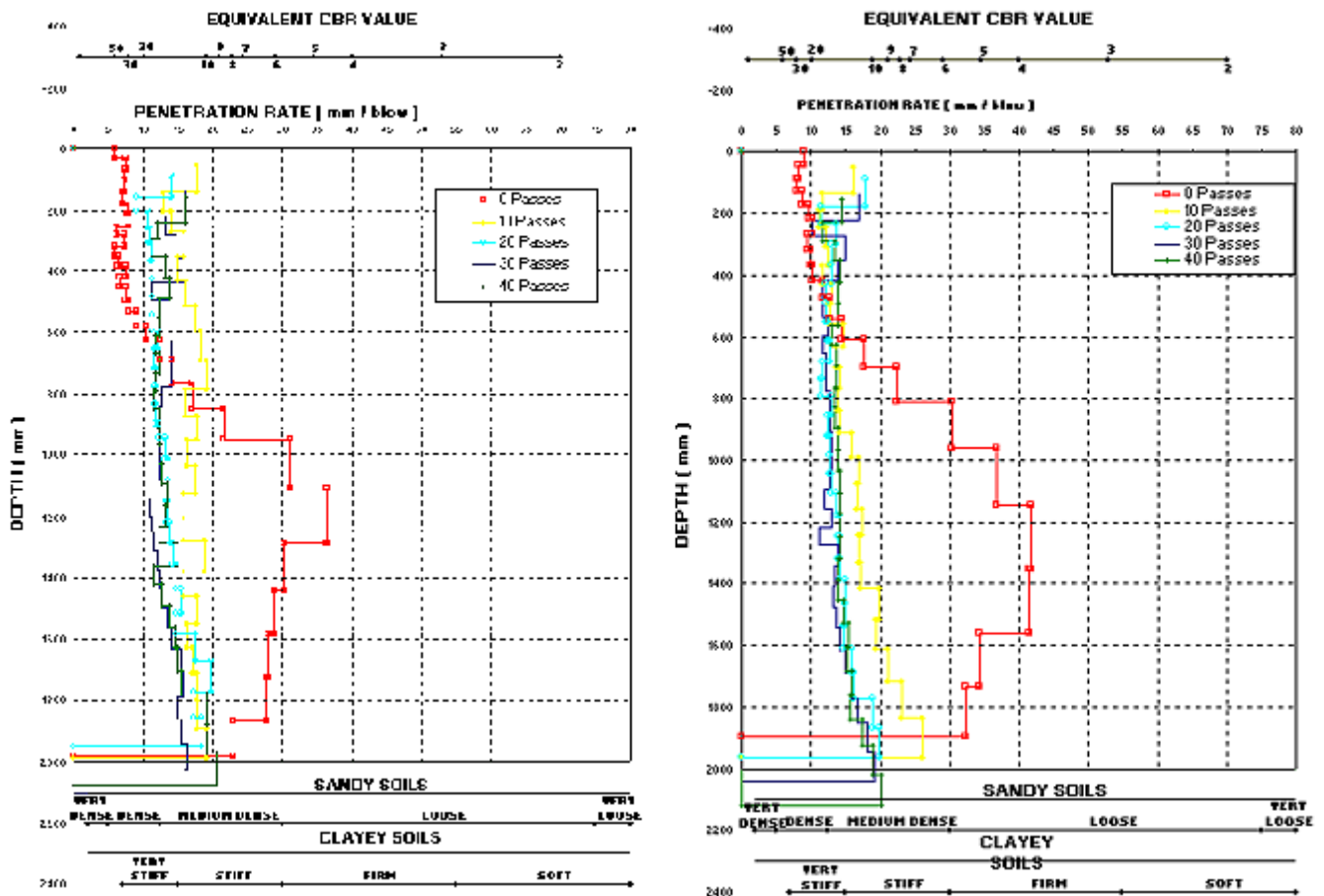


Figure 11. Typical DCP layer strength diagrams measured at the trial section.

The DSN300-1500 values, as measured during the trial section, have been presented in Figure 12. A polynomial graph was fitted through the data and it can be seen that it closely follows the average of the DSN300-1500 values. The graph shows that according to the average DSN values the optimum number of passes in terms of the measured shear strength is in the order of 30 passes. However, if the minimum DSN values are compared to each other it is evident that the effectiveness of compaction decreases markedly after 20 passes. This is confirmed with the results of two typical layer strength diagrams presented in Figure 11.

The standard deviations of the DSN300-1500 per 10 passes have also been plotted on the same graph. A marked decrease of the standard deviation is evident around 20-30 passes. This may be owing to the fact that a more homogeneous subgrade is obtained around 20 passes. However, with more passes, sheer cracks form deeper down in the subgrade, increasing the standard deviation of the DSN300-1500 values which eventually leads to a decrease in strength after 30 passes, as shown in the graph. It seems therefore that the sand subgrade can be over-compacted. The DCP results of the trial section show that this effect might become significant at around 30 passes.

Based on the test results of the trial section, it was decided to give all sections a minimum compactive effort of 20 passes. Decelerations from CIR would then be analysed to establish the optimum number of passes for each road section.

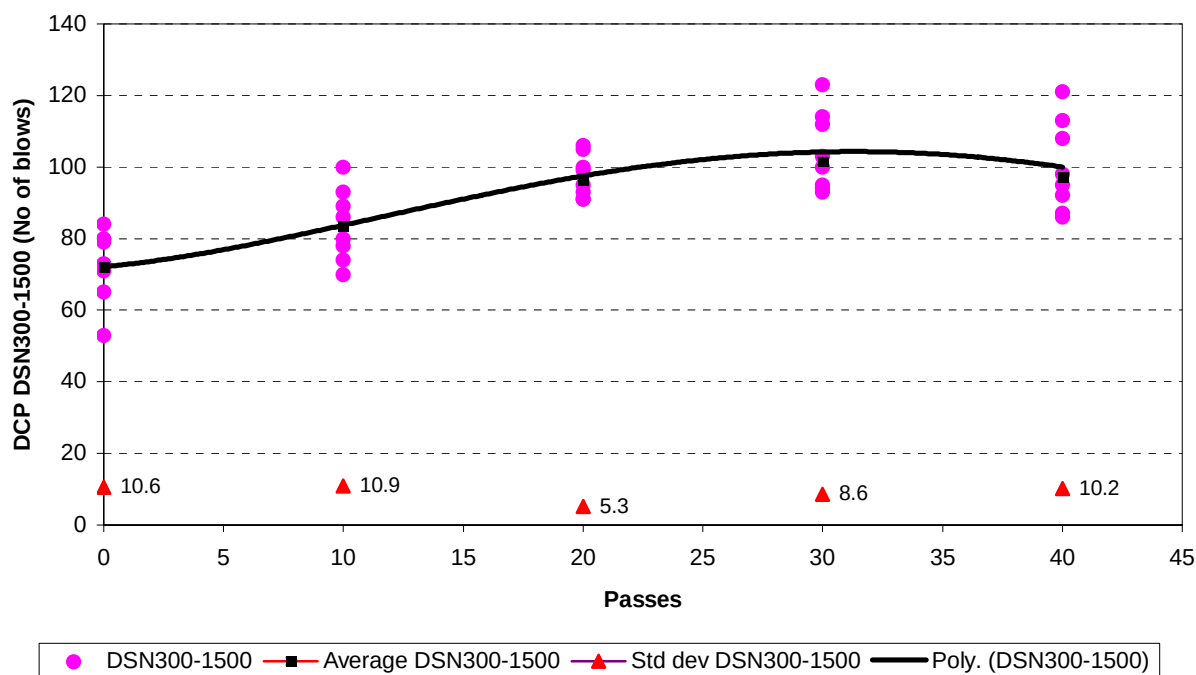


Figure 12. DCP DSN300-1500mm measured at the trial section.

5.2 Compaction Control

The road was made available to the Contractor in 7-km segments and these segments were compacted in 700m sections. The first section in a segment was usually compacted to 30-35 passes to establish a CIR deceleration trend for that 7km segment. This 5-pass increment (5, 10, 15, 20, 25, etc.) data was then analyzed and a decision made as to how many passes all the sections in the segment should receive in general. After each section had been compacted the data was also analyzed for the specific section, and a decision made as to whether further passes were required.

5.3 CIR Data Analysis Approach

The following variables of the decelerations of a section have been calculated for the specified number of passes in 5-pass intervals:

- 10th percentile
- average
- 90th percentile

A graph showing these variables is shown in Figure 13. The most important variable that must be analysed and the results assessed, is the 10th percentile value of the decelerations of the respective number of passes. If this value stabilizes, it means that 10% of the decelerations are lower than this value and, no matter how many passes are added, this value will stay the same. This, in turn, means that for this specific type of impact roller with its specific energy, 10% of the length of the road section will not improve with further rolling. Further improvement of the 10% can only be done by using a bigger roller with a greater energy output.

Secondly, the 10th and 90th percentile curves are analysed together. If the difference between the two lines decreases, this is an indication that the subgrade has been compacted to a more uniform layer. This difference shows a similar trend to the standard deviation curve of the DCP analysis as shown in Figure 11.

The average of the decelerations also gives an indication of the success of the compaction effort, although it may conceal the true state of the compacted section. It may for instance show an increasing trend, but this trend may be created by the 90% values that increase and not the lower 10% values, and nothing would have been gained by the compaction.

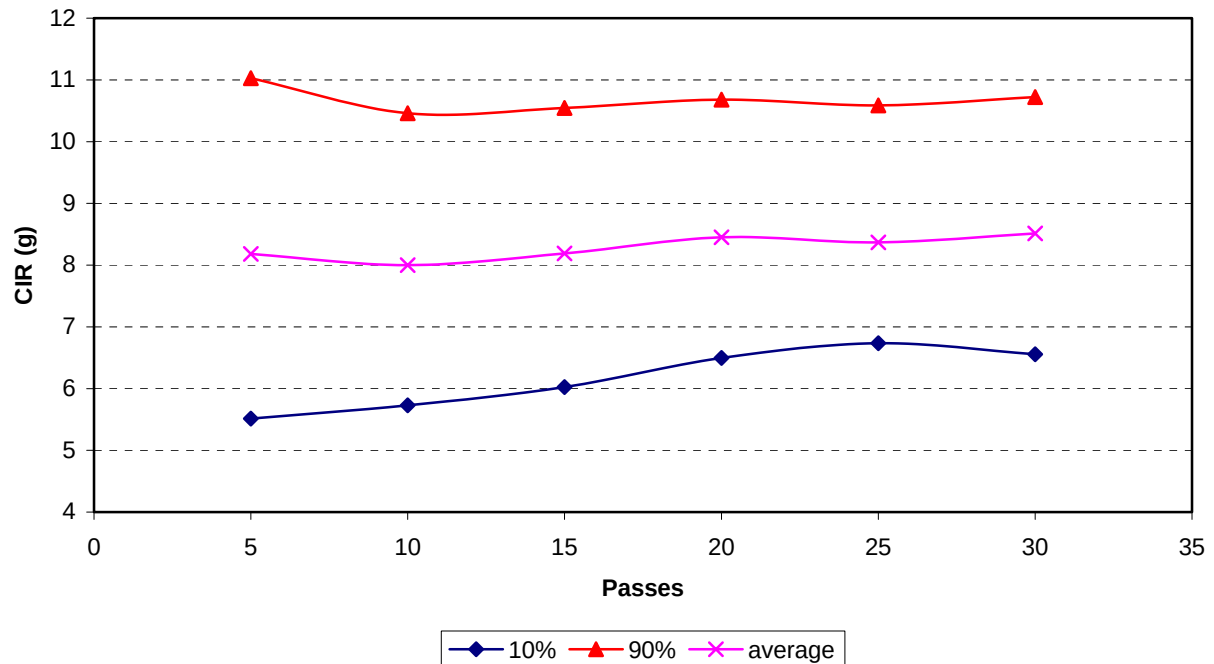


Figure 13. Deceleration variables based on averaged data of 24 x 700m compacted sections.

5.4 CIR Data Analysis

The values given in Figure 13 are based on averaged data from 24 sections of 700m each. The graph is therefore based on data obtained from the compaction of 16.8km of the road.

From Figure 13 it can be seen that the 90th percentile value of the decelerations drops slightly (about 0.5g) after 10 passes. These high 90th percentile decelerations between 5-10 passes are recorded because of the high rigidity of the existing pavement structure of the road. However, this effect is negligible after 10 passes with the 25kJ impact roller.

The average deceleration curve shows the same effect over the first 10 passes, after which the decelerations marginally increase from 8g at 10 passes to 8.5g at 30 passes. This curve does not give a good indication of when to stop compaction or when the optimum compaction has been attained.

The 10th percentile deceleration curve gives a good indication of the increased resistance or densification of the subgrade and the optimum point of compaction. On average on this project, the optimum was 25 passes. If this is compared to the trial section results of the settlement, density, CIR and DCP measurements, the following points are of interest:

The settlement curve shows a smooth transition from a high settlement rate per pass at the start of compaction to a lower rate after 45 passes. No optimum point is evident and a direct comparison with deceleration measurements is therefore not possible. The reason for this is that the settlements at higher passes are influenced by the horizontal displacement of the sand because of inadequate side restraint.

The density relationship, at 300-900 mm, shows that the maximum compaction is reached after 10 passes with another peak after 30 passes. This effect might be due to the fact that layers deeper than 900mm are only compacted optimally between 10 to 30 passes. This will then correspond to the 10th percentile deceleration optimum of 25 passes found on average on the road itself.

- The CIR results measured at the trial section show that the effectiveness of compaction of the in situ subgrade decreases markedly after 20 passes.
- The average DCP results show that the subgrade at the trial section is optimally compacted after 30 passes. If the minimum values are used it shows that the effectiveness of compactions decreases markedly after 20 passes. Again, this corresponds well with what has been found with the CIR deceleration results along the rest of the road.

5.5 Density Data Analysis

Density tests were done at 300mm into the in-situ sands after impact compaction. The purpose of the density tests was to ensure that the in-situ sand was compacted to the specified 95% of Modified AASHTO density. The results below show densities done between km 45 and 46.

	45+	000	100	200	300	400	500	700	800	900
In-Situ Dry Density(kg/m3)		1819	1824	1816	1846	1827	1780	1814	1754	1762
In-Situ Moisture Content(%)		0.8	0.3	1.0	1.1	0.7	1.8	1.8	1.7	0.7
Mod. AASHTO Max. Dry Density (kg/m3)				1816						1810
Optimum Moisture Content (%)				7.2						10.1
In-Situ Relative Compaction (%)		100.2	100.4	100.0	101.7	100.6	98.0	100.2	96.9	97.4

It was thus possible to obtain the specified subgrade density of 95% of Mod. AASHTO density after impact compaction.

5.6 DCP Data Analysis

Before each section was impact rolled, DCP tests were carried out just below the calcrete base i.e. confined to the sand subgrade, at 200m intervals. The DCP tests were carried out again on the exact position after the required number of passes had been carried out.

The DSN800 values were plotted and are shown as 10-point moving averages in Figure 14. The DSN800 value is presented in blows and the number of blows it takes for the DCP to reach a depth of 800mm. Figure 14 shows that no tests were done in some areas such as from km7 to km18 (after impact rolling) and km39 to 46, km52 to km60 and km67 to km72 for both cases.

Figure 14 shows that there was a definite improvement in the DSN800 values as measured with the DCP before and after impact rolling. The figure also shows that this improvement is the greatest in areas where the road before impact compaction was in the worst condition, i.e. from km 10 to km38.

The graph also shows the high strength before and after impact compaction of the road section from km72 to km81. This section was impact rolled when it was originally built. It also has a few layers below the calcrete base layer consisting of sand mixed with calcrete. This added strength is shown by the particularly high DSN800 values found on this section.

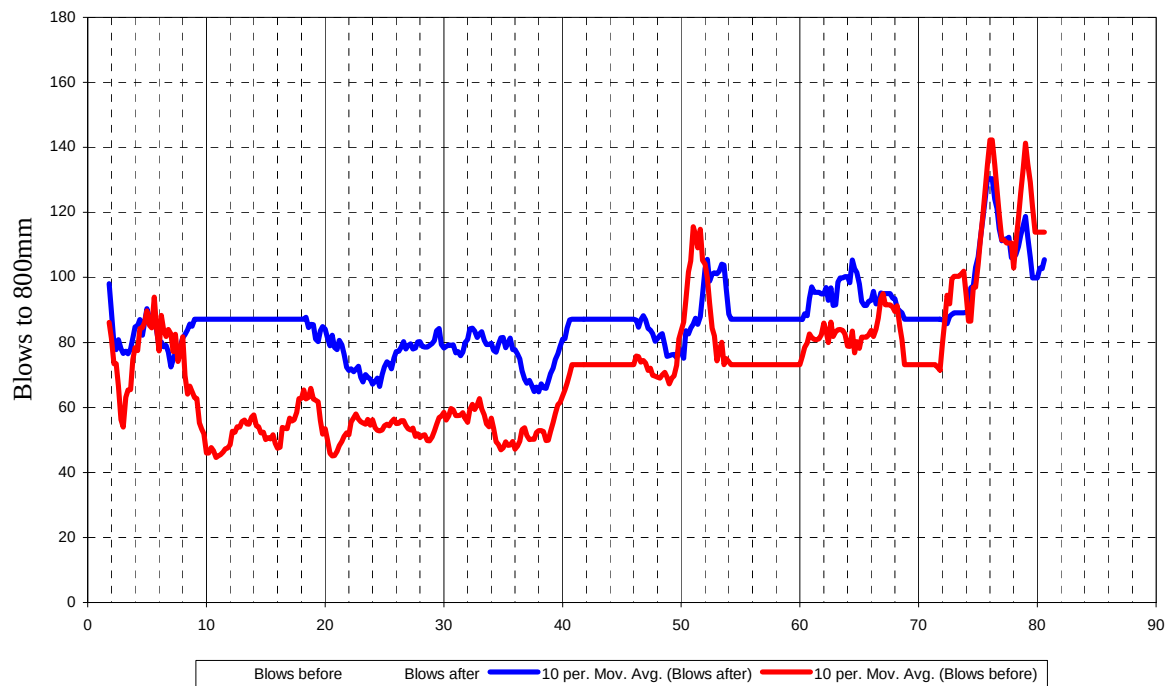


Figure 14. DSN800 blows.

The cumulative difference of the DN800 DCP values before and after impact rolling on the subgrade is shown in Figure 15. The form of the cumulative difference graph before impact rolling corresponds closely with that found from the FWD investigation on the road before impact rolling. This shows that the measured pavement response is mainly dependant on the condition of the in situ sand subgrade and that the pavement layers (calcrete base) did not have any significant effect on the total strength of the pavement. It therefore shows that the layer works did not protect the subgrade adequately and that if the pavement has to be rehabilitated that the subgrade must be improved to a great extent.

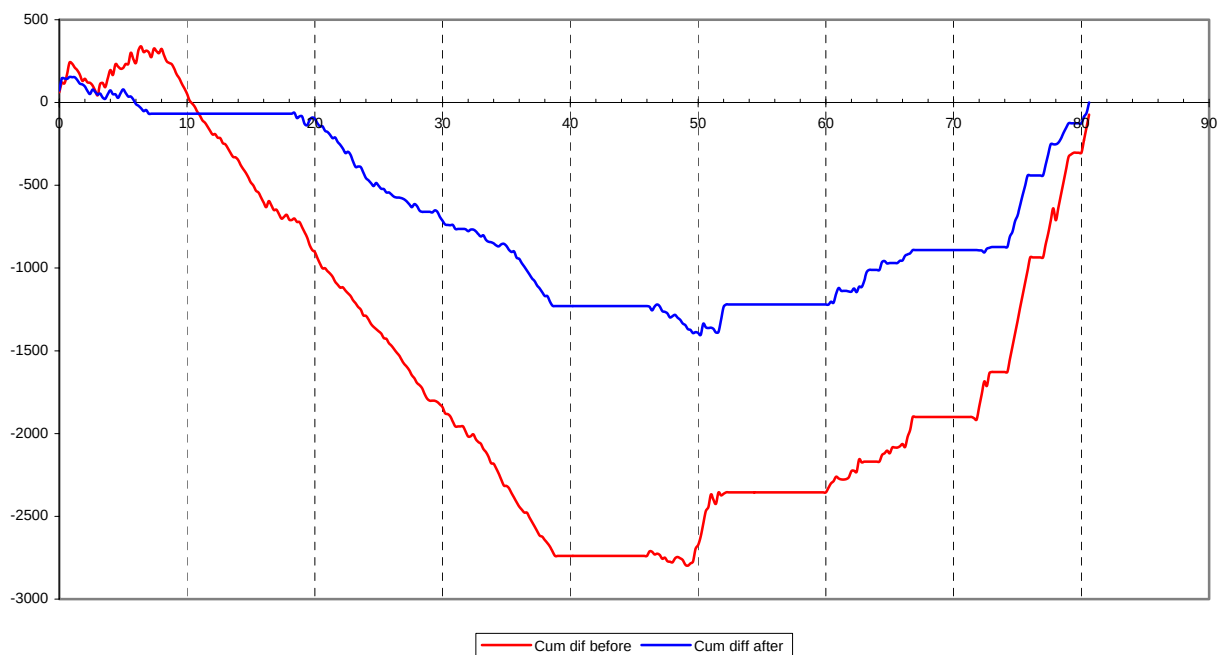


Figure 15. Cumulative difference of the DN800 DCP values before and after impact rolling.

The form of the DSN800 cumulative difference graphs before and after impact rolling also corresponds well with each other except that the total difference of the values after impact rolling are only about half of those before impact value. This is an indication of the strength gained by the impact rolling.

The trend in all the DCP results is fairly consistent as follows:

- The DCP CBRs in the originally compacted formation are relatively high in the uppermost 600 mm of the road formation to original road surface level.
- The DCP CBR results in the uppermost 400mm to original road surface level are generally worse after compaction. This suggests that the amplitude of the compactor (250mm) has had a disturbing (de-densifying) effect to that depth. Normally, the greater the amplitude of the impact compactor, the greater the depth of disturbance of the material. This depth will be the depth of influence of vibratory rollers.
- There is a significant improvement in DCP CBR at depths of 600mm to 2m below original surface level (Figure 11).

The DCPs before and after impact compaction were also analyzed using the CSIR DCP program. The above conclusions can be confirmed as shown below in Figure 15(a). This data is the average for 7km between km0+000 and km7+000.

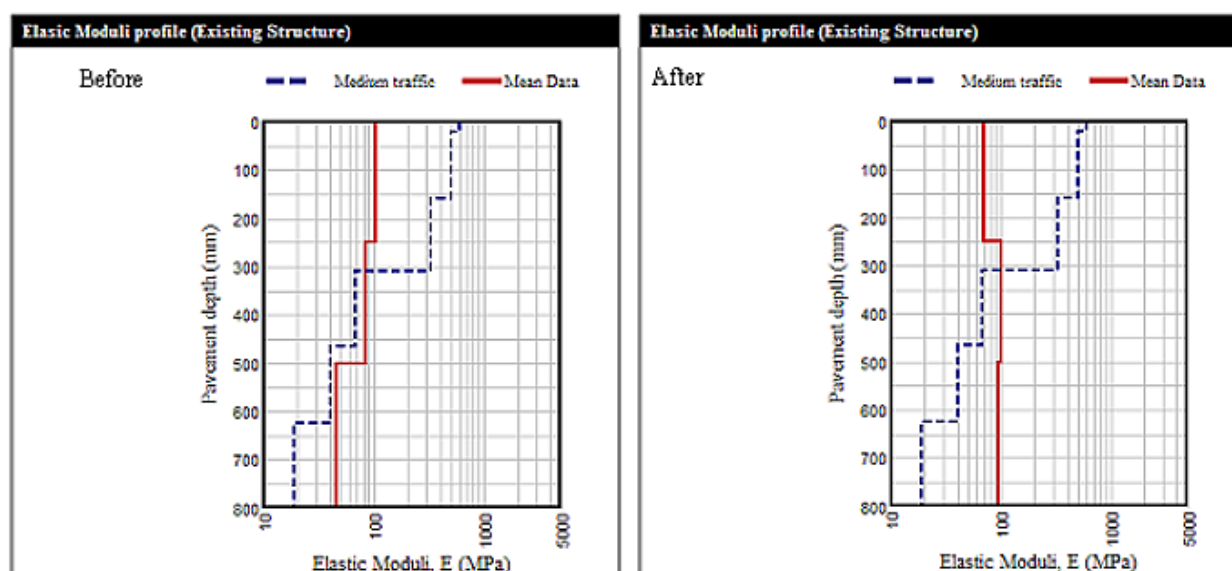


Figure 15(a). Average E-Moduli for 7km before and after impact compaction.

5.7 FWD Tests After Construction

Deflection tests were done at 250m intervals on the road section from km39.00 to km62.00 before and after rehabilitation. These FWD deflection test results are presented in Figure 16 together with the cumulative difference of the deflections over the same road section.

The tests done before rehabilitation are shown in thin red lines and those done afterwards are shown in thick blue lines. The 90th percentile FWD deflection value for this section before rehabilitation is 0.853mm that represents a very flexible pavement (TRH12:1997, Flexible Pavement Rehabilitation Investigation and Design, Pretoria, South Africa) with only about 0.2 to 0.4 million equivalent E80s (MESA) life left. In comparison the same value after rehabilitation is 0.411mm that represents a stiff pavement (TRH12:1997) with an allowed traffic loading of 1.6 to 3.0 MESA. The 90th percentile value of the FWD deflections more than halved after rehabilitation compared to the values taken before rehabilitation.

On the same graph the cumulative differences of the FWD deflections are also compared before and after rehabilitation. The cumulative differences before construction, shown in a thin red line, were halved to make the graph more easily comparable to the values after construction as well as to be able to still present it on the same graph as the normal deflections.

The slopes of the two graphs before and after rehabilitation follow each other closely between km39 and km57. This means that the most important factor influencing deflection before and also after rehabilitation is the sub-grade. This is to be expected on deep sand in situ sub-grade pavements and this is also the reason why it was decided to increase the compaction of the upper 2m of the in situ sand sub-grade.

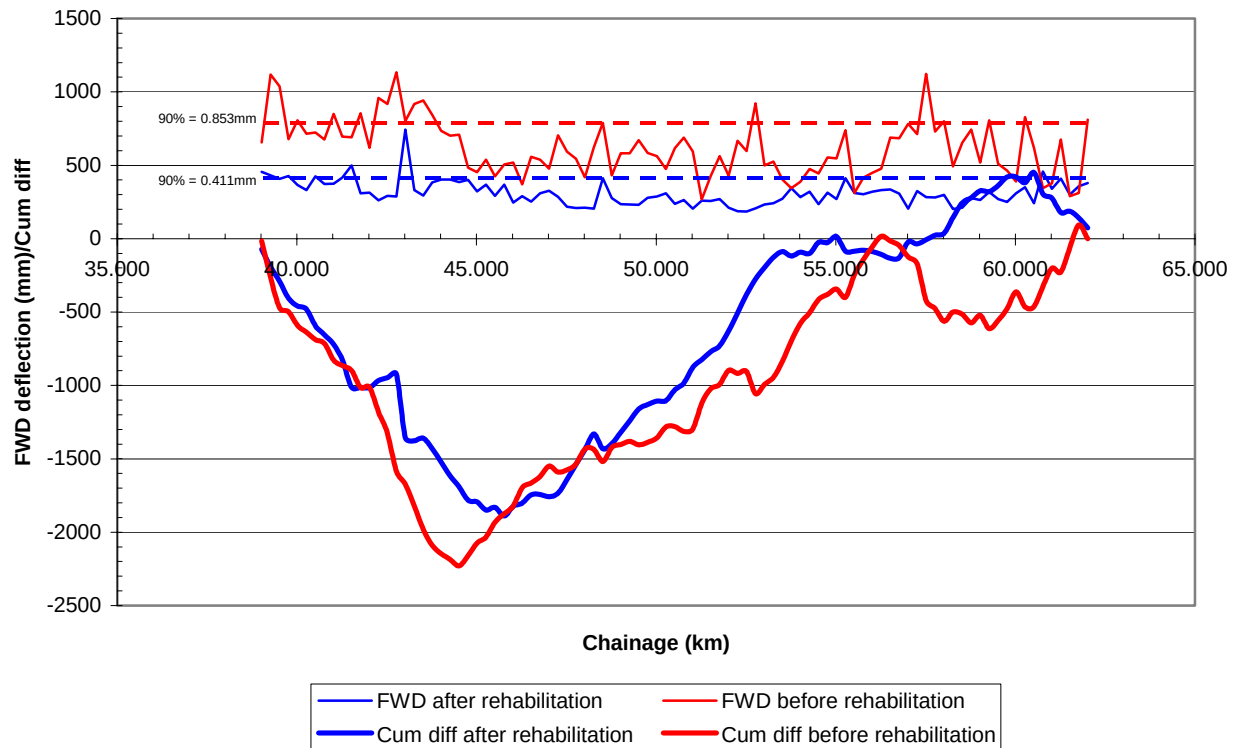


Figure 16. Deflection tests.

The FWD data was used to back calculate the stiffness values obtained after construction of the road. The FWD tests were conducted on top of the primed base. The back-calculated values are compared below to values assumed during the mechanistic design.

DESIGN VALUES						
	E (Mpa)	Parameter (Criteria)	Remaining Life (MESA)			
Base	300	SF= 2	>10			
Subbase	250	SF= 1.13	2.5			
Selected	220	$e_v = 278.9$	4.2			
Subgrade	200	$s_v = 54.33 \text{ MPa}$	2.3			
BACK-CALCULATED VALUES						
EAST LANE				WEST LANE		
	E (Mpa)	Parameter (Criteria)	Remaining Life (MESA)	E (Mpa)	Parameter (Criteria)	Remaining Life (MESA)
<i>Average Stiffness</i>						
Base	663	SF= 2	>99	747	SF= 2	>99
Subbase	412	SF= 1.13	4.6	458	SF= 1.13	4.6
Selected	250	$e_v = 278.9$	>99	249	$e_v = 272.4$	>99
Subgrade	227	$s_v = 51.85 \text{ MPa}$	>99	244	$s_v = 51.19 \text{ MPa}$	>99
<i>80% Percentile Stiffness</i>						
Base	474	SF= 2	>99	539	SF= 2	>99
Subbase	304	SF= 1.14	4.7	412	SF= 1.09	3.6
Selected	250	$e_v = 299.1$	>99	250	$e_v = 287.8$	>99
Subgrade	198	$s_v = 54.63 \text{ MPa}$	>99	250	$s_v = 54.85 \text{ MPa}$	>99
<i>90% Percentile Stiffness</i>						
Base	388	SF= 2	>99	451	SF= 2	>99
Subbase	252	SF= 1.14	4.7	320	SF= 1.11	4.1
Selected	250	$e_v = 310.2$	>99	250	$e_v = 308.8$	>99
Subgrade	189	$s_v = 56.8 \text{ MPa}$	>99	227	$s_v = 56.71 \text{ MPa}$	>99

6. CONCLUSIONS

This paper demonstrates that, based on the deep compaction of the subgrade on the Trans Kalahari Highway between Jwaneng to Sekoma in Botswana, the life of the pavement can be significantly increased by utilizing impact compaction, a process that is appreciably cheaper compared to working the layers in thin shifts using conventional compaction equipment. During the mechanistic design process, the critical element was the thickness of the subbase and the strength of the subgrade. At design, it was predicted that the remaining life of the subbase would be 2.5 MESA and that of the subgrade 2.3 MESA whilst after impact compaction, the back-calculated values indicated a remaining life of more than 99 MESA for the subgrade and more than 4.5 MESA for the subbase. It was further demonstrated that the CIR approach could be successfully applied, in a consistent and reliable manner, to monitor compaction requirements/specifications on a project. With the use of CIR, impact compaction results were also available immediately after the rolling process. This speeded up the decision-making process on site and it helped to make it simpler to identify problem areas where the specified compaction could not be obtained.

7. ACKNOWLEDGEMENTS

The authors wish to acknowledge the work of Jaco Liebenberg of Stewart Scott International (Sandton Office) for his work on the development of the transfer function and his analysis of the FWD results after construction. The work of Hechter Theyse and Chris Semmelink of CSIR Transportek is further acknowledged for their contribution towards the development of the transfer function and on the K-mould tests. The information provided by Pieter Wilken of Landpac, Nigel, Gauteng, South Africa is also appreciated.

8. REFERENCES

- Wilken, P.J., 2001. **The development of the Continuous Impact Response (CIR) system.** Proc The Geotechnical Division of the South African Institution of Civil Engineering. Seminar on Ground Improvement, South Africa, October.
- Barendberg EJ, 1971, **behaviour and performance of aggregate-soil systems under repetitive loads**, Research Report PB 204 267, Department of Civil Engineering, University of Illinois, Urbana, Illinois, Unites States.
- Maree JH, 1978, **Ontwerpparameters vir klipslag in plaveisels** (in Afrikaans), MEng dissertation, University of Pretoria, Pretoria, South Africa.
- Semmelink CJ, Jooste FJ, de Beer M, 1997, **Use of the K-mould in determination and analysis of the elastic and shear properties of road materials for flexible pavements**, Proceedings of the Eighth International Conference on Asphalt Pavements, University of Washington, Seattle, Washington, United States, Vol. II, pp 1643 – 1658.
- Wolff H, 1992, **Elasto-plastic behaviour of granular pavement layers in South Africa**, PhD thesis, University of Pretoria, Pretoria, South Africa.