THE EFFECT OF REHABILITATION ON THE STIFFNESS OF PAVEMENT LAYERS *

P W de Bruin and A T Visser

GIBB Africa, PO Box 35007, Menlopark, Pretoria, 0001, South Africa, pdbruin@gibb.co.za
Professor, Department of Civil Engineering, University of Pretoria, Pretoria, 0002,
avisser@postino.up.ac.za

1 INTRODUCTION

Most of the roads in the road network in South Africa were built between the sixties and mid eighties and are reaching the end of their design life. With a shortage of funds available for new facilities, the emphasis on provision of roads is focussed on the rehabilitation of existing roads. A meaningful assessment methodology is therefore required, for the evaluation of the existing structural condition of pavements, with a realistic appraisal of possible alternative rehabilitation measures (Kekwick, 1987). Thus, material properties (elastic moduli or stiffness etc) of existing pavement layers need to be determined to calculate the structural capacity of the pavement. The elastic modulus or modulus of elasticity used in this paper refers to the “effective” elastic modulus, i.e. the modulus measured under a dual wheel load of 40 kN (corresponding to an axle load of 80 kN) and a tyre pressure of 520 kPa (tyre pressure used during the deflection measurements and required input into the backcalculation program) (Sanders al, 1993). For the purpose of this study, stiffness is the generic term used for effective elastic moduli, whereas effective elastic moduli refer to the backcalculated stiffness values determined from surface deflection basin measurements (modulus measured under a dual wheel load of 40 kN and a tyre pressure of 520 kPa). The term “effective”, as described in effective elastic moduli, is also used to distinguish between the elastic moduli measured in the laboratory from those determined indirectly *in-situ* with the Multi Depth Deflectometer (MDD) system or backcalculated from field measured surface deflection basin measurements (Freeme et al, 1982, Horak, 1986 and de Beer et al, 1988). Following the analysis of the structural capacity of the existing pavement, possible alternative rehabilitation options, with subsequent simulated elastic properties, need to be considered for rehabilitation purposes. The problem is that material properties such as the stiffness of the layers etc change during construction.

The objective of the paper is to investigate the:

- effect of rehabilitation actions on stiffness (effective elastic moduli) and pavement structure behaviour (stress-stiffening and / or stress softening etc), and
- change in the stiffness (effective elastic moduli) during the rehabilitation phase for the different pavement layers.

An experimental section of road of approximately 8.1 km was evaluated and the stiffness and associated stresses were determined for the different uniform sections for modelling purposes along the road. The material properties determined were then used to compare changes of the stiffness in the before (pre) and after (post) rehabilitation phases.

The intention of this paper is not to give comments on the rehabilitation design, bearing capacity or construction phase, but to use the data to establish the material stiffness or effective elastic moduli through surface deflection measurements (before and after rehabilitation). This invaluable information was used to estimate the in-situ material properties, behaviour trends and material characteristics for the different pavement sections (or models) used in this study.

* This paper is based on the MEng project report submitted by the first author to the University of Pretoria

---

2 EXPERIMENTAL DESIGN AND EVALUATION

2.1 Background

An experimental section of road of approximately 25.8 km was identified in 1992 for rehabilitation investigation purposes. In January 1995 work commenced with a substantial part of the section of road requiring re-construction of the existing base to be used as a new stabilised C4 (cemented natural gravel) subbase and the addition of a new G2 (graded crushed stone) base course. However, some sections also required the existing subbase to be stabilised as a C4 cemented lower subbase (upper selected) layer before adding a new C4 cemented subbase and G2 graded crushed stone base course. The material codes G2, G4, G5, G7, G10 and C4 etc used in this paper are well documented and described in TRH 4 (1996) and TRH 14 (1987). A detailed bearing capacity investigation was carried out between km 0.0 and km 8.1 to define the behaviour of the pavement. As-built information as well as data from results of deflection basin measurements conducted with the CSIR Deflectograph was supplied for the after rehabilitation design phase. As-built data, test pit data, visual condition assessment data and deflection measurements from the CSIR Deflectograph, before rehabilitation commenced, were also available.

For the purpose of the paper analyses were performed for two different phases. The first phase will refer to as the “**before”** rehabilitation phase and the second phase to the **“after”** rehabilitation or construction phase.

2.1.1 Rehabilitation investigation and design

During the original rehabilitation investigation in 1994, the section of road between km 0.0 and km 8.1 was divided into the following six uniform sections, i.e.:

- Section 1A: km 0.0 to km 1.0 (length 1.0 km)
- Section 1B: km 1.0 to km 1.6 (length 0.6 km)
- Section 2A: km 1.6 to km 3.1 (length 1.5 km)
- Section 2B: km 3.1 to km 5.1 (length 2.0 km)
- Section 3A: km 5.1 to km 5.9 (length 0.8 km)
- Section 3B: km 5.9 to km 8.1 (length 1.0 km)

These uniform sections were used as a basis for the analysis of the road. However, during the analysis of the *after rehabilitation* deflection basin data, additional variations were identified. This had to be taken into account to compensate for changes due to construction. Consequently, Section 3B was divided into two sections, i.e.:

- Section 3B1: km 5.9 to km 7.0 (length 1.1 km)
- Section 3B2: km 7.0 to km 8.1 (length 1.1 km)

The identification of uniform sections will be briefly discussed in paragraph 3.1.2 (a).

2.1.2 Rehabilitation design specified

The designs for the various uniform sections of road between km 0.0 and km 8.1 as contained in the tender documents are as follows:

- Design 1: km 0.0 to km 3.1
  - Rework existing surfacing and base course as a new 150mm subbase and stabilise to the required specification (C4 cemented subbase), compact to 95% Mod AASHTO (CSRA, 1987a).
  - Import 125mm G2 graded crushed stone base course.

  - 13.2 and 6.7mm double seal surfacing.
- Design 2: km 3.1 to km 5.1
  - As for Design 1 but with a 150mm G2 graded crushed stone base course.
- Design 3: km 5.1 to km 5.9
  - Rework existing surfacing and base course as a new 150mm C4 cemented lower subbase (upper selected layer) and stabilise to the required specification, compacted to 95% Mod AASHTO (CSRA, 1987a). .
  - Import 150mm C4 cemented subbase and stabilise to the required specification, compacted to 95% Mod AASHTO (CSRA, 1987a).
  - Import 125mm G2 graded crushed stone base course.
  - 13.2 and 6.7 mm double seal surfacing.
- Design 4: km 5.9 to km 8.1
  - As for Design 3 but with 125mm G2 graded crushed stone base course.

3 EVALUATION METHODOLOGY AND ANALYSIS OF DATA

3.1.1 Methodology

The methodology, approach and analysis that were used to derive the effective elastic moduli for the different pavement layers will be briefly describe and involved the following steps:

- Collection of all available data. This included a preliminary investigation during which available information on as-built data, pavement structural data, Dynamic Cone Penetrometer (DCP) testing, deflection measurements and material test results were obtained,
- Identification of uniform sections for both the before and after rehabilitation phases,
- Verification of the accuracy and reliability of uniform section and data to minimise any errors in the evaluation process,
- Development of pavement models for each uniform section using deflection basin parameters and pavement structure data,
- Manual backcalculation of the effective elastic moduli with the CHEVRON program (Chevron 15 and Coetzee et al (1982)),
- Comparison of the pavement response in terms of stress dependent behaviour and changes within the pavement layers' effective elastic moduli for the before and after rehabilitation phases, and
- Discussion of the findings of the results.

3.1.2 Material characterisation and analysis

(a) Identification of uniform pavement sections

Uniform pavement sections were identified by using different accepted techniques in a holistic approach. For the before rehabilitation phase, as-built data together with recorded visual assessments, rut depth measurements and deflection basin parameters were used to divide the road into uniform sections. Deflection basin parameters were also used to identify the uniform sections for the after rehabilitation design phase. As discussed previously, Section 3B was divided for both the phases into two new uniform sections. Although small variations were noted within the uniform sections identified in the after rehabilitation design phase, it was found to be unimportant and the uniform Sections 1A to 3A were kept the same for comparisons for both phases.

The identification of uniform sections can be best assessed by taking the deflection basin measurements (and parameters) at fixed intervals along the road. The measurements are then plotted to provide a visual indication of the variation. Combining this with and using different cumulative sum of deviation techniques (AASHTO (1985) after Ullidtz, 1998), uniform sections were determined without difficulties. It should be noted that a relative large number of uniform sections were identified. This was to ensure that the characterisation of the materials used is accurate and reliable. The 90th percentile and representative deflection basin parameters processed for each of the various uniform sections identified, as well as the percentage error that was measured during the backcalculation analysis are explained in detail by de Bruin (2000).

After verification, re-assessment and further analysis of all available data the uniform sections were re-identified (with small alterations). The re-identified uniform sections for both the before and after rehabilitation phases, for the experimental design section are as follow:

- Section 1A: km 0.0 to km 1.1 (length 1.1 km)
- Section 1B: km 1.1 to km 1.6 (length 0.5 km)
- Section 2A: km 1.6 to km 3.2 (length 1.6 km)
- Section 2B: km 3.2 to km 4.9 (length 1.7 km)
- Section 3A: km 4.9 to km 5.9 (length 1.0 km)
- Section 3B1: km 5.9 to km 7.0 (length 1.1 km)
- Section 3B2: km 7.0 to km 8.1 (length 1.1 km)

Length 8.1 km

A good rule of thumb is that the Coefficient of Variation (CoV) for the different deflection basin parameters, especially the peak deflection must be less than 25 % (Jordaan et al, 1992). From the data almost all the CoV's of the different identified uniform section's deflection basin parameters, for both the before and after phases, are less than 25 % (de Bruin, 2000).

(b) Pavement modelling and non-linear behaviour of subgrade

Deflection basin parameters together with the pavement structure data were used to develop a pavement model for each of the identified uniform sections. These models, used in the final analysis of the pavement bearing capacity, are shown in Appendix A. The different pavement structures, as per uniform section and phase (before and after rehabilitation), were modelled by using a multi-layered linear elastic computer programme CHEVRON (Chevron 15 and Coetzee et al (1982)). The effective elastic moduli were modelled under a dual wheel single axle load of 80 kN and a tyre pressure of 520 kPa (load and tyre pressure used during the deflection measurements). Poisson ratios of 0.35 were used for both the granular and cement treated layers ("equivalent" granular). The backcalculated effective elastic moduli of the respective pavement layers for each of the uniform sections are also shown in Appendix A.

One important aspect that needs consideration in the modelling of a pavement structure is the non-linearity of the subgrade. Ullidtz (1983) stated that a small error in the determination of the subgrade modulus could lead to large errors in the effective elastic moduli of the other layers. The non-linearity of the subgrade is accurately accounted for by the following two methods:

- non-linear backcalculation techniques based on a finite element approach and/or programs (Rohde et al, 1992), and
- non-linear backcalculation procedure by means of the Multi-Depth Deflectometer (MDD) approach (Horak, 1986).

Rohde et al (1992) stated that although the non-linear elastic backcalculation approach based on a finite element model is probably the only available method to fully account for the non-linear elastic behaviour of soils, it is too costly, complex and cumbersome for daily use.

Traditionally subgrade stiffness is usually defined by a semi-infinite layer thickness (linear elastic modelling), which made it difficult to account for the non-linear behaviour of the subgrade. Subgrades are usually modelled as a layer containing a constant effective elastic modulus of semi-infinite thickness. In order to simulate pavement response under such conditions the subgrade is represented by an “average” modulus (Jordaan, 1994). This modulus will usually be higher than the actual value at the top of the subgrade and lower than actual in depth to achieve balance. The most vulnerable point in the subgrade is at the top of the subgrade, where the effective elastic modulus is normally the lowest (Jordaan, 1994). By introducing a semi-infinite layer of a high modulus (“rigid layer”) in depth, a more accurate representation of the actual effective elastic moduli can be simulated (Jordaan, 1994). An improvement or minor deviation to the linear elastic approach is thus to use a “rigid layer” approach instead of a semi-infinite layer, which accounts for changes in the subgrade stiffness. Rohde et al (1992) showed that by using an apparent “rigid layer” layer underlying the subgrade, at a determined depth, changes in subgrade stiffness can be accounted for by deflection analysis routines based on the layered elastic approach.

Research by Roque et al (1992) showed that accurate prediction of deflections, stresses and strains can be obtained from non-destructive testing (NDT) devices (i.e. surface deflection measurements) by using linear elastic analysis to simulate the non-linear response of pavements. A comprehensive analysis, on 16 different pavement sections, was performed using both linear and non-linear finite element models on a broad range of pavement structures and material properties. The finite element computer program ILLIPAVE (ILLIPAVE, 1990) was used to simulate non-linear response, while the computer program BISAR (deJong et al, 1973) was used in the linear elastic layer analyses. Excellent correspondence was observed between surface deflections predicted by linear and non-linear analyses performed at design wheel load levels of 40 kN (Roque et al, 1992).

The following conclusions were made by Roque et al (1992):

- effective elastic layer moduli can be determined to simulate the non-linear response of pavement structures accurately when these moduli are used in elastic layer analysis,
- non-linear response of crushed stone base courses may be adequately represented by one effective layer modulus in linear elastic analysis,
- non-linear response of the different subgrade pavement structures investigated may be adequately represented by two elastic layers and corresponding effective layer moduli,
- the upper 300 mm of the subgrade should be modelled as a separate layer for backcalculation of effective elastic moduli from NDT deflections, and in subsequent predictions of pavement response using elastic layer analysis, and
- better prediction of effective elastic moduli of the different layers and / or subgrade may be found if additional elastic layers and corresponding moduli be used to simulate these layers.

For the purpose of this study the upper 150 mm of the subgrade was modelled as a separate layer and a semi-infinite layer of a high effective elastic modulus (“rigid layer”) was placed at a depth of 1.5 m. This depth was “altered” for the after phase by only including the additional layers added during construction to the pavement structure. Layer thicknesses for the before and after phases that were not altered during construction were kept the same for comparison purposes and due to the influence that layer thickness has on the effective elastic moduli. In order to minimise the error due to the correct placement or determination of the “rigid layer” depth (Rohde et al, 1992), the subgrade was further subdivided into 200 mm layers. The subdivision of the subgrade not only assist in building a better “model”, but it also simulates better effective elastic moduli and the “non-linear” behaviour of the subgrade. This may also minimise the error prone to the correct placement of the subgrade in depth.

Effective elastic moduli for the different pavement models are given Appendix A.

(c) Deflection basin measurements

The most accurate way to calculate or establish the changes in material properties (stiffness) is to calculate it in-situ from maximum deflection measurements measured with the Multi Depth Deflectometer (MDD) system (de Beer et al, 1988). This way of measurement is not always feasible or available. An alternative and popular option is to use backcalculation techniques to determine the effective elastic moduli by means of surface deflection basin measurements. This is as a result of the theoretically calculated pavement response that can be directly compared with the measured response in terms of deflections. Various researchers indicated that the structural integrity of pavements can be assessed by using surface deflection measurements and this in turn, can then be used as an important input for the accurate characterisation of pavement materials (e.g. recent research: Horak (1988), Jordaan (1994), McCullough and Taute (1982) and Kilareski (1982)). For the purpose of this study, deflection basin parameters were used to determine the linear elastic properties of the materials and to characterise the pavement models of the different uniform sections.

Deflection basin measurements were obtained by using the CSIR Deflectograph for both the before and after rehabilitation phases. This data was measured at approximately 6 to 7 m intervals, along the entire length of the road section, and were used to process the deflection basin parameters for each 100 m section along the road. The mean value, standard deviation and 90th percentile values were calculated for each parameter. With deflection measurements available at 6 to 7 m and by calculating the 90th percentile value for the surface deflections and / or deflection basin parameters, it was now possible to accurately identify the uniform sections and to further characterise the pavement material models.

Due to the fact that the deflection basin parameters gave full details of the deflection basin (Horak, 1988), these were used for calculation purposes. The significance of using the deflection basin parameters instead of the deflection measurements at different positions in the beam can be motivated by the fact that deflection basin parameters:

- accommodate any changes in stiffness between different *pavement layers* due to fact that it accounts for the positive and negative curvature of the basin,
- facilitate the evaluation of the structural condition of the various layers within the pavement,
- Surface Curvature Index (SCI), Base Damage Index (BDI) and Base Curvature index (BCI) (see Table 1) values correlate well with the effective elastic moduli values of the base, subbase, selected and subgrade layers (Horak (1987), Kilareski (1982) and McCullough and Taute (1982)),
- describe the essential features of the deflection basin in full.

The deflection basin parameters used in pavement characterisation and analysis are given in Table 1.

Table 1: Deflection Basin Parameters (TRH 12 (1997) and Horak (1988))

Parameter	Formula
1. Maximum deflection (mm)	δ_0
2. Radius of Curvature (RC) (m)	$RC = r^2 / [2 * (\delta_0 - \delta_{127})]$ r = 127 mm
3. Surface Curvature Index (SCI) (mm)	$SCI = \delta_0 - \delta_{305}$
4. Slope of Deflection (SD)	$SD = \tan^{-1} (\delta_0 - \delta_{610}) / 610$
5. Base Damage Index (BDI) (mm)	$BDI = \delta_{305} - \delta_{610}$
6. Base Curvature Index (BCI) (mm)	$BCI = \delta_{610} - \delta_{915}$
δ_i = deflection measured at the given distance (i) from the point of load	

Surface Curvature Index (SCI) is indicative of the structural conditions of the upper base and / or subbase layers (0 mm to 305 mm), the Base Damage Index (BDI) of the selected and / or top part of the subgrade (305 mm to 610 mm). The Base Curvature Index (BCI) gives an indication of the structural condition of the subgrade (610 mm to 915 mm), while the Radius of Curvature (RC) is a good indication of the structural condition of the surfacing and / or base layers (top of the structure).

It is important to take cognisance of the fact that a *Log-Normal distribution* be used for the percentile Radius of Curvature (RC) during the evaluation process, as previously described. For all the other deflection basin parameters the Normal Distribution were used to calculate the moduli (Jordaan et al, 1992).

3.1.3 Discussion of the effect of rehabilitation on the behaviour of different material models

(a) Pavement balance

The behaviour of pavement can, inter alia, be described in terms of the balance (strength-balance) of a pavement (Kleyn, 1984). Jordaan (1994) describes pavement balance as the relative relationship between the load bearing properties of the successive pavement layers.

The concept of balance within the pavement supports the principal of the interaction between the layers and helps to determine whether the pavement structure is likely to change its state rapidly or remain in a stable state in the future. Balance is defined as the state of the pavement in which the layers are not overstressed (Freeme et al, 1987). Pavement structures containing different pavement layers will change in time and different trends in behaviour have been documented during HVS tests done in South Africa (de Beer et al (1988), Freeme et al (1982), Freeme et al (1987), Maree et al (1982) and Jooste et al (1997)). Research recently done by Jooste et al (1997) confirmed that pavement balance, behaviour and state change during HVS testing. This was also demonstrated earlier by Kleyn (1984).

A pavement can become unbalanced due to the ingress of water into particular layers or owing to the changed state of the material through the action of abnormally heavy loading and environmental changes (Freeme et al, 1987). A pavement is balanced when the stiffness of the successive pavement layers are such that there is no excessive build up of stress between two successive layers and when strains are compatible (Sanders et al, 1993). To achieve balance within a pavement some of the layers may de-densify or de-compact while other layers may compact depending on the external factors (pavement composition, material type, moisture, loading, density, compaction, stress differences etc.), which will play a role. The stiffness may also show some change as a result of an adjustment in the balance of the pavement. Research by Heukelom and Klomp (1962) showed that the stability between two successive layers can be achieved when the stiffness ratio is between approximately 0.5 and 2.2 (depending on the vertical stress).

During the reconstruction or rehabilitation process a sudden change in the material properties (e.g. effective elastic moduli etc) of successive layers can be experienced due to changes in stress and / or strain concentrations within the layers. The elastic properties of the pavement layers may show some change as a result of an adjustment or reorientation in the balance of the pavement (e.g. when the existing pavement base layer is reconstructed as a stabilised subbase and a crushed base layer is added on top) and / or as a result of different stresses in the different pavement layers for both the before and after phases. The material properties (stiffness) of the pavement layers will automatically be adjusted to try and compensate for the changes in stress and strain concentrations introduced in the successive pavement layers when compaction of the added layers on top takes place. Thus, initial changes in moduli in the different pavement layers during the construction phase took place with subsequent changes in the stress concentrations for the different layers, and / or vice versa.

(b) Stress-stiffening and stress-softening behaviour

Various studies and research performed for different types of materials indicated, in general, that the behaviour under traffic and / or compaction of granular materials can be classified as stress-stiffening and that for fine-grained or subgrade materials as stress-softening (Rohde et al (1992), Brown and Pell (1967), Monismith et al (1967) and Maree et al (1982)). Stress dependency of pavement layers can be directly calculated measuring deflections at different wheel loads. These measurements can then be used to determine the effective elastic moduli.

The stiffness' of different pavement layers are affected by the characteristics of material in the layers above and below the layer (model dependent). This is usually a function of the load applied, which induces different stresses within the different pavement layers. During the rehabilitation or reconstruction phase additional layers were added to the existing pavement structure. Therefore, although the same load was applied to measure the deflections, effective elastic moduli and stresses for both the before and after rehabilitation phases, the actual load applied to the existing (old) base layer (before rehabilitation phase), which become the new subbase (sections 1A to 2B) and the new upper selected subgrade (USSG) (sections 3A to 3B2) (after rehabilitation phases), differ. Thus, due to the "different load levels" applied to the different pavement layers (before and after rehabilitation phases), it was possible to determine the stress dependency of different pavement layers.

For the purpose of this study, the stress dependency for different material types (e.g. granular or fine-grained (sugbrade)) were established. The effective elastic moduli were determined from the surface deflection measurements (deflection basin parameters) through manual backcalculation procedures, and were used as input values to determine the stresses within the different pavement layers. The reason behind the establishment of the stress dependency behaviour for the different layer materials is that it gives valuable insight in the change in effective moduli for the different existing pavement layers after rehabilitation as well as for the newly constructed layers.

For granular materials (G2 and G5) as encountered on the project and used for this study, the stress states were calculated at the centre of the dual wheel load as well as under the single outer wheel load. The principal stresses and / or sum of principal stresses and / or bulk stresses ($\theta = \sigma_1 + \sigma_2 + \sigma_3$) were analysed in the top, middle and lower parts of each layer, whereafter an average value was determined. **Although not discussed in detail in this paper, the stress-stiffening behaviour for granular materials was demonstrated during the analysis for the old (as in its original state) and newly constructed base course layers (de Bruin, 2000)**.
The original G5 subbase layers, from Section 1A to 2B (before phase) that became the new upper selected subgrade (USSG) layer for the after rehabilitation phase, showed quite the opposite (stress-softening behaviour) of the granular materials (G2 and G5 bases) discussed above. The reasoning behind this unusual behaviour may be due to the poor support from the G10 subgrade layers (G5 material constructed on a G10 material) and / or inadequate protection of the newly constructed base and subbase layers. The effective elastic moduli of these sections' (old subbase layer) decrease with an increase in the stress. The effect of different stresses and effective elastic moduli in the adjacent layers and pavement balance may have a direct effect on the stress dependency of different materials. This unusual behaviour needs to be verified and further investigated before final conclusions and recommendations can be made. Taking all the above into consideration, the effect of structural pavement balance on the stress-stiffening and stress-softening behaviour as well as on the effective elastic moduli (modular ratio) of granular materials warrants further research (de Bruin, 2000).

For the fine-grained or subgrade materials (G7 and G10) as encountered on the project (sections 1A to 3B2), the deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) was determined under the centre of the dual wheels as well as under the wheel load, on top of the different layers. The bulk stresses were calculated in the middle of the different subgrade layers as well, in order to compare it with the deviator stress. For both the deviator stresses and bulk stresses **the effective elastic moduli decrease with an increase in deviator stress or bulk stress**. Although it is not discussed in detail in this paper, the theory that fine-grained material is stress dependent and stress-softening, was clearly demonstrated during the research and the findings from various researchers are confirmed (de Bruin, 2000).

(c) Effect of rehabilitation of the effective elastic moduli

As shown and discussed in the previous paragraphs, pavement balance and the stress dependency of materials have an influence on the changes of stiffness during the rehabilitation of pavement layers. Due to the fact, for example, that for fine-grained subgrade materials an increase in moduli can be experienced with a decrease in stress and vice versa, the change and / or simulation of moduli of successive layers may play an important role when modelling pavement structure for future rehabilitation purposes. For example, when additional layers are added during the rehabilitation or reconstruction of a road, the existing fine-grained subgrade layers will experience a decrease in deviator stress with a subsequent increase in effective elastic moduli (see Appendix A, Section 3B2). However, the following factors will also contribute or have an influence on the changes in stiffness during rehabilitation: composition of the pavement, the density of the pavement layers, the moisture content, impact of stabilisation, environmental influences, granular interlocking and construction variables etc.

In Figure 1 and in Appendix A, the effective elastic moduli, for both phases, are presented. For Sections 1A to 2B similar trends were noted. For almost all of these sections, the effective moduli of the subgrade (top of subgrade layer) reduced after construction. This is quite the opposite of what was to be expected. If the stress is reduced in the lower layers, for example when layers are added on top and constructed in such a way that stresses in the lower layers are reduced, the effective elastic moduli for the lower layers (with fine-grained subgrade properties) will eventually increase. As previously discussed, the first four section's (Sections 1A to 2B) subbase layers were not effectively stabilised and can only be regarded as lightly cemented layers. Due to the fact that the newly stabilised subbase layers (after phase) were not stabilised effectively, the base course layer could not been compacted properly. Thus, the top layers did not provide adequate strengthening to protect the lower lying layers. This together with the G5 material (old subbase / new upper selected subgrade (USSG)) overlying a G10 subgrade layer may have caused the unbalanced stress concentrations. The effective moduli of granular layers can be described as a function of the modulus of the supportive layer and granular interlocking during the construction of the layer. Compaction and de-compaction may also have taken place within the other layers, which subsequently had an effect on the effective moduli of the different pavement layers. As previously discussed, this unusual behaviour needs to be verified and further investigated by means of trail pits, more detailed Dynamic Cone Penetrometer (DCP) tests and density tests etc before a final conclusion and recommendation on this unusual behaviour can be made.

The pavement behaviour in terms of the changes in stiffness of the underlying pavement layers (subgrade) of Sections 3A to 3B2 is just opposite to the behaviour of the first four sections. This difference can be hypothesised since more layers were added during construction, the support of the subgrade is better and the new subbase layers were fully stabilised, in particular Section 3B2. Due to these factors, a reduction in stress with subsequent increase in effective elastic moduli was noted in the lower subgrade layers. It is also important to take the quality subgrade support of the different pavement sections, as well as the overall balance of the different pavement models into consideration. In Section 3B2 the subbase layers were effectively stabilised. Comparing the moduli for this section with the other sections, the moduli for all the layers improved. **The effective elastic moduli of the new upper selected subgrade (USSG) layer and subgrade have almost doubled (see Appendix A, section 3B2)**. Thus the effect of proper stabilisation and “lifting” of the grade line of the road had an important contribution to the stiffness of the lower layers, the stress concentrations and subsequent to the overall balance of the pavement. **The changes in effective elastic moduli, due to rehabilitation actions, are influenced by the strengthening action, the balance in the pavement, material properties, the composition of the pavement model and the stress-stiffening and softening behaviour of different material types.** Environmental influences, the impact of stabilisation, granular interlocking and construction variables may also influence a change in moduli during rehabilitation.

Another interesting aspect observed is the convergence of the moduli (over a period of 20 years) of the base, subbase and subgrade layers for Sections 1A to 2B (before phase pavement structures) (see the first three layer's effective elastic moduli in Appendix A and Figure 1).

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

The objective of the paper was to investigate the:

- change in the stiffness during the rehabilitation phase for the different pavement layers, and
- effect of rehabilitation actions on the material properties (effective elastic moduli / stiffness) and behaviour of the pavement structures.

Taking the objective of the study into consideration and from the evaluation and analysis of data in this paper, the following conclusions are made:

- The changes in stiffness, due to rehabilitation actions, are influenced by the strengthening action, the existing balance in the pavement, material properties, the composition of the pavement model and the stress-stiffening and softening behaviour of different material types. Changes in stiffness of underlying layers occur as a result of the addition or change in stiffness of the overlying layers.
- The effective elastic moduli of the underlying subgrade layer almost double when additional layers were added during the construction and the existing base and subbase layers were adequately strengthened. For example in Section 3B2, a decrease in stress was experienced in the lower layers, due to the strengthening of the top layers (added new layers on top of existing pavement structure), with a subsequent increase in effective elastic moduli or stiffness (fine-grained subgrade – stress softening behaviour). A resulting reduction in the subgrade moduli of the first four sections (Section 1A to 2B) was observed. The change in subgrade moduli, is quite the opposite in Section 3B2. This needs to be verified and further investigated before final conclusions and recommendations can be made.

- With reasonable traffic growth over time the effective elastic moduli of the base, subbase and subgrade layers will eventually converge (pavement balance). However when a rigid layer (e.g. concrete) is constructed within a pavement structure this will not be the case, within the same time frame.

4.2 RECOMMENDATIONS

From the above evaluation and analysis of data, the following recommendations emanate:

- There is a need to determine the effective elastic moduli in-situ for before and after rehabilitation phases to establish trends in the change and behaviour of the effective elastic moduli.

- Changes in stiffness of underlying layers occur as a result of the addition or change in stiffness of overlying layers. The extent of changes needs to be further verified and investigated, as they could have a marked influence on rehabilitation design.

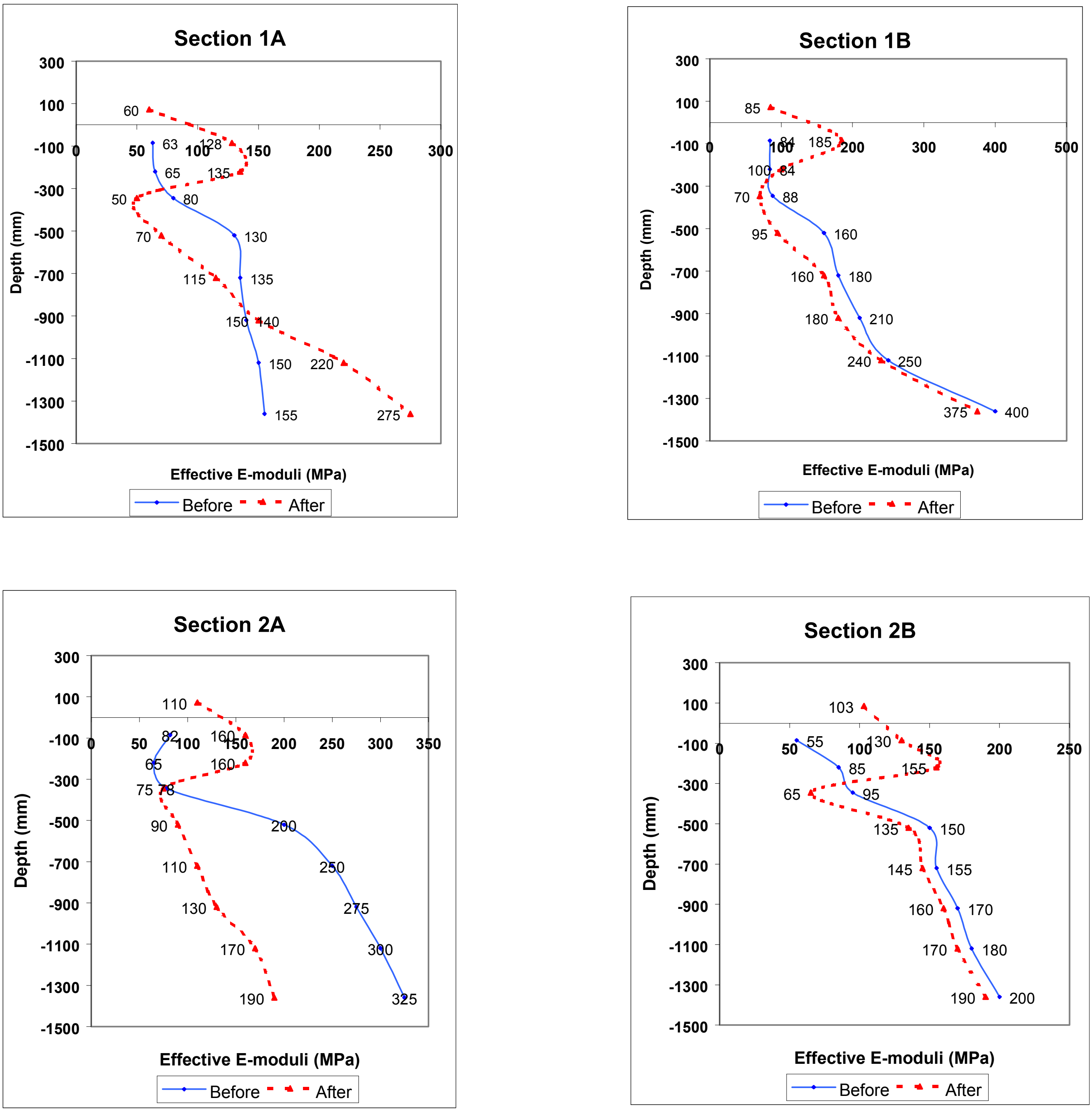

FIGURE 1 : CHANGE OF EFFECTIVE ELASTIC MODULI WITH DEPTH
FOR THE BEFORE AND AFTER REHBILITATION PHASES

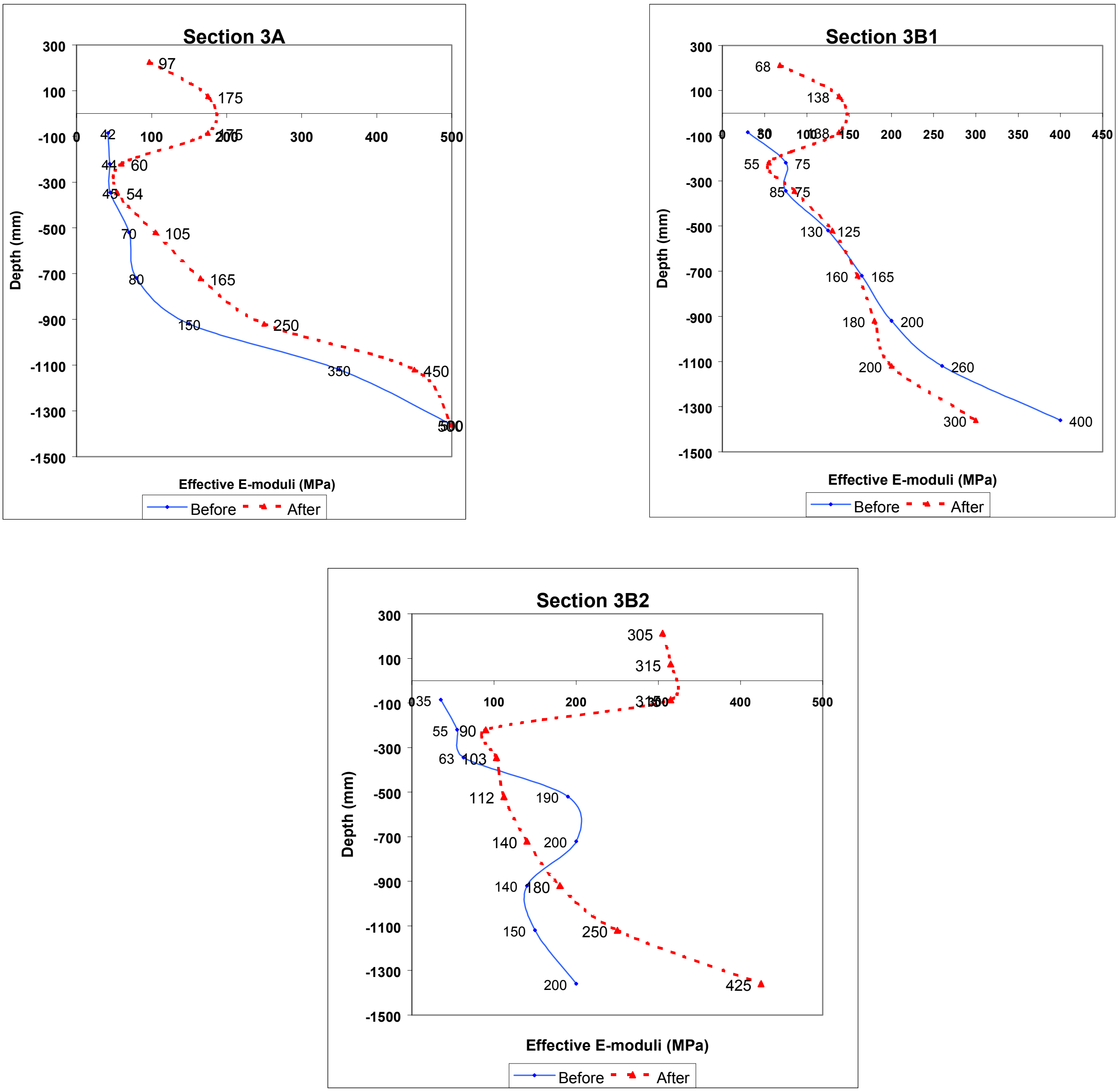

FIGURE 1 (CONTINUE): CHANGE OF EFFECTIVE ELASTIC MODULI WITH DEPTH FOR THE BEFORE AND AFTER REHABILITATION PHASES

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO) (1985). Proposed AASHTO guide for design of pavement structures. NCHRP Project 20-7/24.

Brown, S.F. and Pell, P.S. (1967). An experimental investigation of the stresses, strains and deflections in a layered pavement structure subjected to dynamic loads. Proceedings of the 2nd International Conference on the Structural Design of Asphalt pavements. Preprint Volume, Michigan, USA. pp 384 – 403.

CHEVRON 15. Chevron 15 layered program – Obtained from Chevron research company, Richmond, California.

Coetzee, C.H., Marais, C.P. and Walker, R.N. (1982). CHEV15 program for calculating stresses, strains and displacement in a layered pavement system having a maximum of fifteen layers. NITRR Technical Note TC/27/82, CSIR, Pretoria.

Committee for State Road Authorities (CSRA document) (1987). TRH14: Guidelines for road construction materials. Pretoria, South Africa.

Committee for State Road Authorities (CSRA document) (1987a). Standard specifications for road and bridge works. Pretoria, South Africa.

De Bruin, P.W. (2000). The effect of rehabilitation on the stiffness and behaviour of pavement layers. M Eng project report, Department of Civil Engineering, University of Pretoria, Pretoria.

De Beer, M., van der Merwe, C.J., Rohde, G.T. (1997). Rehabilitation design of flexible pavements in South Africa. Research Report RR 93/296, Department of Transportation, Pretoria.

De Beer, M., Horak, E. and Visser A.T. (1988). The Multi Depth Deflectometer (MDD) system for determining the effective elastic moduli of pavement layers. Reprint of paper prepared for the Proceedings of the 1st International Symposium on Non Destructive Testing of Pavements and Backcalculation of Moduli, ASTM, Baltimore, Maryland, USA.

DeJong, D.L., Peutz, M.G.F. and Korswagen, A.R. (1973). Computer program BISAR, layered systems under normal and tangential loads. External report, Koninklijke-Shell laboratorium, Amsterdam, The Netherlands.

Department of Transport (Colto) (1997). Draft TRH12: Flexible rehabilitation investigation and design. Pretoria, South Africa.

Department of Transport (Colto) (1996). Draft TRH4: Structural design of flexible pavements for interurban and rural roads. Pretoria, South Africa.

Freeme, C.R., de Beer, M. and Viljoen, A.W. (1987). The behaviour and mechanistic design of asphalt pavements. Proceedings of the 6th International Conference on the Structural Design of Asphalt Pavements. Volume I, Ann Arbor, Michigan, USA pp 333-343.

Freeme, C.R., Maree, J.H., and Viljoen, A.W. (1982). Mechanistic design of asphalt pavements and verification of designs using the Heavy Vehicle Simulator. Proceedings of the 5th International Conference on the Structural Design of Asphalt Pavements. Vol1, Delft, Holland, pp 156-173, (CSIR reprint RR362).

Heukelom, W. and Klomp, A.J.G. (1962). Dynamic testing as a means of controlling pavements during and after compaction. Proceedings of the 1st International Conference on the Structural Design of Asphalt pavements. Ann Arbor.

Horak, E. (1986). Effective elastic moduli determined from Road Surface Deflectometer (RSD) measurements. NITRR Technical Note TP/40/86, CSIR, Pretoria.

Horak, E. (1987). The use of surface deflection basin measurements in the mechanistic analysis of flexible pavements. Proceedings of the 6th International Conference on the Structural Design of Asphalt pavements. Volume 1, Michigan, USA, pp 990 - 898.

Horak, E. (1988). Aspects of deflection basin parameters used in mechanistic rehabilitation design procedures for flexible pavements in South Africa. PhD Thesis, Department of Civil Engineering, University of Pretoria, Pretoria.

ILLIPAVE (1990). Calibrated mechanistic structural analysis procedures for pavements. Report to NCHRP 1-26.

Jooste, F.J., Kekwick, S.V., Sadzik, E.S. and Rohde, G.T. (1997). Comparison of accelerated pavement test results with long term pavement behaviour and performance. Proceedings of the 8th International Conference on Asphalt Pavements. Volume II, Seattle, Washington, USA, pp 1027 - 1037.

Jordaan, G.J., Venter, J.S.M. and Van As, S.C. (1992). Optimisation of the number and type of testing needed for the evaluation and assessment of pavements. Project Report PR 91/249, Department of Transport, Pretoria.

Jordaan, G.J. (1994). The South African mechanistic pavement rehabilitation design method. Research Report 91/242, Department of Transport, Pretoria.

Kekwick, S.V. (1987). Application of the mechanistic analysis procedure to pavement rehabilitation: Two case studies. Proceedings of the 6th International Conference on the Structural Design of Asphalt Pavements. Volume 1, Michigan, USA, pp 979 - 898.

Kilareski, W.P., Anani, B.A. (1982). Evaluation of in-situ moduli and pavement life from deflection basins. Proceedings of the 5th International Conference on the Structural Design of Asphalt Pavements. Volume 1, Delft, Holland, pp 349 – 366.

Kleyn, E.G. (1984). Aspekte van plaveiselevaluering en –ontwerp soos bepaal met behulp van die Dinamiese Kegelpenetrometer. M. Ing Verhandeling, Universiteit van Pretoria, Pretoria.

Maree, J.H., van Zyl, N.J.W. and Freeme, C.R. (1982). Effective Moduli and stress dependence of pavement materials as measured in some Heavy Vehicle Simulator tests. Transportation Research Record 852. Transportation Research Board, Washington D.C., pp 52 – 60, (CSIR reprint RR357).

McCullough, B.F. and Taute, A. (1982). Use of deflection measurements for determining pavement material properties. Transportation Research Record 852. Transportation Research Board, Washington D.C., pp 8 – 14.

Monismith, C.L., Seed, H.B., Mitry, F.G., and Chan, C.K. (1967). Prediction of pavement deflections from laboratory tests. Proceedings of the 2nd International Conference on the Structural Design of Asphalt pavements. Preprint Volume, Michigan, USA. pp 52 – 88.

Rohde, G.T., Smith, R.E. and Scullion, T. (1992). Pavement deflection analysis on sections where the subgrade vary in stiffness with depth. Proceedings of the 7th International Conference on Asphalt Pavements. Volume 3, Design and Performance, Nottingham, UK, pp 280 – 295.

Roque, R., Romereo, P. and Hiltunen, D.R. (1992). The use of linear elastic analysis to predict non-linear response of pavements. Proceedings of the 7th International Conference on Asphalt Pavements. Volume 3, Design and Performance, Nottingham, UK, pp 296 – 310.

Sanders, P.J., de Beer, M. and du Plessis, H.W. (1993). Characterisation of the elastic properties of existing pavements: State of the Art. Research Report RR 91/012, Department of Transport, Pretoria.

Ullidtz, P. (1983). Management system for pavement maintenance and rehabilitation based on analytic methods of pavement evaluation. Transportation Research Record 930. Transportation Research Board, Washington D.C., pp 31 – 37.

Ullidtz, P. (1998). Modelling flexible pavement response and performance. Technical University of Denmark, 1st Edition, Denmark.

APPENDIX A

PAVEMENT MODELS WITH DERIVED EFFECTIVE ELASTIC PROPERTIES FROM DEFLECTION MEASUREMENTS (BEFORE & AFTER REHABILITATION) (de Bruin, 2000)

POSITION: **Section 1 A :**
(km 0.0 - 1.1 : Length 1.1 km)

BEFORE
Eff. Emod (MPa)

20mm		Bit. Surface
150mm	**63**	**EG5**
100mm	**65**	**G5**
150mm	**80**	
200mm	**130**	
200mm	**135**	
200mm	**140**	**G10**
200mm	**150**	
280mm	**155**	
v 1500		
Rigid	**50000**	

AFTER
Eff. Emod (MPa)

20mm		Double seal
125mm	**60**	**G2**
150mm	**128**	**C4**
100mm	**135**	**G5**
150mm	**50**	
200mm	**70**	
200mm	**115**	
200mm	**150**	**G10**
200mm	**220**	
280mm	**275**	
v 1625		
Rigid	**50000**	

POSITION: **Section 1B :**
(km 1.1 - 1.6 : Length 0.5 km)

BEFORE
Eff. Emod (MPa)

20mm		Bit. Surface
150mm	**84**	**EG5**
100mm	**84**	**G5**
150mm	**88**	
200mm	**160**	
200mm	**180**	
200mm	**210**	**G10**
200mm	**250**	
280mm	**400**	
v 1500		
Rigid	**50000**	

AFTER
Eff. Emod (MPa)

20mm		Double seal
125mm	**85**	**G2**
150mm	**185**	**C4**
100mm	**100**	**G5**
150mm	**70**	
200mm	**95**	
200mm	**160**	
200mm	**180**	**G10**
200mm	**240**	
280mm	**375**	
v 1625		
Rigid	**50000**	

POSITION: **Section 2 A :**
(km 1.6 - 3.2 : Length 1.6 km)

BEFORE
Eff. Emod (MPa)

20mm		Bit. Surface
150mm	**82**	**EG5**
100mm	**65**	**G5**
150mm	**78**	
200mm	**200**	
200mm	**250**	
200mm	**275**	**G10**
200mm	**300**	
280mm	**325**	
v 1500		
Rigid	**50000**	

AFTER
Eff. Emod (MPa)

20mm		Double seal
125mm	**110**	**G2**
150mm	**160**	**C4**
100mm	**160**	**G5**
150mm	**75**	
200mm	**90**	
200mm	**110**	
200mm	**130**	**G10**
200mm	**170**	
280mm	**190**	
v 1625		
Rigid	**50000**	

POSITION: **Section 2B :**
(km 3.2 - 4.9 : Length 1.7 km)

BEFORE
Eff. Emod (MPa)

20mm		Bit. Surface
150mm	**55**	**EG5**
100mm	**85**	**G5**
150mm	**95**	
200mm	**150**	
200mm	**155**	
200mm	**170**	**G10**
200mm	**180**	
280mm	**200**	
v 1500		
Rigid	**50000**	

AFTER
Eff. Emod (MPa)

20mm		Double seal
150mm	**103**	**G2**
150mm	**130**	**C4**
100mm	**155**	**G5**
150mm	**65**	
200mm	**135**	
200mm	**145**	
200mm	**160**	**G10**
200mm	**170**	
280mm	**190**	
v 1650		
Rigid	**50000**	

POSITION: Section 3 A :
(km 4.9 - 5.9 : Length 1.0 km)

	BEFORE Eff. Emod (MPa)	
20mm		Bit. Surface
150mm	42	EG5
100mm	44	G7
150mm	45	
200mm	70	
200mm	80	
200mm	150	G7
200mm	350	
280mm	500	
v 1500	Rigid 50000	

	AFTER Eff. Emod (MPa)	
20mm		Double seal
150mm	97	G2
150mm	175	C4
150mm	175	C4 / EG5
100mm	60	G7
150mm	54	
200mm	105	
200mm	165	
200mm	250	G7
200mm	450	
280mm	500	
v 1800	Rigid 50000	

POSITION: Section 3B1 :
(km 5.9 - 7.0 : Length 1.1 km)

	BEFORE Eff. Emod (MPa)	
20mm		Bit. Surface
150mm	30	G5
100mm	75	G7
150mm	75	
200mm	125	
200mm	165	
200mm	200	G7
200mm	260	
280mm	400	
v 1500	Rigid 50000	

	AFTER Eff. Emod (MPa)	
20mm		Double seal
125mm	68	G2
150mm	138	C4
150mm	138	C4 / EG5
100mm	55	G7
150mm	85	
200mm	130	
200mm	160	
200mm	180	G7
200mm	200	
280mm	300	
v 1775	Rigid 50000	

POSITION: Section 3 B2 :
(km 7.0 - 8.1 : Length 1.1 km)

	BEFORE Eff. Emod (MPa)	
20mm		Bit. Surface
150mm	35	EG5
100mm	55	G7
150mm	63	
200mm	190	
200mm	200	
200mm	140	G7
200mm	150	
280mm	200	
v 1500	Rigid 50000	

	AFTER Eff. Emod (MPa)	
20mm		Double seal
125mm	305	G2
150mm	315	C3/4
150mm	315	C3/4
100mm	90	G7
150mm	103	
200mm	112	
200mm	140	
200mm	180	G7
200mm	250	
280mm	425	
v 1775	Rigid 50000	

THE EFFECT OF REHABILITATION ON THE STIFFNESS OF PAVEMENT LAYERS

P W de Bruin and A T Visser

GIBB Africa, PO Box 35007, Menlopark, Pretoria, 0001, South Africa, pdbruin@gibb.co.za
Professor, Department of Civil Engineering, University of Pretoria, Pretoria, 0002,
avisser@postino.up.ac.za

CURRICULUM VITAE

PW de Bruin is an associate at ARCUS GIBB in the Pretoria office and is working the past seven years with Dr Gerrit Jordaan. He specialises in pavement management systems, pavement engineering and pavement rehabilitation. During the past seven years, he gained extensive experience in these fields in South Africa, Botswana, Zimbabwe, Ghana, Zambia and Kenya. He is currently involve in feasibility studies and detailed pavement rehabilitation designs, for World Bank and Danida projects in Africa. He was also recently involved with the flood damage assessment in Mpumalanga. He obtained his Master in Engineering in 1999 at the University of Pretoria under Professor Alex Visser