Flexible road pavement structural condition benchmark methodology incorporating structural condition indices derived from Falling Weight Deflectometer deflection bowls

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Abstract: A benchmark analysis methodology utilising the imbedded structural response knowledge of the whole deflection bowl measured with the Falling Weight Deflectometer (FWD) was developed for comparative evaluation of the structural condition of flexible pavement structures. Zones in the pavement structure are associated with structural condition via correlations with various slope parameters on the measured deflection bowl determined from simple spreadsheet calculations. Deflection bowl parameter benchmarking has found application at network level analysis with FWD data by a number of road authorities world-wide. A number of additional area parameters based on various areas under the deflection bowl have recently been added, These additional deflection bowl parameters were evaluated and found to strengthen the established pavement benchmark structural analyses as a preliminary evaluation tool. More recently it was illustrated that structural condition indices such as the internationally known Modified Structural Number (SNP) and the South African specific Pavement Number (PN) can be calculated from the full deflection bowls and can thus be used in such enhanced benchmark analyses of flexible pavement structures. The background to these FWD deflection bowl derived structural condition indices are briefly described. The benchmark methodology is illustrated via two specific examples to illustrate the identification of potential structural deficiency as well as possible origin of distress.

Keywords: Benchmark methodology, flexible pavements, falling weight deflectometer, deflection bowl parameters, benchmark analysis, structural condition index, Modified Structural Number, Pavement Number, slope parameters, area parameters and deflection bowl.

1. Introduction

1.1. Background to original deflection bowl parameter benchmark methodology

It is standard procedure to measure the deflection response of a road pavement structure with a Falling Weight Deflectometer (FWD. The deflection bowl is measured when a 40kN weight is dropped through a standard height onto a load plate with rubber cushions with an automated FWD. The FWD loading plate has a diameter of 300mm [1,2]. In Figure 1 the measuring set-up of the geophones of the FWD is illustrated. In South Africa, the typical FWD geophones are set up to measure deflection (D) at zero (D_0) (under the centre of the FWD loading plate), 200mm (D_{200}), 300mm (D_{300}), 450mm (D_{450}), 600mm (D_{600}), 900mm (D_{900}), 1200mm (D_{1200}), 1500mm (D_{1500}) and 1800mm (D_{1800}). The 40kN dropped weight represent the one half of a standard 80kN axle load of a truck. The deflection bowls, caused by the 40kN dropped weight (566kPa contact stress. , are measured at these discrete offsets representing the full half (due to symmetry) of the deflection bowl in the longitudinal direction of the road. These discrete measurement points on the deflection bowl allow simple spreadsheet calculations of deflection bowl parameters describing various zones or areas of the whole deflection bowl.

Figure 1 also illustrates three distinct zones of the deflection bowl, namely: the positive curvature zone close to the point of loading, the inflection curvature zone between 300mm and 600mm from the loading point centroid and the outer edges of the deflection bowl described as the negative curvature zone normally from approximately 600mm from the load centroid up to 2m away These zones illustrated in Figure 1 have been found to correlate very well with the structural response of specific layers or combination of structural layers vertically in the pavement structure [3,4]..

Table I presents a summary of the most common deflection bowl or basin parameters used in pavement structural evaluation [3,4,5,6]. It also indicates with which pavement structural layer combinations the various deflection bowl parameters have proven to be correlated best. The slope parameters, described in Table I as BLI, MLI and LLI (parameters 3, 4 and 5) formed the basis of the original benchmark methodology developed [2,3,4,5,6,7,8,9]. Additionally, the radius of curvature (RoC) parameter was subsequently proven to evaluate the structural response of the asphalt surfacing and top of the base layer. in the original benchmark methodology.

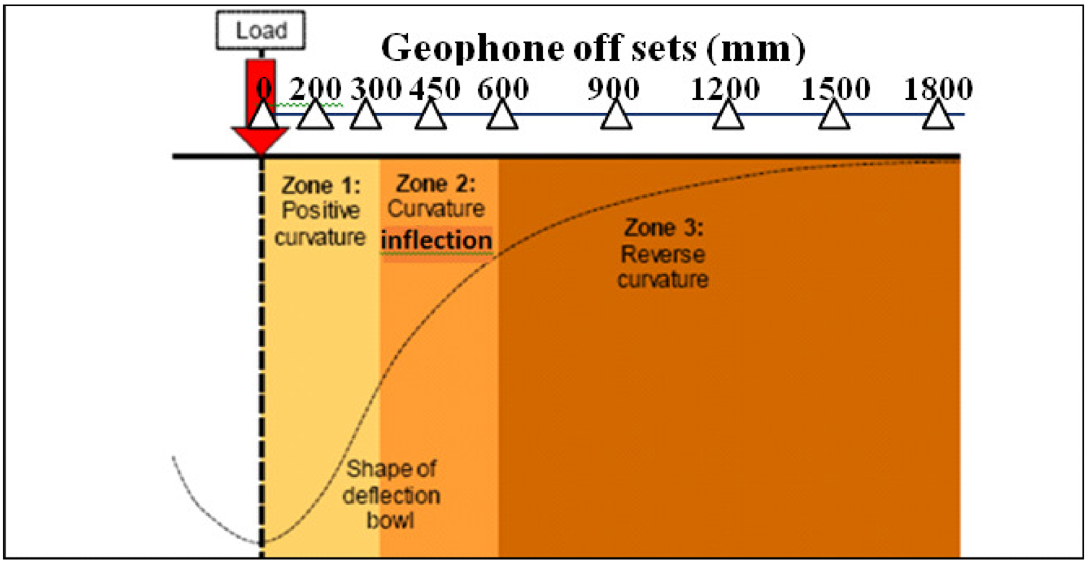

Figure 1. FWD deflection bowl illustration with measuring geophone set-up

Maximum deflection reflects the elastic response to loading of the whole pavement structure and has a long history in empirical structural response relationships with old equipment like the Benkelman Beam, where rebound deflections with plastic deformation elements included in the elastic rebound response were used. The FWD is able to measure mostly elastic response due to the dropped load simulating a moving wheel at approximately 60kph. As much as 60% to 70% of maximum deflection (D_0), measured with the FWD, can be due to the subgrade elastic response [8].

The deflection bowl parameters were originally based on in-depth studies [8] with an accelerated pavement testing device (Heavy Vehicle Simulator - HVS) and its adapted Benkelman Beam (Road Surface Deflectometer - RSD). These parameters were later adapted to FWD deflection bowls and standardised metric offsets as described above [3]. The recent addition of the Area Parameters or Indices (see parameters 8 to 10 in Table I) has proved to have value in benchmark analysis procedures [6].

TABLE I. DEFLECTION BOWL PARAMETERS [13]

Parameter	**Formula**	**Structural indicator and association with pavement zone**
1. Maximum deflection	D_0 as measured under the centre of the load	Gives an indication of all structural layers with about 70% contribution by the subgrade
2. Radius of Curvature (RoC)	$RoC = \frac{L^2}{2D_0[(D_0/D_{200}) - 1]}$ Where L=127 mm in the Dehlen curvature meter and 200 mm for the FWD	Gives an indication of the structural condition of the surfacing and top of the base condition
3. Base Layer Index (BLI) also known as Surface Curvature Index (SCI)	$SCI = BLI = D_0 - D_{300}$	Gives an indication of primarily the base layer structural condition

4. Middle Layer Index (MLI) also known as Base Damage Index (BDI)	$BDI = MLI = D_{300} - D_{600}$	Gives an indication of the subbase and probably selected layer structural condition
5. Lower Layer Index (LLI)) also known as Base Curvature Index (BCI)	$BCI = LLI = D_{600} - D_{900}$	Gives an indication of the lower structural layers like the selected and the subgrade layers
6. Shape factors	$F_1 = (D_0 - D_{600})/D_{300}$ $F_2 = (D_{300} - D_{900})/D_{600}$	*The F_2 shape factor seemed to give better correlations with subgrade moduli while F_1 gave weak correlations*
7. Additional shape factor	$F_3 = (D_{600} - D_{1200})/D_{900}$	*Lower layer condition or depth to a stiff layer*
8. Area under pavement profile	$A_{UPP} = \frac{(5D_0 - 2D_{300} - 2D_{600} - D_{900})}{2}$	*Characterizing condition of the pavement upper layers*
9. Additional areas	$A_2 = \frac{6\,[D_{300} + 2D_{450} + D_{600}]}{D_0}$ $A_3 = \frac{6\,[D_{600} + 2D_{900} + D_{1200}]}{D_0}$	*Condition of middle layer* *Condition of lower layers*
10. Area indices	$AI_1 = \frac{D_0 + D_{300}}{2D_0}$ $AI_2 = \frac{D_{300} + D_{600}}{2D_0}$ $AI_3 = \frac{D_{600} + D_{900}}{2D_0}$ $AI_4 = \frac{D_{900} + D_{1200}}{2D_0}$	*Condition of upper layer* *Condition of middle layer* *Condition of middle layer* *Condition of lower layer*

1.2 Background to Modified Structural Number (SNP)

The embedded knowledge of the whole deflection bowl can be further utilised to determine other structural indices with confidence [6, 13]. The first structural index is the well-known structural number (SN). The origin of the empirical structural number (SN) method is from the American Association of State Highway Officials (AASHO) road tests in the late 1950's. The SN method is described as an index methodology and has found use and application world-wide through the AASHTO design guide [14]. In the mid 1970's the UK Transport and Roads Research Laboratory (TRRL) defined modified structural number (SNC), which includes the effect of the subgrade [14,15,16]. The well-known Highway Design and Maintenance Standards Model (HDM) analysis tool [17,18] makes use of modified structural number (SNC), and more recently the adjusted structural number (SNP) determined in various ways in their latest software such as HDM-4 [19,20]. SNC and SNP are often used interchangeably, and in this paper SNP is preferred. The basic definition of modified structural number is:

$$SNP = SN + SNSG \quad (1)$$

Where SN is the structural contribution of the pavement layers above the subgrade and SNSG represents the contribution of the subgrade.

In the original calculations of SNP, knowledge of detailed material and pavement layer thicknesses were required, and correlation attempts with Benkelman Beam deflection showed that SNP and Benkelman Beam deflection are not directly interchangeable [1,7,8]. Now the Falling Weight Deflectometer (FWD) has taken over from the Benkelman Beam as the preferred non-destructive deflection measuring device [1,7,8]. The HDM-4

Technical Relationships Study (HTRS) on inclusion of FWD measured deflections into the model, evaluated six available procedures to calculate SNC or SNP.

Rhode [18, 19,20] developed a correlation of SN with deflections at D_0 and deflection measured at 1.5·*total pavement thickness (H_p), and the value of H_p. Furthermore, Rhode expressed the elastic modulus of the subgrade (E_{sg}) as a function of the deflections at offsets $1.5 \cdot H_p$ and $1.5 \cdot H_{p+450mm}$ and the value of H_p. In this approach, E_{sg} is related to equivalent California Bearing Ratio (CBR) using a relationship such as that suggested by Emery [21]. SNSG can be determined by means of the well published equation (2) [11, 17,19, 20] based on actual CBR laboratory values or CBR derived from FWD..

$$SNSG = 3.51(\log_{10} CBR) - 0.85(\log_{10} CBR)^2 - 1.43 \quad (2)$$

Based on the HTRS Study analysis, Rohde's relationships were recommended if FWD data and total pavement thickness data are available, whilst Jameson's formula was recommended if only FWD data is available. Jameson's formula uses maximum deflection and deflections at the 900 mm and 1500 mm offsets to determine the SNP components [19, 20, 21]. Subsequently, Salt and Stevens [22] developed a correlation limited to the same three sensor deflections used in Jameson's formulae as shown in equation (3). Although this correlation was developed for granular pavements in New Zealand, an improved correlation was obtained and SNP was formulated as a single relationship inclusive of the subgrade component, referred to here as SNP_{NZ} to distinguish it from other methods of determining SNP [22].

$$SNP_{nz} = 112(D_0)^{-0.5} + 47(D_0 - D_{900})^{-0.5} - 56(D_0 - D_{1500})^{-0.5} - 0.4 \quad (3)$$

A correlation study was done on a flexible pavement in South Africa with very detailed FWD survey and material test pit information [37]. This available data base was used to expand the use of the full deflection bowl and overcome the need for knowledge of pavement or layer thicknesses in the determination of SNP. This relationship was formulated as SN_{eff} and is shown in equation (4).

$$SN_{eff} = e^{5.12} BLI^{0.31} Aupp^{-0.78} \quad (4)$$

The Rohde deflection based methods of SN and SNP determination [17,18, 19,20] were used as reference due to the availability of reliable pavement thickness data, correct material classification and accurate laboratory determined subgrade California Bearing Ratio (CBR) values. SN and SNP were thus correlated with a variety of deflection bowl parameters, which utilised the deflection bowl more effectively by using the parameters or deflection bowl parameters representing the whole extent of the deflection bowl [23]. This correlation relationship, specific for South African flexible pavements, provided a non-destructive method to determine the effective structural number (SN_{eff}) for pavement layers as well as, the modified structural number inclusive of subgrade contribution (SNP_{eff}), for structural benchmarking applications [13].

The work done by Salt and Stevens [22] on a dataset of pavements in New Zealand was revisited. The correlation of SNP_{nz} with SNP_{eff} was very good for all types of flexible pavements [13]. A large database of various types of flexible pavements (granular, asphalt and cement base pavements) [24] was used in this correlation study. Equation (3) is preferred to be used to determine SNP_{eff} as it clearly incorporates the subgrade component of SNP_{eff} and due to its fuller use of the deflection bowl (up to 1500mm from load centroid)[13].

1.3 Background to Pavement Number (PN)

The Pavement Number (PN) was developed as a simple index methodology for structural design and evaluation of flexible pavements and making provision for pavements with bitumen stabilized materials (BSM) [25]. The South African Mechanistic Design Method (SAMDM) did not previously accommodate such materials adequately and their behaviour, including performance. A more simplified, robust design and analysis method was needed as the mechanistic empirical (ME) approach in the current SAMDM is regarded by some researchers and practitioners as too complex and would normally require various detailed material and layer thickness information, coupled with perceived questionable assumptions, extrapolations and simplification of data that lie hidden in the "darker recesses of the methodology" [256]. The SAMDM is under review and will probably become more complex, thus providing ample space for index-based methods to be used for benchmark or first-level analyses [28].

In this PN calculation the performance records of pavements [25,26] were incorporated by using a database including long term pavement performance (LTPP) data and accelerated pavement testing data via the South

African developed Heavy Vehicle Simulator (HVS) and the analysis of the catalogue of designs for flexible pavements [26, 27,28]. In all cases, the structural capacity was known with high certainty. Criteria were developed for the calibrated PN values thus determined to improve the certainty of the derived structural capacity from PN values. As such the method can be described as a knowledge-based method, or heuristic, design method that relies on established rules of thumb to guide a design process [25].

The rules of thumb used as departure points for the PN-based design method are briefly summarised as follows:

a) The subgrade material classification and known ranges of effective elastic modulus or stiffness values form the starting point of the design process.

b) Each layer material class, coupled with modular ratio (MR) of the layer stiffness and the supporting layer stiffness, is used to ensure stress sensitivity in unbound materials is thus addressed. Higher MR values are assigned to cohesive materials subject to fatigue.

c) The effective long term stiffness (ELTS) is determined for each layer, starting with the subgrade and linked to the MR limits prescribed. The subgrade ELTS in the PN model is determined by the material class, the climate and by the depth of cover over the subgrade.

d) The general method for determining the ELTS of pavement layers relies on the modular ratio limit and the maximum allowable stiffness. For these parameters, different values are assigned to different material types and were calibrated using the available knowledge base of pavement structural capacity.

e) The ELTS of a pavement layer is determined as the minimum of (a) the support stiffness multiplied by the material's modular ratio limit; and (b) the maximum allowable stiffness assigned to the material type.

f) Further refinement includes a Base Confidence Factor to ensure that inappropriate base types are not used or to prevent insensitivity to material placement which is the case in the traditional SN approach.

The PN values of the pavement structures in the extensive data base [24] were determined as part of the development of a pavement performance information system (PPIS) and were also used to validate the PN method [26]. The product of the ELTS (MPa) and layer thickness (mm) is divided by 10 000 to scale the PN_{calc} to a smaller one or two digit number similar to that of SNP. These PN_{calc} values were thus calculated from known material qualities, layer thicknesses and environmental conditions.

The proposed PN approximation by means of FWD deflection bowl measurement alone (PN_{eff}) makes use of two simple structural evaluation methods. The first method used is to convert the pavement layered structure above the subgrade to an ideal or theoretical equivalent elastic half space via [29] the Odemark approximation to equivalent layer thickness (H_e) equations [8,9,30,31]. The second method of approximation is to use Boussinesq's equations, as described by Ullidtz [38], to calculate the Surface Moduli values as "weighted mean modulus" of the idealised equivalent half space. These Surface Moduli (SM_i) values can be calculated at any offset, *i*, from the centre point of loading. The equivalent SM contribution of the pavement structure in total (SM_{pav}) can thus be determined by making a distinction of what the subgrade SM contribution is and subtracting it, or assuming the equivalent SM contribution of the pavement layered system is the same as for the subgrade as per the ideal elastic modulus half space.

Thus by converting the pavement structure in effect to a Boussinesq ideal elastic half space, the equivalent layer thickness (H_e) is multiplied with the SM_{pav} representing an approximation of a weighted ELTS of all pavement layers combined. In short the PN_{eff} thus determined by the product of the SM_{pav} and H_e is in effect a simple two layered pavement system of which the subgrade is the lower layer and the converted total pavement structure as similar theoretically idealised material on top [25] (See Figure 2).

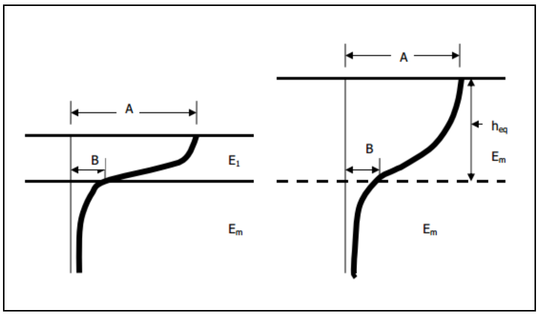

Figure 2. Odemark's equivalent layer thickness (h_{eq}) theory applied to equalise stress at subgrade interface for a two layer system [31]

The Odemark equivalent layer thickness can be calculated as follows:

$$H_e = a\, \Sigma^{L-1} h_i\, [E_i(1-\nu_s^2)/E_s(1-\nu_i^2)]^{1/3} \qquad (5)$$

Where H_e = Equivalent layer thickness (m)
a = constant ranging between 0.85 to 0.9 for flexible pavements
L = number of layers
h_i = individual layer i thickness (m)
E_i = elastic modulus of layer i (MPa)
E_s = elastic modulus of subgrade (MPa)
ν_s = Poisson ratio of the subgrade, normally assigned a value of 0.35
ν_i = Poisson ratio of layer i, normally assigned a value of 0.35 for granular materials and 0.44 for bituminous material.

However, H_e can also be determined from deflection bowl parameters based on previous correlation studies without knowledge of the pavement layer thicknesses of flexible pavement structures used in South Africa. The deflection bowl shape parameter F_1 (See Table I) was found [1, 7,8] to give the following correlation equation with H_e for flexible pavements irrespective of the subgrade modulus;

$$H_e = 10^{(\log F_1 + 0.268)/(-1.432)} \qquad (6)$$

The general formula for surface moduli (SM_i) determined by any deflection (D_i) at any point *i* (in mm) away from the point of maximum deflection (D_0) is shown in equation (7).

$$SM_i = \sigma_0 * (1-\upsilon^2) * (a^2)/(i * (D_i)) \qquad (7)$$

Where σ_0 = contact stress under the FWD loading plate (typically 566kPa contact stress for a 40kN drop weight)
υ = Poisson's ratio (usually set at 0.35)
a = radius of the loading plate (normally 150mm with diameter 300mm)
D_i = deflection taken at offset i from the loading plate centroid measured in micron.

Ullidtz [16] determined that the gradient of the SM further away from the point of maximum deflection (D_0) can be used to identify whether the subgrade has stress softening, stress hardening or purely linear elastic behaviour. The simple slope differential of the SM, or SMD (such as SMD= SM_{600}- SM_{900}), can be used to determine whether the subgrade response is stress stiffening, or stress softening. However it was found that SM_{300} gives the most consistent or representative value for the pavement structure [6]. Therefore the equation (8) can be used to determine PN_{eff} based purely on FWD deflection bowl information. The PN_{eff} values determined for a large data set of South African flexible pavement information correlated very well with calculated PN values (PN_{calc}) [6].

$$PN_{eff} = (SM_{300} * H_e)/10 \qquad (8)$$

1.4 Background to Structural Condition Index (SCI)

FWD surveys form the basis of most structural evaluation procedures by a variety of road authorities on a project level as well as in pavement management systems (PMSs) on the network level and the use of structural indices features strongly in these PMSs [22,33,34]. A good application of this approach is by the Texas DOT (TxDOT) in their Pavement Management Information System (PMIS). TxDOT developed FWD-based structural condition estimators, using primarily SNP and the Structural Condition Index (SCI) as screening tools in their PMIS for decisions regarding maintenance and rehabilitation.

The SCI is determined as follows:

$$SCI = SNP_{eff}/SNP_{req} \qquad (9)$$

Where:
SCI= Structural Condition Index
SNP_{eff} = existing Structural Number
SNP_{req} = required Structural Number typically calculated as needed for the next 20 years based on known material qualities and thicknesses.

Zhang et al. [34] states: '*Because of the simplicity of the SCI, the interpretation of its meaning is straightforward. An SCI greater than one would indicate a sound pavement structure while SCI less than one* *means the pavement is no longer structurally adequate*.' The original use of SCI by Texas DoT was clearly on a network level basis to identify project level sections which may need rehabilitation and more detailed level investigation. It is therefore also an ideal tool on a project level as a preliminary evaluation. It is anticipated that SCI can also be determined by means of the $PN_{eff}/PN_{design\ or\ required}$ for South African flexible road pavements.

2. Benchmark Parameter Condition Ranges

The well-known RAG condition rating system, often applied in pavement management system (PMS) and pavement condition ratings, was originally utilised as a simplified deflection bowl parameter benchmark evaluation method. RAG represents red for severe condition, amber for warning condition and green for sound condition. The criteria for the RAG relative structural condition states are based on a semi-empirical model which provides for accurate benchmark or relative evaluation of pavement structural capability [1,3,4,5,10, 35,36]. Over the years these ranges were adapted and new parameters added. They were used on flexible road and airport pavements and have lately been updated in a review study [6].

Revised RAG ranges of an expanded and updated FWD derived deflection bowl parameters for roads only are suggested for granular base pavements in Tables II and III. Granular base pavements are the pavement type most commonly used in South Africa. It is important to note that these parameters are biased towards the more flexible ranges of elastic response and not towards the very stiff or rigid pavements or upper traffic ranges [1, 36] shown in Table II. Stiff to very stiff or rigid pavements will in any case generally not be in need of rehabilitation and therefore the focus and discrimination is set for the more flexible elastic response of flexible pavements. The newer set of deflection bowl derived structural indicators and their suggested RAG ranges are included in Table III [6].

These ranges can be adjusted if the focus would be on stiffer pavements to achieve sensitivity, greater definition and discrimination on a benchmark evaluation basis. The newer area parameters are therefore also given initial ranges for granular base pavements and should be checked and adjusted when different flexible base type pavements are evaluated with the benchmark analysis methodology.

It is reiterated that the benchmark methodology is aimed at first order or preliminary relative structural analyses to help guide further more detailed analyses cost effectively. In spite of the traffic ranges given in Table II it should not be used for remaining life calculations, but rather seen as an indication of linked ball park of residual life and associated pavement condition for a flexible pavement. The basis for this was the original empirical relationships for maximum deflection in the transitional stage from Benkelman Beam deflections to FWD derived deflection. Therefore, it is only a relative strength indication and nothing more than that.

TABLE II. CLASSICAL BENCHMARK RANGES FOR 566 KPA CONTACT STRESS (40KN)

Original rating of flexible pavements		**Deflection bowl parameter benchmark analysis ranges and elastic response classification**				
Elastic response classification	**Traffic Range MESA**	**Structural condition rating**	**Maximum deflection (micron)**	**BLI (micron)**	**MLI (micron)**	**LLI (micron)**
Firm to very stiff/rigid	>3	Sound	<500	<200	<100	<50
Firm to flexible	0.8 to 3	Warning	500 to 750	200 to 400	100 to 200	50 to 100
Flexible to very flexible	<0.8	Severe	>750	>400	>200	>100

TABLE III. ADDITIONAL BENCHMARK RANGES FOR 566KPA CONTACT STRESS (40KN)

Structural condition rating	**Deflection bowl parameter benchmark analysis ranges**					
	SNP_{eff}	**PN_{eff}**	**SCI (Structural Condition Index)**	**AI_1 Area parameter**	**A_{upp} Area Parameter**	**RoC_{200} (m)**
Sound	>6	>10	>1	<20 000	<400	>100
Warning	4 to 6	5 to 10	0.75 to 1	20 000 to 30 000	400 to 800	50 to 100
Severe	<4	<5	<0.750	>30 000	>800	<50

3. Demonstration of Benchmark Analysis with Structural Indices

3.1. Benchmark analysis with SNPeff and SCI for a road with premature surface failure

The use of SNP_{eff} derived from deflection bowl information, and then used to calculate SCI, is illustrated by using a well documented road pavement where premature failure in the top of the high quality freshly crushed continuous graded granular base and the 40mm continuous grade asphalt surfacing with 20mm ultra-thin friction course (UTFC) layers occurred. The rest of the pavement had a cement treated subbase and well-designed and constructed selected subgrade on good quality subgrade. Detailed test pits and laboratory surveys were followed by back-analysis of effective elastic moduli, which confirmed the source of distress as originating from a combination of the top of base and surfacing. Benchmark analyses with parameters LLI and MLI (not shown) also confirmed sound subbase and subgrade structural strength condition.

Visual distress in the form of advanced crocodile cracking at isolated spots in the outer wheel tracks was observed. The standard 40kN dropped weight FWD survey was done at 10m intervals in the slow lane in both wheel paths making it ideal for detailed survey analysis. In Figure 3 the FWD (40kN) maximum deflection is shown and via the RAG benchmark system illustrates that no structural problem can be detected by using maximum deflection alone. There is a spot (km 13.06) where maximum deflection is larger than the rest in the left wheel path (LWP), but this does not show up as being in a warning or severe condition; it is in fact the spot where the visual distress had been detected over this section of road.

The SNP_{eff} values for the same stretch of road are shown in Figure 4. In this case the RAG ranges set in Table III identify the visually distressed spot as being in a warning condition. This is heartening as it means that SNP_{eff} can pick up a structural defect like this known to originate from the surface and top of base layer. Even though this is more informative than looking at maximum deflection alone, it still cannot identify origin of distress. SNP_{eff} on its own is already a valuable structural index value, but Salt and Stevens [36] correctly states "*Therefore, SNP is not able to give any indication of how a particular pavement structure would behave for a given layer configuration. For example, a road consisting of a stabilised base on top of inferior material may have a high SNP, but would in fact fail rather quickly due to cracking of the base layer*."

The designed SNP_{req} could be calculated accurately as $SNP_{req} = 8$ from as-built and test pit information. This was calculated with inclusion of the known subgrade contribution expressed in CBR as material properties and layer thicknesses were well documented. Therefore, SCI as per equation (9) could be calculated with confidence. In Figure 5 the SCI benchmark analysis for the same stretch of road is shown. The RAG limits in Figure 5 are indicated and are as listed in Table III. As can be seen it is in the left wheel track that distress can be identified as visually observed and the sections in close proximity is now also showing a warning condition. The right wheel track showed no visual distress yet and therefore the SCI values in the right wheel track also show a sound structural condition.

This is already a much better indication of distress and possible pending distress than what maximum deflection or SNP_{eff} could indicate. Even though areas with possible distress were thus identified, it was confirmed that SCI or SNP_{eff} alone cannot indicate where in the pavement structure the problem may originate. Further deflection bowl benchmark analysis was thus needed to identify the origin of distress.

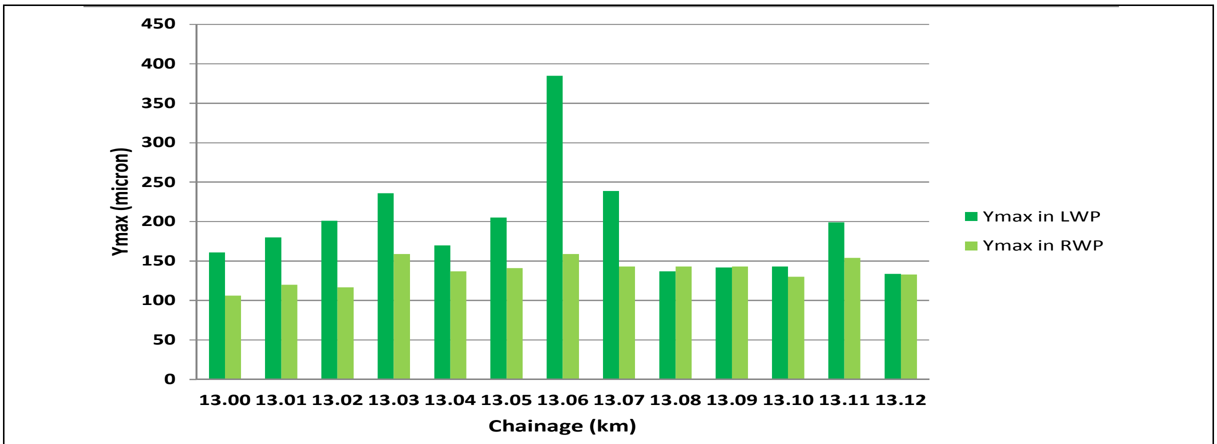

Figure 3. Maximum deflection benchmark analysis for road section with premature visual distress

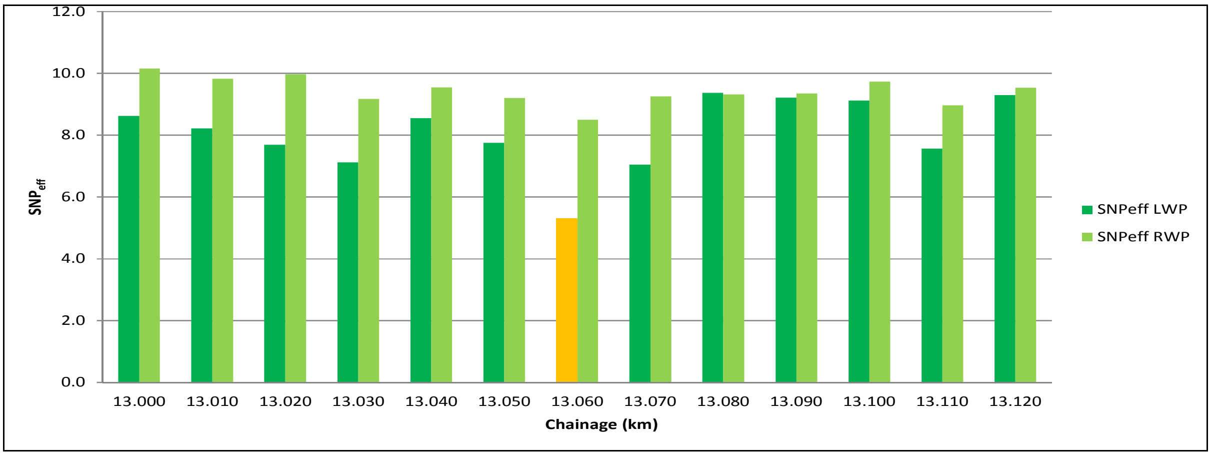

Figure 4. SNP$_{eff}$ benchmark analysis of road with visual premature distress

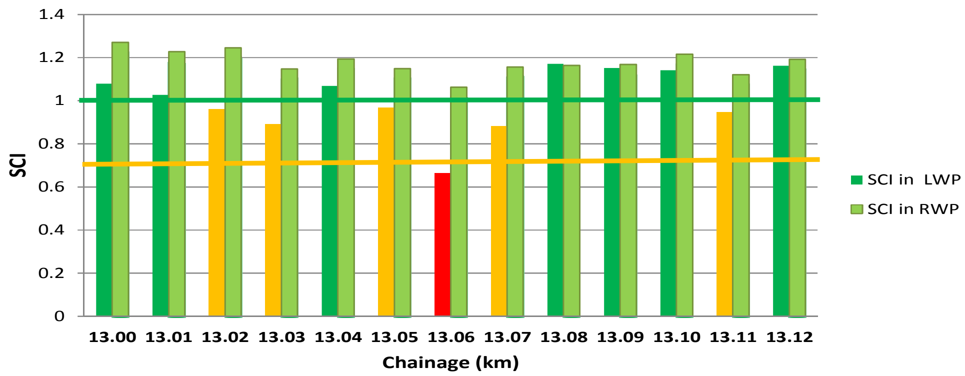

Figure 5. SCI benchmark analysis for a distressed road section

As stated before LLI and MLI benchmark analysis (not shown) confirmed the detailed back analysis and test pit observations that no structural deficiencies occurred in these lower structural support layers. In Figure 6, the BLI benchmark analysis is shown. The visually confirmed distressed spot is again identified indicating the base and surfacing combination or zone is in a warning condition at that spot. This spot coincides with the spot identified in a severe condition by the SCI benchmark analysis and the SNP$_{eff}$ in the warning condition.

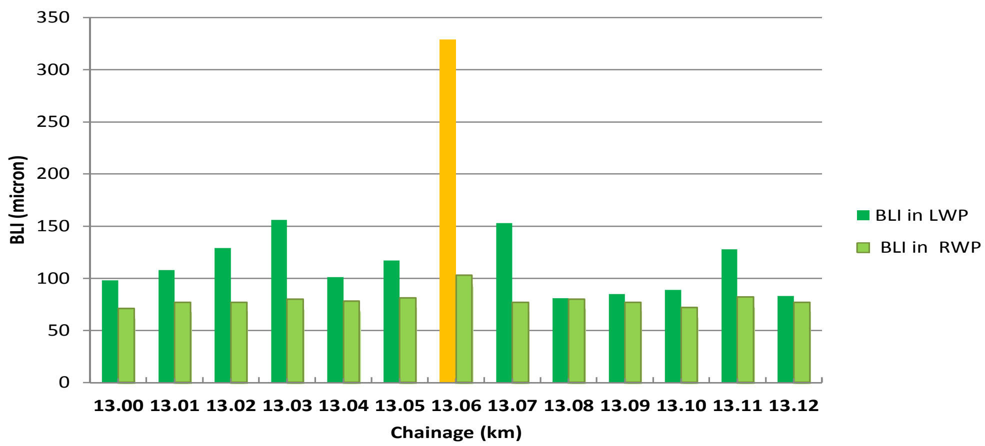

Figure 6. BLI benchmarking analysis to identify origin of distress

No further spots in warning were identified with the BLI benchmark analysis. This may indeed be an indication that the possible distress is confined to the top of the base layer and surfacing as the combination of base in total and the asphalt surfacing elsewhere shows no warning or distress as alluded to by the SCI analysis before. A further analysis with RoC_{200} shown in Figure 7 enabled a more detailed analysis of the nature of the origin of distress as this deflection bowl parameter is known to correlate with the surfacing and upper portion of the base layer. This RoC_{200} benchmark analysis was able to identify areas where the severe RoC_{200} coincided with identified visual surveyed severe conditions confirming premature distress in the asphalt surfacing and the top of the crushed stone base. Over and above the spot in a severe condition other potential problems in a warning condition also in the left wheel track could now be observed. Of significance is the fact that the right wheel track, next to the identified severe spot in the left wheel track, also now shows potential problems signaling RoC_{200} values close to or in the warning condition.

The mechanism of distress was identified as water infiltration via a porous asphalt surfacing after seasonal rain. Highly channelized traffic caused water accumulation at the top of base and underside of the asphalt surfacing to be exposed to excessive pore water pressure (EPWP). This EPWP thus caused longitudinal as well as transverse growth of the asphalt surface de-bonding. This de-bonding could also be confirmed by listening to the differential hollow sound when tapped with hammer. This potential for cracking extended up to 1m from the original area of distress in the wheel path. Cracking on de-bonded areas only showed up much later after additional seasonal rain, but growth was limited after removal of restrictive traffic accommodation measures (lane closures with highly channelized traffic) to normal traffic flow coinciding also with a shifted transverse normal distribution wheel path. Mechanistic analyses confirmed that the asphalt fatigue life is reduced with as much as 90% if there is no interlayer grip left in such a de-bonded condition and confirming the restriction of fatigue related cracks (crocodile) with no rut deformation observable. Therefore the RoC_{200} parameter here was able to indicate that other areas may be prone to premature cracking if the channelized traffic situation was to continue. This was confirmed by subsequent repair and maintenance actions once the pavement was exposed to a normal multilane traffic situation

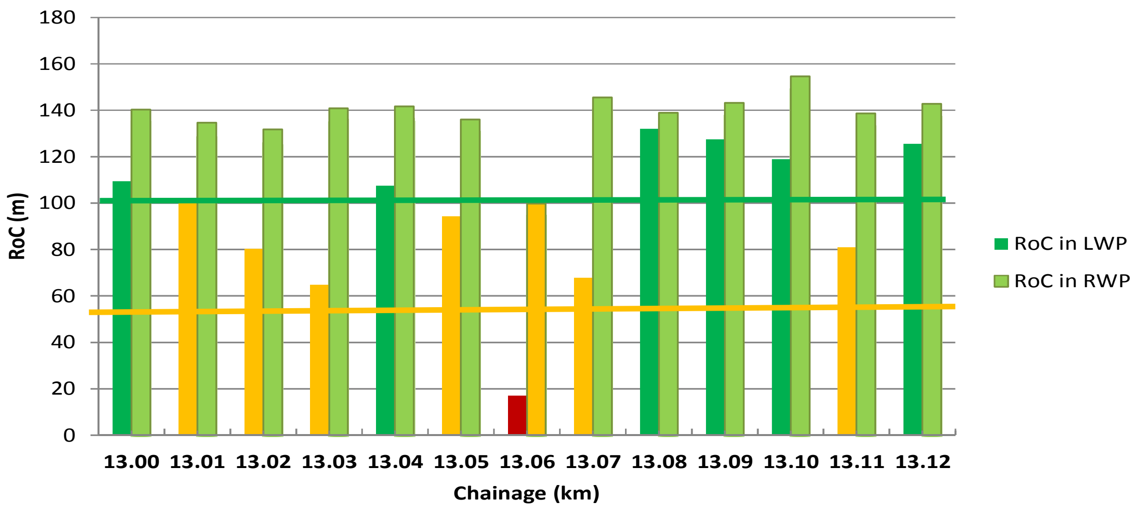

Figure 7. RoC_{200} benchmark analysis for short distressed road section

3.2 Benchmark analysis with PN_{eff} for a very flexible road

In Figure 8 the PN_{eff} values determined from FWD survey data on a short sample road with a light pavement structure recently analysed in Gauteng Province is shown versus distance. It is clear that sections at the start of the road length is in distress and interspersed short sections all along the road. However, the cause or origin of possible distress cannot be derived from this Figure 8. This first level benchmark analysis clearly and correctly identifies uniform sections of the road which should be treated differently in terms of rehabilitation needs. It can identify possible areas of structural inadequacies, but PN_{eff} benchmark analyses cannot identify the origin of distress within the pavement structure.

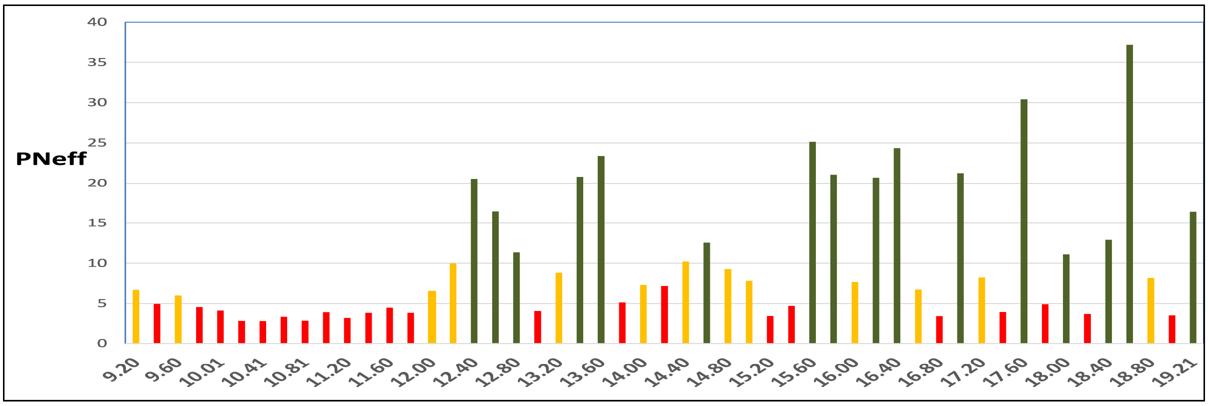

Figure 8. PN_{eff} benchmark analysis for a short road section in Gauteng

In order to complement the PN_{eff} benchmark analysis as a preliminary structural analysis, a more detailed deflection bowl analysis is needed [37] where the embedded structural response associated with the whole deflection bowl is unlocked. In Figure 9 the LLI benchmark analysis shows that the origin of distress is not in the selected and subgrade layers as the whole section is sound and in the green. Therefore the origin of distress must be in the layers on top of the subgrade and selected layers.

In Figure 10 the middle layer index (MLI) benchmark analysis is shown. It identifies severe and warning areas which are very well correlated with the PN_{eff} benchmark analysis. The benchmark analysis of the base layer index (BLI) is shown in Figure 11. BLI is describing the structural condition of the base and surfacing combination. If it is viewed in conjunction with the MLI graph in Figure 10 it shows that much the same areas and zones in warning and severe structural condition persist. It also further coincides remarkably with the PN_{eff} areas in warning and distress. This implies the origin of distress is largely in the subbase and therefore affecting the base and surfacing layer combination as the base layer did not 'bridge" or improve the subbase weaknesses, but rather reflected it through due to the lack of support. Thus by enhancing the PN_{eff} analysis with other deflection bowl benchmark analyses the main cause or origin of distress could be identified. In this case the road surface was also highly cracked and fatigued and therefore the RoC_{200} (not shown) only confirmed the visually distressed state of the surfacing as well.

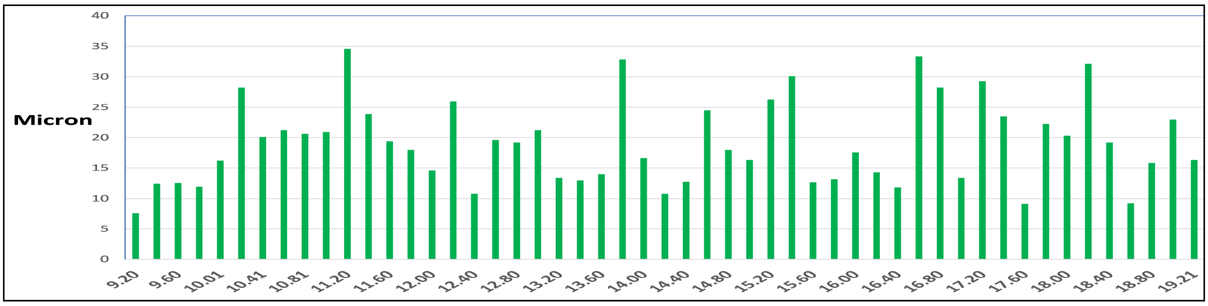

Figure 9. Lower layer index (LLI) benchmark analysis for a short road section in Gauteng

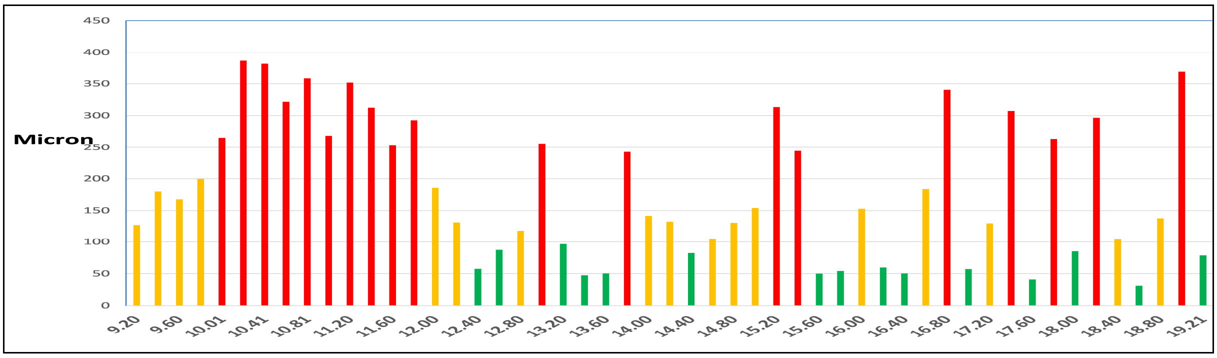

Figure 10. Middle layer index (MLI) benchmark analysis for a short road section in Gauteng

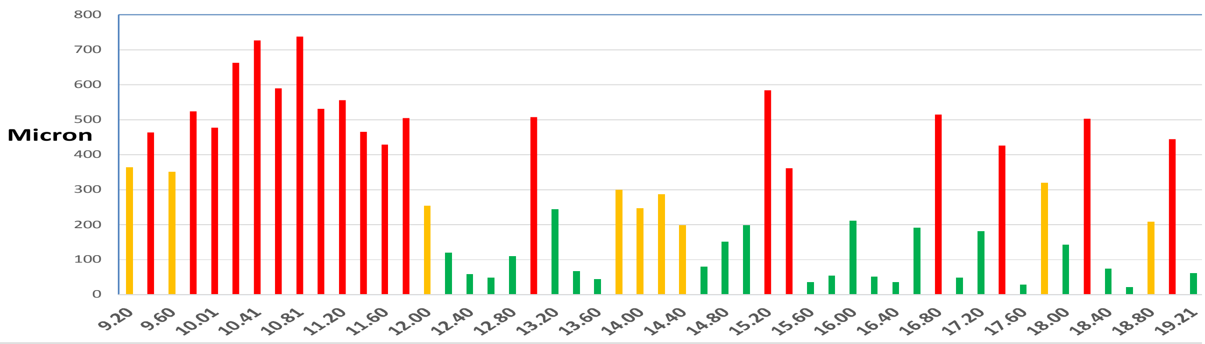

Figure 11. Base layer index (BLI) benchmark analysis for a short road section in Gauteng

4. Conclusions

Benchmark analysis with deflection bowl parameters representing various zones on the FWD determined deflection bowls has a proven track record as a preliminary method for pavement structural analysis. The deflection bowl has significant embedded knowledge which is under-utilized if only one point or two discrete points on the whole deflection bowl is used in structural evaluation of pavement structures. The various slope parameters and radius of curvature parameter have proven that they collectively can represent the whole deflection bowl and therefore reflect information of the structural response and the relative structural condition of layers and layer combinations in depth of the pavement.

The deflection bowl based benchmark analysis methodology was enhanced by determining other structural indices such as structural number (SN_{eff}), the adjusted structural number (SNP_{eff}) and Pavement Number (PN_{eff}) directly from the whole deflection bowl. Such novel calculations of these structural indices are based on correlation equations developed from a large South African data base of various pavement types. This enables typically preliminary structural evaluation before any detailed material type and layer thickness information is available.

Structural Condition Index (SCI) values can also be calculated via SNP_{eff} and SNP as for design or as required (SNP_{req}). This indicator can also be used in a benchmark methodology. As illustrated via specific examples, SCI, SNP_{eff} and PN_{eff} can provide a preliminary analysis level structural evaluation of the road section analysed. It has obvious value at project level investigations and obviously application at network level PMS management of the road network.

The value of this effective unlocking of the embedded knowledge of the full FWD deflection bowl has been illustrated via benchmark analysis of two different flexible road sections with different forms of distress. The latter was first identified via visual surveys. The use of the benchmark analysis via structural indices (SNP_{eff}, SCI and PN_{eff}) confirmed the visual survey information. However, the origin of distress could only be identified with a further deflection bowl benchmark analysis. This ability to "drill down" with the benchmark analysis methodology allows for more effective detail analyses to follow.

It is recommended that SNP_{eff}, SCI and PN_{eff} be complemented with the well known deflection bowl parameter structural benchmark methodology. These simple to calculate, mostly slope parameters and radius of curvature deflection bowl parameter values, provide for an effective three tiered relative structural condition rating. Various zones and combinations of layers can thus be identified which may be the origin of distress.

No further detailed analyses like structural life predictions are performed because these benchmark analyses methods are intended only to be used as preliminary screening tools, to help guide more detailed investigations and analyses. It does however enable more analysis potential and unlocking of embedded knowledge of the deflection bowl in full. This has relevance on the network level with pavement management systems as well as on project level investigations.

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