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Improvement of the Algorithm for Computing Adjusted Structure Number for Determination of Backlog Maintenance

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Abstract

The pavement strength is very important for the evaluation of backlog maintenance. The current trend in many developing countries used pavement conditions index-PCI in estimating maintenance costs. The PCI can only justify periodic and routine recurrent maintenance. The condition strength is rarely determined in a flexible pavement creating an opportunity for back long maintenance. This paper reports the study conducted to develop and improve the algorithm for estimating the adjusted structure number to estimate the remaining thickness of the flexible pavement. The analysis of eight ways of computing structure numbers from FWD data ware analyzed and found that the improvement of the HDM 3 - 4 models can influence the usefulness of data collected from road asset management in Tanzania. The algorithm for estimating structural numbers from CBR was improved to compute adjusted structural numbers finally used to estimate the remaining life of the flexible pavement. The analysis of the network of about 6900 km including ST and AM was found that 64.72% were very good, 12% were Good, 10% were fair and 7.84% were poor and 5.4% were very poor.

Keywords

Falling Weight Reflectometers, Pavement Condition Index, Structure Number, Adjusted Pavement Number, Deflection Basin Parameters, Pavement Modulus, Structural Number

1. Introduction

Falling Weight Reflectometers (FWDs) have been in use since the 1980s. The FWD testing is a nondestructive pavement structural evaluation technique routinely performed on highway and airfield pavements to estimate pavement layer properties from measured deflection basins. The FWD was first developed to measure pavement surface deflection in airports, due to aircraft loading while moving at intermediate speeds [1]. Even though these devices can be used to evaluate the structural capacity of both rigid and flexible pavements, the testing method with FWD and the analysis of data obtained from testing on both pavement types are usually performed differently. The reason behind this is that the physical structure and the in-service behavior of both pavement types are significantly different [2]. The good side of FWD is that it is a nondestructive method [3]. It is understood that Pavement Strength information is important in the evaluation of pavement for planning and ascertaining maintenance strategies as well as reporting network conditions [1]. FWD data is required whenever a structural pavement overlay design is required, and California Bearing Ratio (CBR) or Resilient Modulus (RM) data are not available. Furthermore, FWD data are required for Concrete Pavement Restoration (CPR) projects, overlays of existing concrete pavements, determining the amount of required patching as well as determining the location of full-depth repairs, dowel bar retrofits, and sub-sealing [4]. Unfortunately, the algorithm in the Road Maintainance management system (RMMS) estimates the road condition using structural numbers computed from CBR, resulting in many faults in identifying pavement failure [5] [6].

It is asserted by [7] that in a sense all pavement failures are functional failures, accessing failure categories makes the understanding of a failure somewhat easier. They further elaborate that failures may be categorized as structural, functional, or materials failures. The failure can also result from subgrade, sub-base cause, base cause, and surface course [8]. Certainly, these categories may overlap. Structural failure may be defined as the loss of load-carrying capability of the pavement section resulting in the need for significant repair or replacement [9]. A functional failure is a broader term, which may include the loss of any function of the pavement such as skid resistance, structural capacity, and serviceability or passenger comfort [10]. A materials failure is the disintegration or loss of material characteristics of any of the component materials [11]. Early indications of pavement failure are not always available [7]. It is claimed in [12] that pavement and premature failure, as well as distress, continue to happen despite significant advancements in pavement technology. The root causes of this failure need to be identified. Some failures may be due to exposing the road to traffic which was not designed for. Others are due to road settlement, flexural cracks, as well as weather of the road network. However, the mechanics of road failure is quite complex and it is tedious to identify the root cause [13].

Nevertheless, the result of pavement failure is not a catastrophe like the collapse of a building or dam. But they represent a serious financial loss as well as planned strategies of maintenance either for periodic or rehabilitation [14].

This article is organized as follows. Section 1.1 justifies the problem and Section 2 gives details of the related works. Section 3 gives details of the method of collecting data with FWD. The result is provided in Section 4 and the key contribution of this paper is found in Section 4.7. Finally, Section 4.8 provides an analysis of the data resulting from the improved algorithm.

Current Practice and Estimation of Condition

The motivation for this study is driven by practice of road asset management in the country. The RMMS planning and estimation of the cost of maintenance is very highly appreciated and adopted by national strategies in road maintenance for trunk and regional roads. However, the system has some shortfalls in estimating maintenance needs described as follows. With RMMS conditions of the pavement are assessed using cracking, rutting, roughness, patching, and distress data. The system does not use actual pavement strength in the computation of maintenance costs for rehabilitation. The planning is therefore far less from optimum because they do not capture the actual pavement strength in the estimation of the maintenances for rehabilitation. This study intended to include structure numbers obtained from FWD data into RMMS and avoid the use of hypothetical structural numbers in the computation of Rehabilitation costs.

2. Related Works

2.1. History and Practice of SN, SNC, and SNP

The origin of the empirical structural number (SN) method is from the American Association of State Highway Officials (AASHO) road tests in the late 1950s [14] [15]. The SN method is described as an index methodology and has found its use and application worldwide through the AASHTO design guide [14]. In the mid-1970s the Transport and Roads Research Laboratory (TRRL) establish and defined the modified structural number (SNC), which includes the effect of the subgrade [16]. Typically the well-known Highway Design and Maintenance Standards Model (HDM) analysis tool 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 [17].

In a sense, the Structural Number is taking some critical design information – such as material properties, traffic loads, and pavement performance criteria – that exists in very different forms and boiling them down into a single number. Doing this is what makes empirical asphalt pavement design possible, so the Structural Number is a key to the entire process of designing a quality pavement (Pavement Interactive, 2014). In general, the structural condition of pavement can be expressed through three major characteristic values named Deflection Basin parameters, Pavement modulus; and Structural Number.

2.2. Maintenance Interventions

The pavement condition has a high coloration with the required maintenance intervention. Pavement in poor to very poor would normally require rehabilitation or reconstruction. **Figure 1** shows the relationship between pavement age

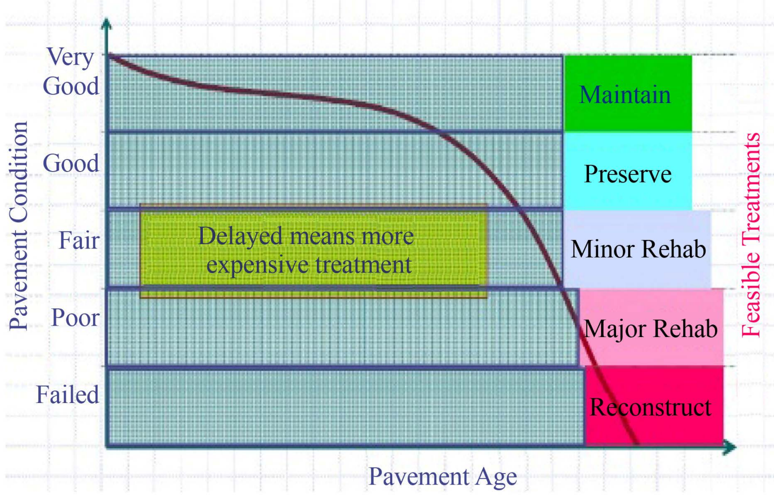


Figure 1. Pavement vs maintenance needs.

and pavement condition investment. The probability of failure can therefore be computed based on the age and Pavement type [18].

2.3. Methods for Estimating the Structural Capacity of the Pavement

There have been numerous studies to find simple methods for calculating the SNP parameter or index using either destructive or non-destructive tests [14]. In this study, we aim on using data collected by FWD as a non-destructive testing device to fully utilize the whole measured deflection. The literature studies also showed that there are eight methods of determining the SN as described in the section below.

2.4. Description of the Structural Number

A structural number is used as an indicator of pavement strength in many pavement designs and deterioration models [19]. The required Structural Number depends on a combination of existing soil support, total traffic loads, pavement serviceability, and environmental conditions [19] [20]. The SN can be deduced from the design equation bellow

$$\log_{10}(W_{18}) = Z_r \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)5.19}} 2.32 \times \log_{10}(M_r) - 8.07$$

Although design equations can be used in different ways depending on the inputs available, one of their most common applications is effectively solving for the Structural Number. Once the value of the Structural Number is known it can be used to determine appropriate thicknesses for each of the pavement layers.

The Structural Number is a value that applies to the overall pavement structure. Structural capacity can therefore be defined as the ability of the pavement to carry traffic loads over its service life forgiven subgrade support and environmental conditions [21]. However individual thickness need to be computed from each deflection presented by question bellow [22].

$$SN = \left(a_1 D_1 M_1 + a_2 D_2 M_2 + a_1 D_1 M_2\right) + \cdots$$

2.5. Deferent Approach of Determining Structural Number

1) Rhode Method for SIP

The Rhodes study and explores extensively the computation of Structural Index Pavement-SIP [21] [23]. Rhodes shows that the peak deflection measured below on the FWD loading plate is a combination of the deflection in the subgrade and the elastic compression of the pavement structure. As cited in [15] Irwin suggested a "two-third" rule based on the fact that 95 percent of the deflections measured on the surface of a pavement originate below a line deviating 34 degrees from the horizontal. However, Rhode concluded that, with this simplification, the surface deflection measured at an offset of 1.5 times the pavement thickness originates entirely in the pavement subgrade. By comparing this deflection value with the peak deflection under the loading plate, the Structural Index of a Pavement (SIP) could be defined as follows:

$$SIP = D_0 - D_1 * 5HP$$

Which relate to the question

$$SN = k_1 SIP^{k_2} HP^{k_3}$$

where:

SIP = structural index of pavement.

D_0 = peak deflection measured under a standard 9000-lb FWD load.

$D_1 \cdot 5Hp$ = surface deflection measured at an offset of 1.5 times of Hp under a standard 9000-lb FWD load.

Hp = total pavement thickness.

SN = pavement structural number.

SIP = structural index of pavement (microns).

k_1, k_2, k_3 = regression coefficients.

2) Wimsatt Method

The Wimsatt method [24] is focusing on the assessment of the modulus of the pavement structure as a whole concerning the subgrade modulus using the ratio of $W7$ to $W1$ ($W7/W1$), which is the ratio of pavement modulus to subgrade modulus. The deflection underneath the loading plate ($W1$) gives the stiffness of the pavement and the subgrade; whereas, the deflection 72 inches away from the plate ($W7$) gives the stiffness of the subgrade only. Wimsatt developed a regression equation for calculating pavement to subgrade modulus ratio ($Ep/Esubgrade$). Such equations are a function of the $W7/W1$ ratio and the subgrade modulus ($Esubgrade$). The subgrade modulus proposed by Wimsatt is adopted from AASHTO Guide for Design of Pavement Structures [25] and is described as follows

$$ESubgrade = \frac{0.192 \times P}{W7} \times 72 \text{ in}$$

where:

Esubgrade is = is the back-calculated subgrade resilient.

P modulus = is the applied load in pounds.

W7 = is the deflection at sensor 7 in mils.

From the above equation the pavement to subgrade modulus ratio regression equation for 21-inches pavement is presented as follows:

$$Ep/ESubgrade = \frac{(516.94 \times w_7) \times 5/2}{W1} - \frac{(214.46 \times w_7) \times 2}{W1} + \frac{(159.56 \times w_7) \times 5/2}{W1} - \frac{6.143 \times w_7}{W1} + \frac{(1.0826 \times w_7) \times 1/2}{W1}$$

where:

(*Ep*/*Esubgrade*) is the pavement to subgrade modulus ratio. *W*1 = is the deflection at sensor 1.

From this relationship, *Ep* can be calculated.

Once the pavement modulus is determined, the relationship given in the AASHTO Guide for Design of Pavement Structures can be deployed to calculate the effective Structural Number denoted as follows

$$SN_{eff} = 0.0045 \times D_0 \times EP$$

where:

D = is the total thickness of the pavement layers.

Ep = is the existing pavement modulus of all layers above the subgrade.

3) HDM-4 Method for SNP

The Highway Development and Management Model (HDM-4) is a software system for evaluating options for investing in road transport infrastructure. Worldwide, the HDM-4 model is most commonly used as a basis for feasibility studies, in which a road project is evaluated in terms of its economic viability. Its capacity in computing SN is explained in the relationships that HDM-4 uses to convert Benkelman beam deflections (*DEF*) to *SNP* values. Described as follows:

Cemented base:

$$SNP = 3.2\left(DEF^{-0.63} + DSNPKb\right)$$

For Not Cemented Base the coefficient remains 2.2 described as:

$$SNP = 2.2\left(DEF^{-0.63} + DSNPKb\right)$$

For FWD Deflections the central deflection at 700 kPa is used as the equivalent Benkelman beam deflections. The FWD deflection will need to be multiplied by 1.25 and apply the above formula to find the adjusted structural number due to cracking.

$$dSNPK = 0.000758\left\{MIN\left(63, ACX_a\right)HSNEW + MAX\left[MIN\left(ACX_a - PACX, 40\right), 0\right]HSOLD\right\}$$

where:

DEF: Benkelman beam rebound deflection under 80 kN axle load, 520 Kpa Tyre pressure, and 30˚C average asphalt temperature for the season (mm).

dSNPK: Reduction in adjusted structural number due to cracking.

ACX_a: Area of indexed cracking at the start of the analysis year (%oftotal carriageway area.

HSNEW: The thickness of the most recent surface (mm).

PACX: Ara of the previous Index creaking in old surface.

HSOLD: The total thickness of previous Underlying Surfacing layers(mm).

4) New Zealand Method for SNPnz

A correlation study reported by [26] showed greater promise when well-documented New Zealand unbound granular pavements were used to calculate SN [27]. This SNP correlation was based on deflection points at 0, 900 mm and 1500 mm of the measured FWD deflection bowl under standard 40 kN dropped weight. The derived *SNP* or *SNC* values were provided by the following equation:

$$SNP_{nz} = 112(D0)^{0.5} + 47(D0 - D900)^{0.5} - 56(D0 - D1500)^{0.5} - 0.4$$

where:

SNP_{nz} *is the SNP or SNC value determined for New Zealand unbound granular pavements.*

$D0$, $D900$, *and* $D1500$ *are deflections in microns at offsets* 0, 900, *and* 1500 *mm, respectively, under the standardized* 40 *kN FWD impact load.*

5) Schnoor and Horak Method

Structural Number (SN) is explained as an index that indicates the strength of the pavement layers and the total pavement structure. Such an empirical approach is derived by taking the layer material type-specific coefficient multiplied by the layer thickness and the sum of these are then called pavement Structural Numbers [15]. They found that a road network with detailed layer thickness, material classification based on extensive test pit and laboratory testing, and detailed FWD testing is used to correlate deflection bowl parameters with SNP. A number of these deflection bowl parameters correlate very well individually with SNP via a stepwise multiple regression procedure, where the deflection bowl are utilized more effectively with the following derived regression equation (R^2 = 0.98);

$$SNeff = e^{5.12} * BL^{0.31} * AUPPL^{-0.78}$$

where:

SNeff: is the effective SNP at the time of measurement.

e: is the natural logarithm.

BLI: is the slope parameter determined by the difference between $D0$ and $D300$.

AUPP: is also determined by simple spreadsheet calculation with the formula based on deflections measured at 0, 300, 600, and 900 mm respectively.

BLI: is *D*0-*D*300.

$AUPP = (5D0 - 2D300 - 2D600 - D900)/2$:

*D*0, *D*200, *D*300, *D*600, and *D*900 are deflections measured in micron at the corresponding offsets.

6) Structural *Condition Index* (*SCI*)

Since the SN estimates are sensitive to the pavement deterioration variables, the SN values can be used as a good indicator of the effective structural condition of a pavement. With the effective and required SN values of pavement, the Structural Condition Index (*SCI*) can be established for the pavement [28]. The Structural Condition Index (*SCI*) can be expressed by the ratio of the effective SN and the required SN as follows.

$$SCI = \frac{SNeff}{SNreg}$$

where:

SCI: is Structural Condition Index Computed from IRI & Distress data.

SNeff: is Effective Structural number.

SNreg: is Required Structural Number.

In this method, the required SN is usually calculated according to the estimated ESALs for the next 20 years. However, for the maintenance work, it is up to the agency to determine the time frame for which the accumulated ESALs become appropriate for the estimation.

Because of the simplicity of the SCI, the interpretation of its meaning is straightforward. An SCI value that is equal to or greater than one would indicate that the pavement is in a sound structural condition for the estimated future ESALs. However, this method is disputed in the argument that SCI less than one means that the pavement is no longer structurally adequate; as a result, rehabilitation work that will increase the structural capacity of the pavement should be considered.

7) Pavement Condition Index-PCI

To effectively plan and manage assets at the network level, there must be some means of comparing one asset condition to the next. For pavements, there are several ways of computing pavement condition indices as discussed in much literature [28] [29] [30] [31], typically the SCI, IRI, AUPP, Area, BLI, etc. A condition index takes into account asset evaluation data acquired through periodic monitoring. A condition index needs to be relevant, reliable, affordable, and appropriate.

According to [28] and as demonstrated in [29] the PCI is defined as a numerical rating of the pavement condition that ranges from 0 to 100, and they depend on aggregated factors of roughness, cracking, patching, reveling, rutting, and potholes. The rating level may depend on the agency or calibration of the machine in use [31].

The PCI score for the selected indicator can be determined by the following equation

$$SAI_j = 100\left(1 - \frac{I_j - I_{\min}}{I_{\max} - I_{\min}}\right)$$

or

$$SAI_j = 100\left(\frac{I_j - I_{\min}}{I_{\max} - I_{\min}}\right)$$

where

SAI_j: is the Structural Adequacy Index of Indicator being considered (j).

I_j: is the measured indicator being rated.

$I_{\min}$: is the statistical minimum of the sample of data that is being analyzed.

$I_{\max}$: is the statistical maximum value of the sample being analyzed.

The choice of which formula to use depend on the interpretation of the indicator being considered such that the 100 will indicate the best score and zero (0) will indicate the poor score. The downside of the PCI is that it can only provide information on pavement conditions only at the time of the survey, but cannot provide the prediction of the pavement in the future.

8) Estimation of pavement life using Kenlayer

The study on the behavior of interface conditions for asphalt pavement structures reported in [32] used the Kenlayer software in estimating the remaining life. The Kenlayer is an American computer program used to estimate the remaining life of asphalt pavement structures. Even though not much popular as the HDM4 model, it is simple enough to estimate the remaining life of the flexible pavement. Kenlayer can be applied to layered systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, either linear elastic, non-linear elastic, or viscoelastic. Maximum 19 layers and 24 load groups are allowed in this program. To study the influence of the interface condition on the pavement life, the stresses and strains in the pavement structure must be computed for each case strain value by using the finite element method. The equations used in the shell method is selected because of their simple form (subgrade strain model to a decrease in serviceability of 2.5) and derived as follows:

$$N_r = \left(0.028/\varepsilon_r\right)^4$$

where:

N_r: is the number of equivalent standard Axles to flexible pavement serviceability.

ε_r: is the vertical compressive at the top of the subgrade surface.

Fatigue cracking model (derived with laboratory specimens subjected to displacement-controlled four-point bending fatigue tests is also presented as follows:

$$N_c = 0.685\left(1/\varepsilon_t\right)^{5.671} \times \left(E_1\right)^{-2.363}$$

where:

N_c: is the number offload repetition to the failure by fatigue cracking.

ε_t: is the horizontal tensile strains at the bottom of the asphalt layer.

E_1: Is the asphalt elasticity modulus.

3. Method and Materials

3.1. Methodologies of Collecting FWD Data

Every year TANROADS collect pavement condition. At least each road section is visited after every 5 years. Recently the use of FWD has been introduced with a clear methodology of data collection.

The paved network amounting to about 7000 km was collected and the interval of the collection is about 250 m on the driving lane, making it about 3 - 4 data points per kilometer of road. Data were collected on the driving lane to reduce the risks associated with working with FWD on the high-traffic paved roads. The data were exported from FWD to CSV. The CSV file was programmed using VB.net into a relational database of the RMMS. The interface for the user selection option was modified to allow the use of SN or SNP. The algorithm for the Homogenous section was modified to incorporate SNP. Furthermore, the algorithm for using SN was maintained to allow user selection of suing SN or SNP in computing the road condition. Furthermore, TANROADS RMMS has also been adjusted to import and store the FWD data in RMMS. The remaining challenge is how to use the FWD data for reporting network conditions, maintenance planning, and adjusting the RMMS to automate the process. To decide the best way for integrating FWD data in the RMMS various methods were analyzed and the suitable and feasible method for integrating it in RMMS was selected.

3.2. The Necessity of FWD Data

Falling weight deflectometer (FWD) forensic testing is a valuable method for assessing the structural condition of existing pavement structures. For jointed plain concrete pavements (JPCPs), FWD testing is used to detect voids, monitor joints and crack performance, and back-calculate the modulus of elasticity of the existing Portland cement concrete (PCC) and the k-value of all supporting layers. For asphalt concrete (AC) pavements, FWD testing is used to back-calculate the stiffness of each layer and to estimate the amount of damage in the existing asphalt [4].

FWD data is required whenever a structural pavement overlay design is required, and California Bearing Ratio (CBR) or Resilient Modulus data are not available [2]. Furthermore, FWD data are required for Concrete Pavement Restoration (CPR) projects and overlays of existing concrete pavements.

3.3. Analysis of FWD Data

In this study, TANROADS collected deflection data for about 6934 km out of its paved network of about 9000 km. (The interval of the collection is about 250 m from point to point making about 29,000 data points for the 6934 km of roads).

The data points were aggregated into sub-links which are one kilometer each unless it is the last sub-link of a link. Various network strength indicators were calculated and summarized below.

3.4. Intelligence of RMMS

The intelligence of the PM module is that it can create homogenous sections for the network that are selected by the user for annual PM and uses the treatment matrix of traffic and distress data for the decision of the optimum treatment and cost for each homogenous section. The system then uses these treatments and costs as the basis for the PM needs of the network. The achieved needs are then prioritized using the multi-criteria analysis concept (MCA) to prioritize the needs. Criteria considered for MCA are Traffic, population server, production centers, social services, tourism attractions, road class, and connectivity. A score of each section for each criterion is summed up to an MCA score and all sections are then arranged in order of their scores and applied a cutoff point depending on the available budget. The selected sections become the basis for the constrained program under budget constraints.

Since there is only one Machine for FWD the activities of data collection are structures in the zone and then in the region carried out in March and April 2016 and 2019-2020. The data collection is structured by the zone, then by the region but all are collected in one quarter of the physical year. The system provides an API for computing SN and SNP as shown in Figure 2.

The structural analysis module computes the structural data for each sub-link of the selected network and accumulates them to link and finally to a road in a similar fashion to the analysis performed in routine recurrent maintenance. FWD analysis is not done on the unpaved network so the selection of the Unpaved Network is not considered.

The result of the API is the summary of network selection and description of each out presented in Figure 3. The summary shows no of link selected (1364), length of sub-link processed (70.2%), length of links with FWD data (29.80%), and Length of sublinks not processed (0.029%).

4. Results and Discussion

4.1. The Required Structural Number SNreq

The required Structural Number (SNReg) was assessed based on the Tanzania Pavement and Materials Design Manual with many references from the ASHTO method for calculation of Structural Number using layer thicknesses and material coefficients. The criteria for the assessment are presented in Table 1. Factors considered included the calculation of SN for proposed designs of pavement layers on different traffic levels, climatic conditions, and material properties. The assessment indicates that there is no significant difference between required SN for different climatic zones. However, some difference in SN was noted in the network with Asphalt base pavements. The author, therefore, recommends two

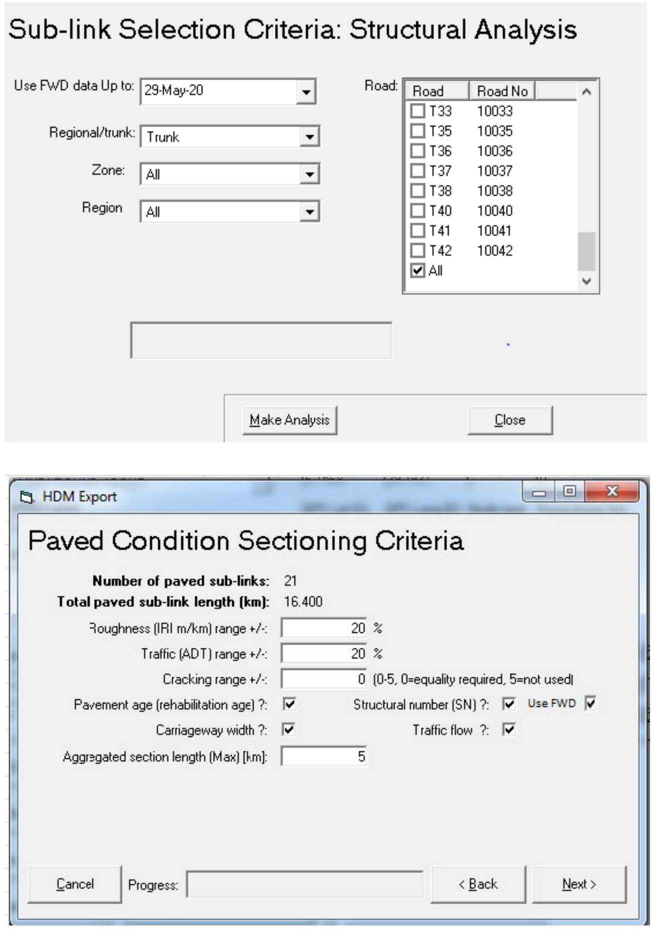
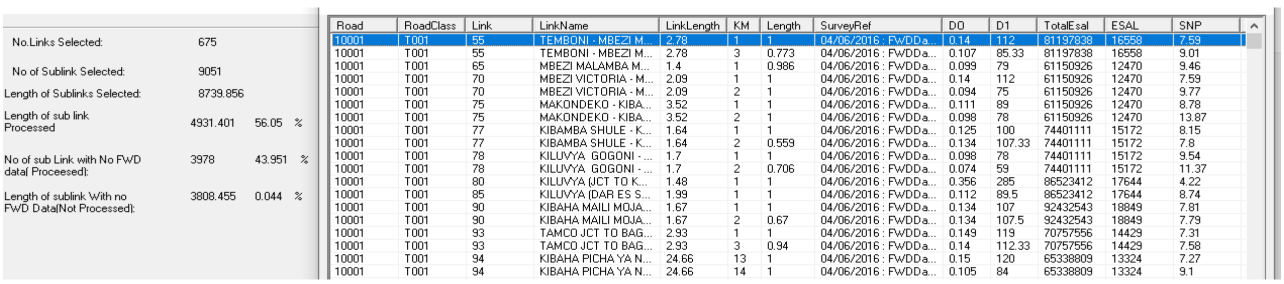


Figure 2. an API for computing SNP.



No.Links Selected:	675	
No of Sublink Selected:	9051	
Length of Sublinks Selected:	8739.856	
Length of sub link Processed	4931.401	56.05 %
No of sub Link with No FWD data(Proceeded):	3978	43.951 %
Length of sublink With no FWD Data(Not Processed):	3808.455	0.044 %

Road	RoadClass	Link	LinkName	LinkLength	KM	Length	SurveyRef	DO	D1	TotalEsal	ESAL	SNP
10001	T001	55	TEMBONI - MBEZI M...	2.78	1	1	04/06/2016 : FWDDa...	0.14	112	81197838	16558	7.59
10001	T001	55	TEMBONI - MBEZI M...	2.78	3	0.773	04/06/2016 : FWDDa...	0.107	85.33	81197838	16558	9.01
10001	T001	65	MBEZI MALAMBA M...	1.4	1	0.986	04/06/2016 : FWDDa...	0.099	79	61150926	12470	9.46
10001	T001	70	MBEZI VICTORIA - M...	2.09	1	1	04/06/2016 : FWDDa...	0.14	112	61150926	12470	7.59
10001	T001	70	MBEZI VICTORIA - M...	2.09	2	1	04/06/2016 : FWDDa...	0.094	75	61150926	12470	9.77
10001	T001	75	MAKONDEKO - KIBA...	3.52	1	1	04/06/2016 : FWDDa...	0.111	89	61150926	12470	8.78
10001	T001	75	MAKONDEKO - KIBA...	3.52	2	1	04/06/2016 : FWDDa...	0.098	78	61150926	12470	13.87
10001	T001	77	KIBAMBA SHULE - K...	1.64	1	1	04/06/2016 : FWDDa...	0.125	100	74401111	15172	8.15
10001	T001	77	KIBAMBA SHULE - K...	1.64	2	0.559	04/06/2016 : FWDDa...	0.134	107.33	74401111	15172	7.8
10001	T001	78	KILUVYA GOGONI - ...	1.7	1	1	04/06/2016 : FWDDa...	0.098	78	74401111	15172	9.54
10001	T001	78	KILUVYA GOGONI - ...	1.7	2	0.706	04/06/2016 : FWDDa...	0.074	59	74401111	15172	11.37
10001	T001	80	KILUVYA (JCT TO K...	1.48	1	1	04/06/2016 : FWDDa...	0.356	285	86523412	17644	4.22
10001	T001	85	KILUVYA (DAR ES S...	1.99	1	1	04/06/2016 : FWDDa...	0.112	89.5	86523412	17644	8.74
10001	T001	90	KIBAHA MAILI MOJA...	1.67	1	1	04/06/2016 : FWDDa...	0.134	107	92432543	18849	7.81
10001	T001	90	KIBAHA MAILI MOJA...	1.67	2	0.67	04/06/2016 : FWDDa...	0.134	107.5	92432543	18849	7.79
10001	T001	93	TAMCO JCT TO BAG...	2.93	1	1	04/06/2016 : FWDDa...	0.149	119	70757556	14429	7.31
10001	T001	93	TAMCO JCT TO BAG...	2.93	3	0.94	04/06/2016 : FWDDa...	0.14	112.33	70757556	14429	7.58
10001	T001	94	KIBAHA PICHA YA N...	24.66	13	1	04/06/2016 : FWDDa...	0.15	120	65338809	13324	7.27
10001	T001	94	KIBAHA PICHA YA N...	24.66	14	1	04/06/2016 : FWDDa...	0.105	84	65338809	13324	9.1

Figure 3. Structural analysis summary.

Table 1. Required structural number for different traffic load classes.

Road		Traffic						
		T02	T05	T1	T3	T10	T20	T50
Traffic volume		<200,000	200,000 - 500,000	500,000 - 1,000,000	1,000,000 - 3,000,000	3,000,000 - 10,000,000	10,000,000 - 20,000,000	>20,000,000
Base Type	Granular and Cemented	2.9	3.1	3.2	3.5	4.1	5	6
	Asphaltic Base	2.4	2.6	2.6	2.9	3.6	3.8	5

matrices to be adopted for the required SN, one for Asphalt base and another for other pavements as shown below.

4.2. Deflection Bawl Parameters

Different deflection bawl parameters were calculated and summarized as follows:

1) Structural Condition Indices (SCI)

The SCI is a ratio of the existing/effective Structural Number (SNeff) and the required Structural Number (SNreq). To assess the SCI, the FWD data from 6934 sublinks (about 29,000 data points) along with the lookup data from the RMMS database were analyzed with Excel spreadsheets, where the SCI for each of the sublinks was calculated for different formulas of calculating the effective SN and the required Structural Number derived from the provisions of the Pavement and Materials Design Manual 1999. The summary of the SCI analysis results for each effective SN method is shown Table 2.

The above analysis shows that SCI calculated using effective SN determined by Rhode is unreasonable because it shows that on average the SCI of the network is generally inadequate. The SCI calculated using SN from HDM-4, AUPP and Australian formulae are more sensible because it indicates the average SCI of the Tanzania Network is generally adequate (1.04 to 1.65). Given the general knowledge and experience of the Tanzania network, this generalization is not surprising because except for a few aged roads the Tanzania network is not much suffering from a lack of structural capacity.

2) Pavement Condition Index (PCI)/Structural Adequacy Index (SAI)

The PCI of all collected data (6934 sub links) was calculated according to ASTM D-6433-99, using various SN calculation options and the selected deflection basin parameters. Furthermore, the SAI scores and SAI ratings were calculated for the selected structural condition indicators as shown in Table 3. The analyzed structural indicators are:

a) Area.

b) Area Under Pavement Profile (AUPP).

c) Structural Adequacy Index using HDM-4 effective SN.

d) Structural Adequacy Index using Rhodes effective SN.

Table 2. Statistical summary of SCI analysis results for different methods.

	(Rhode')	(HDM)	SCI3 (AUPP)	New Zealand	AREA	AUPP
Mean	0.64	1.04	1.65	1.22	16.46	672.54
Max	3.82	3.48	5.24	3.00	33.17	3322.33
Min	0.28	0.30	0.62	0.45	8.28	17.28
STD	0.20	0.46	0.52	0.38	2.86	388.63
97 percentile	1.02	2.14	2.79	2.08	22.23	1567.22
3 percentile	0.36	0.46	0.89	0.65	11.76	199.36

Table 3. Summary of result analyses for the network for different methods.

	SAI1 (Rhode SN)	SAI2 (HDM_SN)	SAI3 (Aupp_SN)	SAI4 (NZ_SN)	SAI5 (AREA)	SAI6 (AUPP)
	SAI INDICATORS (No. Sub Links)					
V. Good (>80)	702	3685	6538	5328	740	2590
Good (60 - 79)	1052	871	56	464	1168	1935
fair (40 - 59)	1751	905	58	395	1927	1185
Poor (25 - 39)	1644	585	30	245	1456	573
V. Poor (0 - 24)	1785	888	252	502	1643	651
	SAI INDICATORS (%)					
V. Good (>80)	10%	53%	94%	77%	11%	37%
Good (60 - 79)	15%	13%	1%	7%	17%	28%
fair (40 - 59)	25%	13%	1%	6%	28%	17%
Poor (25 - 39)	24%	8%	0%	4%	21%	8%
V. Poor (0 - 24)	26%	13%	4%	7%	24%	9%

e) Structural Adequacy Index using New Zealand effective SN.

4.3. Summary of the Analysis of the Different Methods

From Table 3, HDM-4, New Zealand, and AUPP deflection bawl indicate that about 11% - 21% of the network is in poor and very poor condition. However, the HDM-4 formula is considered more appropriate to be adopted for data collected in the RMMS network because of the following reasons:

1) HDM-4 is integrated with RMMS and it is preferred to use the same formula of Calculating SNP that HDM-4 uses for its internal calculations. This enables a comparison of the results from HDM-4 to be those from RMMS.

2) HDM-4 is a wide researched software and most of the research was carried out in tropical and developing countries similar to Tanzania.

3) HDM-4 utilizes the central deflection and knowledge of the base type only. While it minimizes the need for detailed information that may not be available in most road management systems (surface, base, sub-base thickness, material types, subgrade properties, etc.), it uses the basic information (base type) that is easily available in RMS.

4) Deflection bawl parameters do not use information that relates to the actual road details such as traffic levels, layers types or thickness, etc. The judgment from deflection bawls parameters will therefore not distinguish the high trafficked road from the low trafficked road, roads that need a low standard of construction from those that need a high standard of construction, etc.

4.4. Structural Condition Index Limits for the Adopted Condition Indicator

Critical analysis was carried out to determine the cut-off point for various SCI indicators. Table 4 shows the limiting values of SCI for different condition indicators for the proposed SCI indicator to be adopted for our network.

This concept and the SCI analysis can be used to filter the road sections which require rehabilitation/strengthening from those that require normal Periodic Maintenance (PM) intervention. It is proposed to use this concept to filter those sections that need reconstruction/rehabilitation from those that need PM. The sections that require strengthening (Rehabilitation/reconstruction) are assigned treatment from a rehabilitation matrix, while those that require PM are assigned treatment from the existing treatment matrix that triggers maintenance interventions using distress (roughness, cracking, raveling) and traffic. The proposed Strengthening matrix is shown in Table 5.

4.5. Proposed and Adopted Pavement Strength Indicator

As earlier indicated several pavement strength indicators ranging from deflection bowl parameters, Structural Number, and structural Condition Index were analyzed. Critical analysis carried out indicated that the Structural Condition Index (SCI) calculated using the Structural Number calculated using HDM-4 formula and required structural number indirectly obtained using the Pavement and Materials Design manual was found to represent the expected structural pavement condition in Tanzania context. It is there adopted that this SCI should be used to report the Structural Condition of Tanzania's paved roads network. For this reason, RMMS will need to be adjusted to include an additional module (Pavement Strength Module) for the analysis and reporting of the FWD data.

4.6. Computation of ESAL from Traffic Data

To ascertain the backlog Axle loading is important. The routine for calculating ESAL and updating Highway Ordnance was introduced. ESAL was read from the database where it is stored per link.

ESAL was calculated from MTAADT of the survey data as follows:

$$ESAL = \sum_{i=1}^{n} \left(Vi * VEF \right)$$

Table 4. Limiting values of SCI for condition categories.

Condition Category	V. Good (1)	Good (2)	Fair (3)	Poor (4)	V. Poor (5)
SCI Value	>0.89	0.78 - 0.89	0.68 - 0.78	0.59 - 0.68	<0.59

Table 5. Proposed strengthening matrix.

SCI	<0.59	0.60 - 0.68	0.68 - 0.78	0.78 - 0.89	>0.89
Treatment	REHAB	LREHAB	PM	PM	RM

where:

VEF = *Vehicle Equivalent factor from Table LookupTrafficType in RM4HDM Database*. (*in this case n* = 14).

Vi = *Vehicle Categories*.

4.7. Algorithm for Computing the SNP

The inventory data of the road section is stored per sub-link of a maximum of 1000 m. For each sub-link in the road selection, the following rules apply:

1) Read the current deflection values from table PavementStrengthPavedFWD PavementStrengthPavedRaw). Read the values D1, D2, D3… D9.

2) Read ESAL of the Link from HighwayOrdance.

3) Read TrafficGrowthRate (r) and Analysis period (n) from PMParams.

$$ESAL = 365 * 0.5 * (ESALForStarLink) * \left[\frac{1 + (0.01 * r)^n - 1}{r} \right]$$

4) Determine Required structural number SN req of each Homogenous section from lookup tables.

Calculate Structural Condition Index *SCI* using the formula

$$SCI = \frac{SNP}{SNreg}$$

5) Use the SCI (SCICode) and Total ESAL to determine the Appropriate Rehabilitation from a lookup table Assigned RehabilitationTreatment Matrix RerhabPaved.

6) Calculate the Remaining Life of the Overlay thickness as follows

$$Remaining\ Life = \frac{SNP}{SNreq} * DesignPeriod.Tthis\ is\ the\ same\ as\ SCI * pavmentAge$$

7) Determine the Overlay Thickness as follows

$$Overlay\ Thickness = \frac{SNreq - SNP}{0.4}$$

Determine the Treatment Cost by Reading Treatment unit cost from lookup table Treatment Type in RM4HDM

$$RehabCost = Length * 100 * Width * Unit\ Cost$$

From the computation of the remaining life of the pavement, it was reasonable to conclude that FWD data be used to screen sections that have structural problems. The aim is to identify sections that would require rehabilitation/reconstruction. Other sections that do not require rehabilitation would be periodic maintenance using the existing treatment matrix that uses the distresses and traffic level to decide on the suitable treatment type.

4.8. Presentation of Analysis of Results for the Proposed Algorithm

The summary of ESAL (Equivalent standard Axles Loading) is summarized in

Table 6.

The mean value of ESAL was found to be 10,805,883.80 while the standard deviation read at 12,958,241.82. The Maximum value was 155,643,084.00 while the minimum value was 127,500.00.

The final programmed algorithm was able to produce different types of reports including Structural condition, backlog maintenance, overlay thickness, SCI, overlay thickness Remain life of the pavement as well as ESAL loading of each road section. Table 7 shows the results for the structural condition summary.

The network length analyzed for structural adequacy was about 193,554.4. Kms both ST and AM were shown in Table 7. It was established from the network level that, structurally the pavement is 64.72% was very good, 12.04% was Good, 10.00% was Fair, 7.84% were Poor, while 5.40% were very poor. The deeper analysis on the surface type indicated that those sections with ST were found to have good structurally compared to AM sections. For the very good condition were 15.36%) for Am (49.37%) for ST, good were (5.34%) for AM and (6.70%) ST, fair (4.07%) for AM and (5.93%) for ST, poor were (3.42%) for AM and (4.42%) for ST while very poor were (2.53%) for AM and (2.87%) for ST.

The results on backlogs maintenance are shown in Table 8. The section that needs rehabilitation is about 11.44% of which about 6.41% required light rehabilitation while 5.03% requires heavy rehabilitation. Likewise, when categorized with surface type those sections with ST require more rehabilitation (6.23%) compared to those sections with AM which is about 5.2%.

The analysis of the remaining life of the flexible pavement was presented in Table 9. A range-based analysis shows that those sections with thickness

Table 6. ESAL values.

	Values
Mean	10,805,883.80
Max	155,643,084.00
Min	127,500.00
Standard Deviation (STD)	12,958,241.82

Table 7. Structural condition summary.

Road Class	**Surface**	**V_Good**	**Good**	**Fair**	**Poor**	**V_Poor**	**Total**
Trunk Roads	AM	29,724.05 (15.36%)	10,333.6 (5.34%)	7879.1 (4.07%)	6616.73 (3.42%)	4902.65 (2.53%)	59,456.09 (30.72%)
	ST	95,550.36 (49.37%)	12,974.14 (6.70%)	11,477 (5.93%)	8550.83 (4.42%)	5546.22 (2.87%)	134,098.34 (69.28%)
	Total	125,274.41 (64.72%)	23,307.74 (12.04%)	19,356 (10%)	15,167.56 (7.84%)	10,448.87 (5.40%)	193,554.43 (100%)

Table 8. Backlog maintenance.

Surface	Light REHAB	REHAB	PM	RM	Total
AM	5463.6 (2.82%)	4611.1 (2.38%)	18,502.6 (9.56%)	30,878.7 (15.95%)	59,456.1 (30.72%)
ST	6935.2 (3.58%)	5131.6 (2.65%)	25,307.1 (13.07%)	96,724.4 (49.97%)	134,098.3 (69.28%)
Total	12,398.8 (6.41%)	9742.8 (5.03%)	43,809.7 (22.63%)	127,603.1 (65.93)	193,554.4 (69.28%)

Table 9. Overlay thickness.

	Overlay Thickness				
Surface	15 - 20	10 - 15	5 - 10	0 - 5	Total
AM	44,705.05 (23.1%)	13,460.58 (6.9%)	1290.46 (0.67)	0	59,456.09 (30.7)
ST	114,731.7 (59.3%)	18,430.54 (9.5%)	936.08 (0.5)	0	134,098.3 (69.3%)
Total	159,436.8 (82.37%)	31,891.12 (16.5%)	2226.54 (1.2%)	0	193,554.4 (100%)

between 15 - 20 were 82.37%, 10 - 15 were about 16.5% while those with 5 - 10 were 1.2%. The section with surface Am was about 30.7% while those with St were 69.3%.

5. Conclusion and Recommendations

5.1. Conclusion

This study examines eight methods of determining SN and the conclusion was to apply those proposed by HDM3-4 with AASHTO Guideline. A minor modification was done to accommodate data collected into RMMS. A new algorithm was designed to calculate ESAL which was used to compute SN, SNP, SNReq, SNeff, SCI, remaining life of the pavement, Overlay thickness as well as rehabilitation cost. The FWD data of a network of structural adequacy of about 193,554.4 was collected. Both ST and AM were analyzed and it was established that structurally the pavement of about 64.72% was very good, 12.04% was good, 10.00% was Fair, 7.84% were Poor, while 5.40% were very poor. Concerning overlay thickness, it was concluded that those sections with thickness between 15 - 20 were 82.37%, 10 - 15 were 16.5% while those with 5 - 10 were 1.2%. Likewise, it was concluded that a network of about 11.44% required rehabilitation.

5.2. Recommendations

It is recommended that TANROADS start using the FWD measurement in the ongoing projects as the means of quality assurance during the construction period and handover of road pavement projects. Nevertheless, this study did not consider the temperature effect on the calculation of strength, therefore more study is required in establishing temperature as a correction factor in the computation of the remaining life. Notwithstanding it is recommended to carry study on combining FWD data with PCI in reporting the road condition.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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