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A NEW METHOD TO DETERMINE MAINTENANCE AND REPAIR ACTIVITIES AT NETWORK-LEVEL PAVEMENT MANAGEMENT USING FALLING WEIGHT DEFLECTOMETER

Amir KAVUSSI[a], Mojtaba ABBASGHORBANI[a], Fereidoon MOGHADAS NEJAD[b], Armin BAMDAD ZIKSARI[c]

[a]*Department of Civil and Environmental Engineering, Tarbiat Modares University, Tehran, Iran*
[b]*Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran, Iran*
[c]*Technical and Soil Mechanics Laboratory of Ministry of Road and Urban Development, Tehran, Iran*

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Abstract. Pavement condition assessment at network level requires structural evaluation that can be achieved using Falling Weight Deflectometer (FWD). Upon analysing FWD data, appropriate maintenance and repair methods (preservation, rehabilitation or reconstruction) could be assigned to various pavement sections. In this study, Structural Condition Index (*SCI*), defined as the ratio of Effective Structural Number (SN_{eff}) to Required Structural Number (SN_{req}), was used to determine if a pavement requires preservation or rehabilitation works (i.e. preservation $SCI>1$, rehabilitation $SCI<1$). In addition to FWD deflection data, SCI calculation requires pavement layer thicknesses that is obtained using GPR with elaborated and time consuming works. In order to reduce field data collection and analysis time at network-level pavement management, SCI values were calculated without having knowledge of pavement layer thicknesses. Two regression models were developed based on several thousand FWD deflection data to calculate SN_{eff} of pavements and resilient modulus (M_R) of their subgrades. Subgrades M_R values together with traffic data were then used to calculate SN_{req}. Statistical analysis of deflection data indicated that Area under Pavement Profile (AUPP) and the deflection at distance of 60 cm from load center (D_{60}) parameters showed to have strong correlation with SN_{eff} and M_R respectively. The determination coefficients of the two developed models were greater than those of previous models reported in the literature. The significant result of this study was to calculate SN_{eff} and M_R using the same deflection data. Finally, implementation of the developed method was described in determining appropriate Maintenance and Repair (M&R) method at network level pavement management system.

Keywords: Falling Weight Deflectometer (FWD), Structural Condition Index (*SCI*), Maintenance and Repair (M&R) method, network-level pavement management, structural assessment, subgrade M_R.

Introduction

Pavement Management System (PMS) is implemented at network and project levels. At network level management, detailed data with high accuracies are not required. At this level, the type of Maintenance and Repair (M&R) methods and required budget are determined for a road network. Pavements will then be assessed more accurately for specific M&R method at project level. There are several categories of M&R methods (Fig. 1) including pavement preservation, rehabilitation, and reconstruction (Pavement Interactive 2010).The most significant feature of a pavement is its structural condition, specifying either M&R preservation or rehabilitation methods. Structurally weak pavements require rehabilitation or reconstruction works, which are costly and time consuming. For preservation works, minor road works and low cost preventive maintenance methods (e.g. fog seal, slurry seal, chip seal and hot in-place recycling) will be applied. Determination of distinction level between preservation and rehabilitation/reconstruction works is of great importance. Falling Weight Deflectometer (FWD) can be used to assess pavement structural condition and determine if a pavement requires preservation or rehabilitation. Figure 1 shows a general overview of the various maintenance level (Chowdhury *et al.* 2012).

Structural Condition Index (*SCI*) based on FWD data is used to decide on the appropriate M&R method (Shahin 2005; Zhang *et al.* 2003; Kim *et al.* 2013). *SCI* is the ratio between Effective Structural Number (SN_{eff}) and Required Structural Number (SN_{req}) (Eqn (1)). These parameters were recommended by AASHTO to design pavement structure (AASHTO 1993):

$$SCI = \frac{SN_{eff}}{SN_{req}}. \quad (1)$$

Corresponding author: Amir Kavussi
E-mail: *kavussia@modares.ac.ir*

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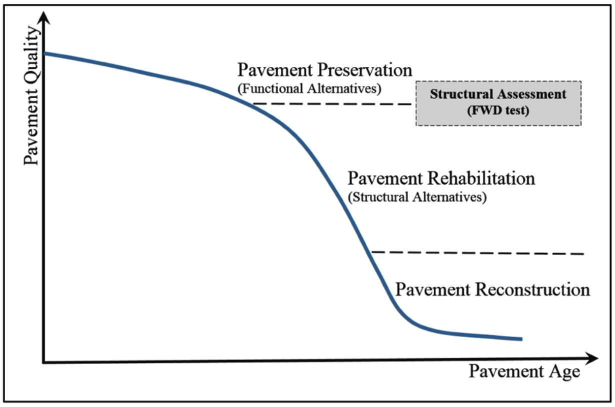

Fig. 1. Categories of various required M&R activities versus pavement life

If $SCI \geq 1$, the pavement will have enough load bearing capacity to sustain traffic loading and only preservation activities will be required. If $SCI < 1$, the pavement is structurally weak and requires rehabilitation or reconstruction (Stubstad *et al.* 2012; Bryce *et al.* 2013). SN_{eff} describes bearing capacity of the existing pavement including aspht, base and subbase layers on top of the subgrade. SN_{req} represents the required load bearing capacity of pavements regardless of the existing pavement condition.

The total pavement thickness and FWD deflection data are used in AASHTO (1993) rehabilitation procedure to determine SN_{eff} of pavement and M_R of subgrade (AASHTO 1993). The analysis initiates by a trial and error procedure to find subgrade deflection. SN_{eff} is then determined using pavement effective modulus (E_P) and the total pavement thickness. In addition, SN_{req} is calculated using pavement design life traffic and the subgrade resilient modulus (M_R) determined from FWD deflection data. Based on AASHTO pavement design method, both SN_{eff} and SN_{req} require deflection data and total thickness of pavement, which must be determined in the field (AASHTO 1993). Structural evaluation of pavements at network level is associated with several complications, summarized below:

1. Analysis of deflection data needs knowledge of layer thicknesses that is measured either using non-destructive equipment such as Ground Penetrating Radar (GPR) or destructive methods such as core drilling and pit boring of pavements. Both approaches are time consuming and require sophisticated works.
2. Data analysis in AASHTO method requires elaborated back-calculation works, which are complicated and time consuming.
3. The main purpose of network level pavement management is to determine the type of appropriate M&R methods requiring annual budget dedication, so that large amounts of data with high accuracies are not required.

The aim of this research was to solve the outlined problems without having knowledge of pavement thickness. With this regard, two regression models were analysed to predict SN_{eff} and M_R of subgrade based on Deflection Basin Parameters (DBP). The M_R of the subgrade together with traffic data and reliability level are used to calculate SN_{req} at the point where SN_{eff} has already been calculated. These two *SNs* are then used to calculate *SCI* values in homogenous pavement sections and determine the distinction level between functional and structural alternatives at road network.

1. Deflection basin parameters

Deflection basin is a profile formed from deflections data determined from the FWD device as it is shown in Figure 2.

With reference to this figure, it is noted that the largest portion of basin variation occurred near the loading center. Thus, parameters extracted from this part of basin can truly be used to detect changes in pavement structural capacity. Deflection Basin Parameters (DBPs), reported in Table 1, are the most frequently used equations for FWD data analysis. Among various parameters shown in Table 1, central deflection (D_0) is the most significant parameter used in pavement evaluation (both at network and project levels). This parameter defines the pavement and subgrade behaviour (Dasari 2013). It is also utilized to divide pavement into homogenous structural sections. Parameters such as Base Layer Index (*BLI*), Base Damage Index (*BDI*) and Base Curvature Index (*BCI*) are useful for structural assessment of asphalt, base and subbase/subgrade layers, respectively (Aavik 2003). The AREA parameter in Table 1 determines the relative structural condition of pavement to subgrade and Area Under Pavement Profile (*AUPP*) represents condition of upper portion of pavement (Horak *et al.* 2009).

2. Research methodology

The Database used in this research were taken from several main roads in Khuzestan province in south of Iran.

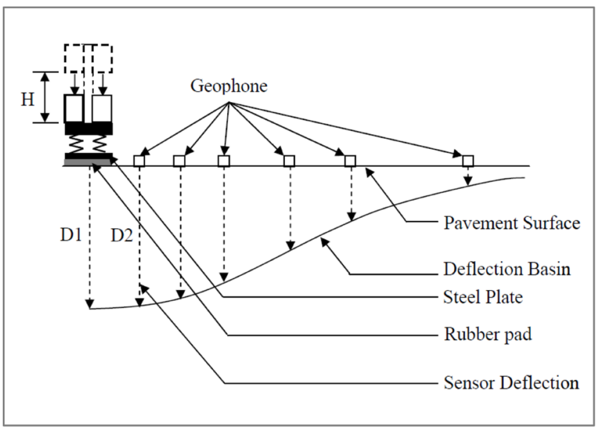

Fig. 2. Pavement deflection basin resulted from FWD device

Table 1. Deflection basin parameters (DBPs) (Horak *et al.* 2009)

Parameter	Equation	Description
D_{max}	Central deflection	Pavement Structural evaluation and pavement sectioning
BLI	$BLI = D_0 - D_{200}$	Structural assessment of asphalt layer
BDI	$BDI = D_{300} - D_{600}$	Structural assessment of base layer
BCI	$BCI = D_{600} - D_{900}$	Structural assessment of subbase and subgrade
AREA	$AREA = 6\left(1 + 2\frac{D_{300}}{D_0} + 2\frac{D_{600}}{D_0} + \frac{D_{900}}{D_0}\right)$	Structural assessment of pavement and subgrade
AUPP	$AUPP = \left(\frac{5D_0 - 2D_{300} - 2D_{600} - D_{900}}{2}\right)$	Structural assessment of upper portion of pavement

These included 2453 FWD (Dynatest) deflection basins and corresponding thicknesses using GPR. The data were collected using Dynatest FWD Type 9000 and Mala GPR Type 1 GHz ground couple. Table 2 reports the statistical analysis of the data used in this research.

As it can be seen from Table 2, the database showed a good scattering in measured parameters (deflection and thickness). FWD central deflections were adjusted to 20 °C and were normalized to stress of 570 kPa to simulate the loading of half-standard axle load. Temperature correction factors were applied based on AASHTO (1993) design guide for central deflections. To eliminate the effect of temperature on DBPs (particularly at *AREA*, *AUPP* and *BLI* parameters), the deflections at the distances of 30, 60 and 90 cm from the loading center were adjusted using MODULUS program method. Table 3 reports the adjustment factors of the above deflections as percentage of temperature correction factor from the central deflection. As it can be seen in this table, the effect of temperature on deflections increased as a result of increasing thicknesses of asphalt layers. If the thickness of asphalt layer is less than 75 mm, no temperature correction will be required.

Prior to calculating DBPs, the first four FWD deflections (up to 90 cm) were adjusted to temperature of 20 °C. The role of temperature adjustment is of great importance in calculating *BLI*. It can be negative if temperature correction factor is just applied to the central deflection (adjusted central deflection is less than deflection at 30 cm). DBPs such as *BLI*, *BDI*, *BCI*, *AREA* and *AUPP* were then calculated at each point of the database. SPSS statistical analysis software was used to correlate M_R of

Table 2. Summary of statistical analysis of the research database

Type of statistics		D_0[1] (micron)	D_{60}[1] (micron)	D_{90}[1] (micron)	D_{120}[1] (micron)	SN_{eff}	SN_{req}	Subgrade M_R (MPa)	AC[2] (mm)	BS[3] (mm)
n		2453	2453	2453	2453	2453	2453	2453	2453	2453
Mean		198	81	53	37	5	5	71	171	258
Median		158	74	52	37	5	5	62	171	260
Std. Deviation		126.8	36.0	21.8	16.1	1.7	1.3	36.3	51.2	73.3
Range		728	183	126	90	7.5	6.5	277	321	411
Minimum		40	17	10	6	1.5	2.2	25	52	100
Maximum		768	200	136	96	9.0	8.7	302	373	511
Percentiles	60	185	84	57	41	5	6	69	186	280
	70	224	96	65	46	6	6	80	199	302
	80	275	112	72	51	7	6	93	212	322
	85	323	120	76	54	7	7	103	221	332
	90	389	133	82	58	7	7	119	234	344
	95	492	150	90	64	8	8	143	255	367

(1) D_x is the deflection at distance of *x* cm from FWD loading center.
(2) The total thickness of asphalt layers.
(3) The total thickness of base and sub-base layers.

Table 3. Percentage of temperature correction factor applied to each FWD sensor (Fernando *et al.* 2001)

FWD Sensor	*AC* Thickness <75 mm	*AC* Thickness 75 mm – 125 mm	*AC* Thickness >125 mm
D_0	0	100	100
D_{30}	0	45	62
D_{60}	0	12	34
D_{90}	0	5	10

subgrade and SN_{eff} of pavement with deflection basin parameters. Subgrade M_R and pavement SN_{eff} values were determined according to AASHTO (1993) design method. These two parameters were considered as dependent variables and DBPs were assumed as independent variables in the statistical analysis. The database was inserted into SPSS software and were divided randomly into 80% and 20% divisions. The first part was used to develop the models of M_R and SN_{eff}, while the second part (i.e. 20%) was used for their validation purposes.

3. Comparison of AASHTO (1993) versus current models

Several models were developed by researchers using deflection basin parameters without using any layer thicknesses. In this research, current SN_{eff} and M_R models were reviewed and their correlation with AASHTO (1993) results were calculated.

3.1. SN_{eff} models

Many researchers have studied the relationship between SN_{eff} and DBPs. Table 4 reports five major models. As it can be seen, from this table the subgrade resilient modulus or deflections corresponding to subgrade responses (i.e. D_{900}, D_{1200}, D_{1500}) were used to calculate SN_{eff} in many models. This justifies the dependency of SN_{eff} to relative pavement/subgrade strength (Crook *et al.* 2012).

The database of this research were used to compare the above models for estimating SN_{eff} parameters. In Table 4, parameters such as slope, intercept and R^2 of the simple linear regression in form of $y = ax + b$ (between each model and AASHTO (1993) proposed method) are reported. In Noureldin model (Noureldin 1993), it was assumed that the deflection (d_x) at the distance of r_x detects the subgrade response (deflection). This deflection is specified among FWD sensors when the equation $d_x \times r_x$ becomes maximum. SN_{eff} is then calculated using subgrade deflection and its distance from the load center using the equation presented in Table 4. The low R^2 and high intercept value prove the poor results obtained by Noureldin Model. R^2 coefficients of SN_{eff}, obtained from the other four models were better and were close to each other (compared with AASHTO values). Based on regression parameters (slope close to one, intercept close to zero and high R^2) COST 336 (1998) model showed to provide better estimate of SN_{eff} among all the other models.

3.2. Subgrade M_R models

Determining SN_{req} according to AASHTO (1993) design guide, requires determination of M_R of subgrade based on FWD deflection data. In FWD testing, geophones located at longer distances from the loading center measure deflections of the lower pavement layers. For example, if the stress is distributed with an angle of 45°, the geophone located at radial distance of 90 cm from the load center measures deflection at the depth of 90 cm from pavement surface. However, this assumption is not accurate because stress distribution varies at each layer due to changes in stiffness of the various layers. Accurate estimation of M_R is of great importance due to the large impact on SN_{req}. Furthermore, M_R should be determined using the same deflection basin, by which SN_{eff} has been calculated. This is the correct method of comparing SN_{eff}

Table 4. Current SN_{eff} prediction models based on only deflection data

Equation	Model Parameters	Statistical characteristic			References
		a	*b*	R^2	
$SN_{eff} = \frac{\left(4r_x^2 - 36\right)^{1/2}}{17.234\left(r_x.D_x\right)^{1/3}}$	R_x (in), D_x (in)	0.250	2.524	0.639	Noureldin (1993)
$SN_{eff} = 0.0182 l_0 \times \sqrt[3]{E_{sg}}$	l_0 (cm), E_{SG} (MPa)	0.478	1.724	0.838	Hoffman (2003)
$SN_{eff} = 1.69 + \left(\frac{842.8}{D_0 - D_{1500}}\right) - \left(\frac{42.94}{D_{900}}\right)$	D_0, D_{900}, D_{1500} (in)	1.034	–0.529	0.848	COST 336 (1998)
$SN_{eff} = 13.5 - 6.5 \times log D_0 + 3.7 \times log D_{900}$	D_0, D_{900} (micron)	0.269	2.588	0.757	Jameson (1993)
$SN_{eff} = e^{5.12} AUPP^{-0.78} BLI^{0.31}$	*AUPP*, *BLI* (micron)	0.457	2.588	0.827	Schnoor and Horak (2012)

with SN_{req} and SCI determination on each point of the pavement. It should be noted that M_R of subgrade, resulted from back-calculation analysis must be corrected by a factor of 0.33 in accordance with AASHTO (1993) method so that it could be considered equivalent to M_R obtained in laboratory. Table 5 presents some models of M_R estimation based on DBPs. Similar to SN_{eff} models analysis, R^2 coefficient and parameters of simple linear regression between M_R at each model and AASHTO (1993) are represented. In these models, deflections at 90 cm and further distances from the load center were used to calculate M_R of subgrade. In Horak model (Horak 1987), deflection at 180 cm from the load center (instead of 200 cm) was used to calculate M_R values, because this is the farthest deflection in the database. There were no significant relationship between AASHTO (1993) and Horak (1987) results. In the research database, the deflections located at 60 and 90 cm from load center were often used to calculate M_R of subgrades (this is described in the following sections) while Horak (1987) model uses D_{200} in the model. This issue reduced the accuracies of Horak (1987) model. In Table 5, one of the Washington State models which used the deflection at 90 cm (radial distance) showed the best correlation (i.e. $R^2 = 0.76$).

4. Development of pavement SN_{eff} model

A correlation analysis was conducted with the aim of understanding relationship between DBPs and SN_{eff} obtained from AASHTO (1993) method (see Table 6). As it can be seen in this table, *AREA* is directly related to SN_{eff} and the others had inverse relationship. Moreover, it can be concluded that parameters such as D_0, *BLI* and *AUPP* showed better correlations with SN_{eff}. Table 6 also indicates that all parameters have non-linear relationship with SN_{eff}. Therefore, it can be resulted that DBPs in non-linear model can provide a better model to show the variation of dependent variable (SN_{eff}).

Figure 3 represents SN_{eff} variation versus central deflections (D_0). As it can be seen, the trend of D_0 shows a non-linear relationship. This figure also indicates the scattering of D_0 and SN_{eff} in the database.

Regression analysis in SPSS software was performed using AASHTO (1993), SN_{eff} as the dependent variable and parameters presented in Table 6 as the independent ones. From this statistical analysis, main six regression models were developed using 80% of data as presented in Table 7. The first three models indicate the effect of each DBP to predict SN_{eff}.

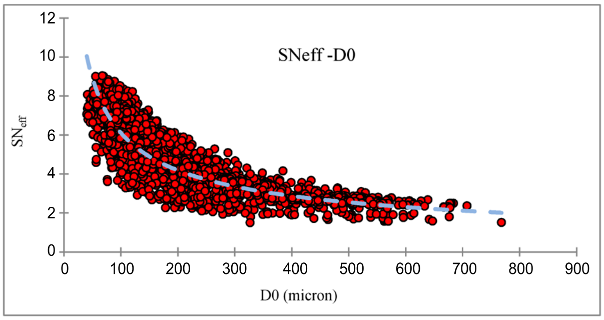

Fig. 3. SN_{eff} variation with D_0

Table 5. Models of subgrade resilient modulus based on DBPs

Equation	Model Parameters	Statistical characteristic			References
		a	b	R^2	
$\log E_{SG} = 9.727 - \log D_{200}$	E_{SG} (Pa), D_{200} (micron)	0.037	8520	0.182	Horak (1987)
$E_{SG} = -371 + 0.00671\left(\dfrac{2p}{D_{900} + D_{1200}}\right)$	E_{SG} (psi) P (lb.) D_{900} (in) D_{1200} (in)	0.186	2707.2	0.696	Zhang *et al.* (2003)
$E_{SG} = -198 + 0.00577\left(\dfrac{p}{D_{1200}}\right)$		0.129	4746.6	0.544	
$E_{SG} = -466 + 0.00762\left(\dfrac{p}{D_{900}}\right)$		0.2172	1779.2	0.760	

Table 6. Correlation factor of independent variables with SN_{eff}

Parameter	D_{90}	*AUPP*	*AREA*	*BDI*	*BCI*	*BLI*	D_0
Correlation Factor	–0.111	–0.818	0.621	–0.778	–0.702	–0.817	–0.783
Relationship	N.L	N.L	N.L	N.L	N.L	N.L	N.L

Table 7. Regression models developed to determine SN_{eff}

No.	Models	80% data			20% data		
		a	b	R^2	a	b	R^2
1	$62.245 \times D_0^{-0.503}$	1.026	–0.132	0.728	1.026	–0.153	0.745
2	$0.000243 \times AREA^{1.565}$	1.000	0.018	0.675	1.013	–0.058	0.677
3	$35.473 \times AUPP^{-0.389}$	1.021	–0.099	0.864	1.015	–0.091	0.864
4	$34.171 \times D_0^{-0.638} \times D_{90}^{0.33}$	1.002	–0.032	0.876	0.996	–0.012	0.878
5	$4.181 \times AUPP^{-0.334} \times AREA^{0.293}$	1.024	–0.098	0.872	1.019	–0.091	0.871
6	$28.007 \times AUPP^{-0.22} \times BLI^{-0.17}$	0.944	–0.093	0.868	0.942	–0.096	0.868

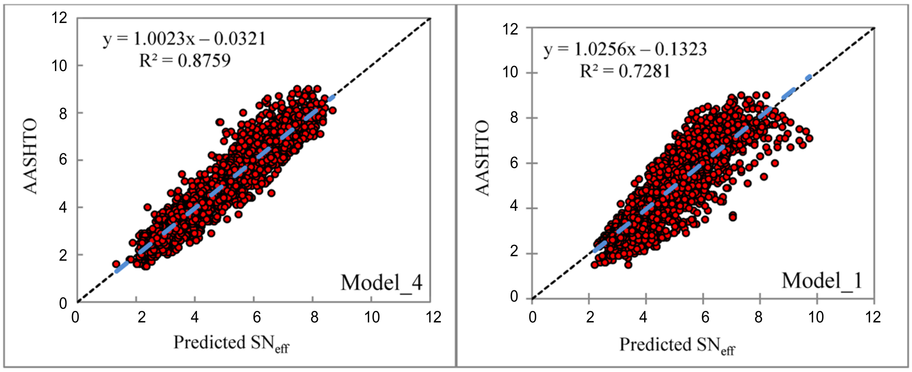

Fig. 4. Variation of predicted SN_{eff} versus AASHTO (1993) observed SN_{eff}

For comparison purposes, the predicted SN_{eff} (determined from the models) versus observed SN_{eff} (AASHTO (1993) method) were drawn and their simple linear regression parameters were determined. For validation purposes, SN_{eff} values were calculated using 20% data (this portion did not participate in model development) and compared with AASHTO (1993) method in the same way. As it can be seen in the first three models, the strength of D_0 and *AUPP* parameters to estimate SN_{eff} is more than *AREA*. The combination of the parameters were then used in other models to enhance their accuracy. For example, Model No. 4 was developed using D_0 and D_{90}. Adding D_{90} as the effects of lower layers improved Model No. 4 compared with Model No. 1. *AUPP* was accompanied by *AREA* and BLI parameters to develop the above last two models. Among the six models presented in Table 7, Models No. 4 and 5 showed the best results. Model No. 4, due to its better regression parameters (slope close to one, intercept close to zero and higher R^2) was selected as the best model for estimating SN_{eff}. This model was also better validated in 20% of data, compared with Model No. 5. Figure 4 shows variations of predicted SN_{eff} obtained from Models No. 1 and 4 versus AASHTO (1993) SN_{eff}. The data points were better concentrated around the line with 45° angle in Model 4.

5. Development of subgrade M_R model

The initial step in model development was to perform correlation analysis between DBPs and AASHTO (1993) M_R using SPSS software. Correlations reported in Table 8 show that the relationship becomes stronger with increasing distance from the load center (up to 60 cm) and then the relationship decreased gradually. D_0 shows a correlation factor of 0.54 because it describes pavement and subgrade structural behaviour. As expected, *BLI*, representing upper layer structural response, had the lowest relationship with M_R of subgrade. *BCI* representing subbase and subgrade behaviour had stronger relationship, compared with *BLI* and *BDI*.

With respect to Table 8, the best correlation belonged to inversed deflections at distance of 60 cm from the load center. As it can be seen in this table, inverse deflections showed linear relation with M_R, while the other parameters showed non-linear relationship.

Table 9 shows the best models developed based on deflection basin parameters in SPSS Software. The procedure for developing the model was similar to those expressed in SN_{eff} models.

The model using D_{60} was more reliable for estimating M_R of subgrade. Hence, Model No. 1 (with determination coefficient of 0.92) was selected as the most

Table 8. Correlation of independent variables with M_R

Parameter	$\frac{1}{D_{60}+D_{90}}$	$\frac{1}{D_{90}}$	$\frac{1}{D_{60}}$	D_{150}	D_{120}	D_{90}	D_{60}	BCI	BDI	BLI	D_0
Correlation Factor	0.953	0.867	0.961	–0.668	–0.724	–0.79	–0.796	–0.593	–0.431	–0.328	–0.539
Relationship	L	L	L	N.L	N.L	N.L	N.L	N.L	N.L	N.L	N.L

Table 9. The developed models to determine M_R of subgrade

No.	Models	80% data			20% data		
		a	b	R^2	a	b	R^2
1	$4545.04\times(1/D_{60})+1.76$	1.003	–0.171	0.921	0.999	0.101	0.927
2	$2502.28\times(1/D_{90})+13.24$	0.997	0.169	0.753	1.001	–0.230	0.754
3	$7509.77\times(1/(D_{60}+D_{90}))+2.8$	1.007	–0.482	0.911	1.010	–0.761	0.918

appropriate model to predict M_R of subgrade. This model was also validated accurately in 20% of the data due to low variation in linear regression parameters.

6. Implementation of models at network-level pavement management

Based on results obtained in this research, two regression models were developed to predict SN_{eff} of pavements and M_R of subgrades. These models are presented in Eqns (2) and (3).

$$SN_{eff} = 34.171\times D_0^{-0.638}\times D_{90}^{0.33}; \quad (2)$$

$$M_R = 4545.04\times(1/D_{60})+1.76. \quad (3)$$

Subgrade modulus obtained from Eqn 3 (together with traffic data) were used to calculate SN_{req} according to AASHTO (1993) Pavement Design Guide. Finally, SCI was calculated based on Eqn (1).

Shoush-Andimeshk roadway was selected to describe how the presented method should be applied for decision making at network-level pavement management. SCI values in the selected road were calculated at each point loaded by FWD. These values can be significantly varied point to point along the pavement. Hence, these could not be suitable to determine the type of M&R methods at network-level pavement management. To solve this problem, pavement should be divided into homogenous structural sections. This issue was performed by drawing Cumulative Difference Graph of D_0 versus road change in Figure 5. The sectioning method was derived from the method described in Appendix J of AASHTO Pavement Design Guide (AASHTO 1993).

Each uniform slope in Figure 5 indicates a homogenous pavement structural section. The five sections can be separated based on slope variations. As it can be seen in this figure, Sections 2 and 4 due to their high positive slopes (deflections at these sections were higher than the mean deflection of the whole pavement) were considered weaker than the other sections. *SCI* can be averaged in homogenous sections so that to decide which M&R methods should be assigned to those. Table 10 represents the mean SCI values calculated using the proposed method and that of AASHTO (1993).

As it can be seen in Table 10, *SCI* differences between the two methods were negligible in all sections. Sections 1, 3 and 5 have $SCI \geq 1$ in both methods. Hence, preservation activity can be assigned to these sections. In Sections 2 and 4 where $SCI < 1$ indicates the need for rehabilitation activity. After calculating *SCI* in homogenous structural sections, engineering judgment

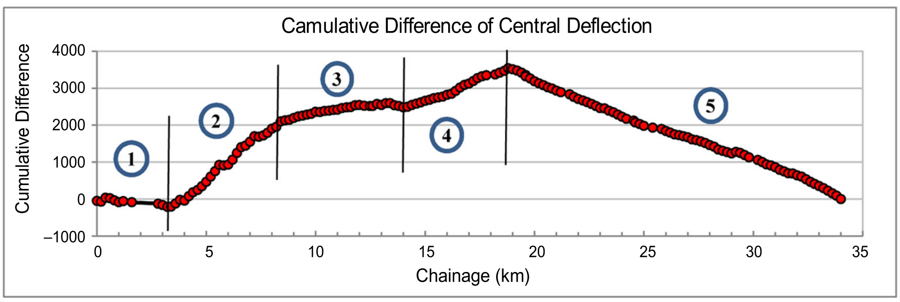

Fig. 5. Cumulative differences of D_0 at Ahvaz-Shoush roadway

Table 10. Comparison of *SCI* values obtained from proposed method and AASHTO (1993)

Sec.	Start	End	*SCI*		M&R of Proposed Method
	(km)		AASHTO	Models	
1	0	3.4	1.4	1.2	preservation
2	3.4	8.4	0.8	0.8	rehabilitation
3	8.4	14	1.1	1	preservation
4	14	19	0.9	0.9	rehabilitation
5	19	34	1.6	1.6	preservation

and considering operational restrictions could help road authorities to finalize sections that should be evaluated at project level.

The proposed method is of great importance at the road network where no previous PMS was implemented. In these conditions, it is necessary to identify current pavement conditions with methods that can be performed with low cost and at a rather short timing.

Conclusions

In this research, *SCI* was suggested for network-level pavement management of flexible pavements. *SCI* determination requires SN_{eff} and SN_{req} (based on AASHTO (1993) method). This study focused on developing two regression models to predict SN_{eff} and M_R. Models were developed based on only DBPs and without having knowledge of pavement layers thicknesses. The findings can be summarized as it follows:

1. COST Model for estimating SN_{eff} was the best among the other models due to its better linear regression parameters.
2. Although *AUPP* and *AREA* parameters are calculated using the same four initial deflections (up to 90 cm), SNeff variations can better be predicted using *AUPP*. This parameter with R_2 of 0.82 had the strongest relationship with SN_{eff} and was the best DBPs.
3. The significant result of this study was to calculate SN_{eff} and M_R using the same deflection basin that belonged to a certain point of the pavement. This resulted in a better comparison between SN_{eff} and SN_{req} in order to determine *SCI* values.
4. Subgrade M_R had the best correlation with the invers of deflection at 60 cm distance from load center with R_2 of more than 0.9. Therefore, it was predictable that among current M_R models, the one that uses D_{90} should be the best with R_2 = 0.76 in comparison with those that used farther deflections to develop MR model.
5. Two regression models were developed to predict SN_{eff} and subgrade M_R without using layer thicknesses. The accuracy of these models was greater than the current models reviewed.
6. The best approach in implementing this method is to divide road network into homogenous structural sections using cumulative difference method of maximum deflections (D_0). Average *SCI* values in each section indicate whether the section is structurally deficient (that needs rehabilitation activities or it requires preservation activities only).

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Amir KAVUSSI. PhD, Associated Professor, Tarbiat Modares University, Department of Civil and Environmental Engineering, Tehran, Iran. Research interests: Asphalt Mixtures and Technology, Pavement Management System (PMS), Non-Destructive Test (NDT).

Mojtaba ABBASGHORBANI. PhD candidate at Road & Transportation Engineering, Tarbiat Modares University, Department of Civil and Environmental Engineering, Tehran, Iran. Research interest: Pavement Management System (PMS), Non-Destructive Test (NDT), Performance Tests.

Fereidoon MOGHADAS NEJAD. PhD, Associated Professor, Amirkabir University of Technology, Department of Civil and Environmental Engineering, Faculty of Geotechnical and Transportation Engineering, Tehran, Iran. Research interest: Asphalt Pavement Modelling, Pavement Management System (PMS), Non-Destructive Test (NDT).

Armin BAMDAD ZIKSARI. MSc in Geotechnical Engineering, Technical Soil and Mechanics Laboratory of Ministry of Road and Urban Development, Tehran, Iran. Research interest: Pavement Management System (PMS), Non-Destructive Test (NDT), FEM Modelling.