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ASSESSMENT OF THE DETERIORATION MODEL FOR ASPHALT CONCRETE PAVEMENT

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ABSTRACT

Rutting is one type of major distress, it exhibits a negative impact to the serviceability characteristics of the flexible pavement, to the residual life of pavement structure and to the safety and ride quality for traffic. Monitoring the accumulation of pavement distress is beneficial in obtaining the deterioration model which can be implemented in the decision of maintenance. Moreover, such model can focus on the properties of materials with sustainable behavior. Simple empirical relationship between the elastic strain and the long-term plastic behavior of pavement materials was established. The aim of this work is to study the in-service behavior of pavement material and relates the pavement layers composition to pavement age. The -expired life since construction- of Asphalt concrete pavement of Mosul-Baji highway was related to gradation variables, Asphalt and voids content and structural properties variables (Marshall properties, and Stiffness). An Asphalt Concrete deterioration model was developed by combining the data from laboratory testing on core and slab specimens obtained from the various layers of the pavement and performing a regression analysis. The model describes the structural deterioration of Asphalt Concrete pavement. It was concluded that such modeling may give a correct basis for treatment planning, project evaluation, management and funding.

Contribution/ Originality: This study contributes in the existing literature, a mathematical deterioration models were developed for each asphalt concrete pavement layer to predict the structural condition based on the gradation parameters and volumetric properties of each layer. Moreover, a combined deterioration model was provided which can be used for maintenance management.

1. INTRODUCTION

Flexible pavements are often subjected to mixed heavy traffic loading and varying environmental conditions which highly influence their performances in terms of distress developments. Pavement design models concentrate on the resilient behavior of the various pavement layers (e.g. tensile strain at the bottom of flexible pavement layer and the vertical compressive strain at the top of the sub-grade) and the service life of the pavement as estimated using empirical formulae relating these strains to the number of standard axil load applications, Boulbibane and Collins [1]. There is significant international activity in pavement deterioration and performance model development, such modeling is essential as a correct basis for treatment selection, project evaluation, strategic planning, management and funding while the useful or expired life of the pavement could be predicted, Madanat, et al. [2]. Iraq is now faced with the tremendous challenge of preserving and enhancing the transportation system. Maintenance plays an essential and integral part in the life of a pavement. Pavements which are left to deteriorate without timely maintenance treatments are likely to require major rehabilitation and reconstruction much sooner than those which are properly maintained. Deterioration models could help in such process, Robert [3]. Three types of deterioration models exist. The first type is the time decay model (time related deterioration of pavement) in which a pavement will deteriorate over time from environmental forces in absence of truck traffic. Thermal cracking, differential heaving due to swelling sub grade, disintegration of surface materials, bleeding, and other climatic/ aging effects of material are largely a function of the environmental zone, and will result in a loss of pavement serviceability, Prozzi and Madanat [4]; Oliver [5]. Rutting of AC is not only dependent on the compressive stress developed on the layers but also the formation of up heals on the side of the wheel track is believed to be due the shear stress developed within the layers just to the outer periphery of the projected area, Feyissa [6]. Deterioration models have been developed by many researchers, Sarsam [7]; Sarsam [8]; Said and Hakim [9]; Sarsam [10]; Sarsam [11]; Haas, et al. [12]. Such deterioration model is likely in the form of the negative exponential function which suggests that pavement condition declines rapidly when initially exposed to the element, but then deteriorates at a decreasing rate over time, WSDot [13]. The second type is the pavement damage model in which the deterioration of pavement can be analyzed with a damage function that relates decline in pavement serviceability to axle load passes which causes accumulation of damage or distress, Pradhan, et al. [14]. The third type is a combination of time and traffic effects which causes deterioration. Misra and Roohanirad [15] showed a mathematical model which correlates the pavement age in years with the pavement condition index. Loizos, et al. [16] studied the structural strength degradation of the pavement at mild climatic condition and compares the model with HDM-4 models. Ullidtz [17]; Sarsam [18] presents theoretical models which simulates the deterioration of a section of pavement over time and considers the variation of pavement materials, layer thickness and traffic loads as well as seasonal variation. In this study, the composition of materials in each pavement layer, and the structural properties and pavement age has been utilized in the formation of deterioration model.

2. ROAD PERFORMANCE HISTORY

Mosul- Baji road (section one) is a two-lane highway of 40 Km length, it is a part of highway No.1 (Mosul-Baghdad). The typical cross-section of the highway is of 6cm Asphalt Concrete binder course, 10cm of Asphalt stabilized base course, 50cm granular sub-base course with 40% CBR value and 15cm of compacted sub-grade with 4% CBR value. It was constructed and opened to traffic at the early1970. It was subjected to surfacing with another Asphalt Concrete binder course of 6cm thickness during the mid of 1977. During June-1986, the pavement shows block cracking, then it was subjected to Benkelman beam test and the rebound deflection was measured, it was in the range of (98-92) %, then the highway was resurfaced with another Asphalt Concrete binder course of 6cm thickness at 1988.

3. FIELD ASSESSMENT

The highway experiences sever rutting and block cracking during 1989, and an extensive study on this failure took place by the author, slab and core samples have been obtained from nine selected locations of the defected area of the highway representing different road geometrics. Core samples were cut to the full depth of asphalt concrete layers along the road cross section at successive increments of 20-30 cm starting from the pavement edge of the slow lane. Samples were tested for Marshall Properties, field density, voids, gradation analysis, Asphalt content and specific gravity and analyzed. Figure 1 illustrates typical profiles along the study section taken at the coring locations; it shows that the road section studied shows a variation of more than 6 cm of pavement surface level through the cross section between the adjacent longitudinal sections; this may clearly represent the functional failure of the highway.

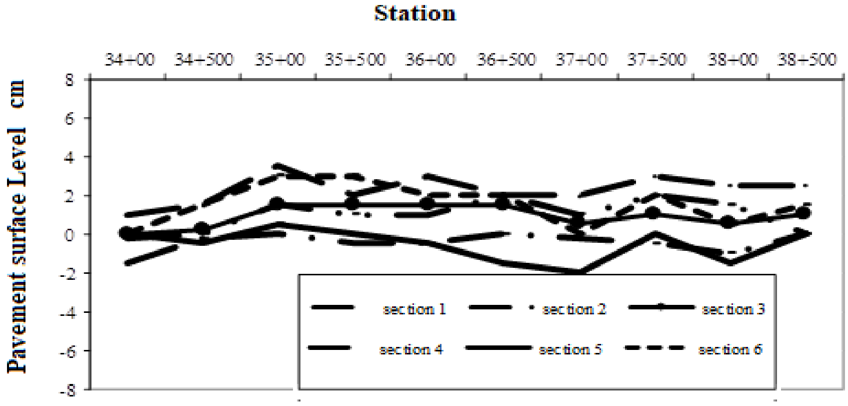

Figure-1. Typical profiles along the study area.

Source: Surveyed by the author and plotted from field data.

Figure 2 illustrates typical cross sections of the tested area at station 34+000 to station 34+500 (horizontal curve turning right); it can be observed the rutting starts at asphalt stabilized base course at station 34+000, the binder course was badly distressed by shear failure and the thickness of the course increased between the wheel path. On the other hand, after 500 meters from the first section, more distress could be observed at the base course and even the failure of subbase could be observed. Surfacing and resurfacing action seems not to be a perfect solution for that section rather than milling the underneath binder and base layers and maintaining and leveling the subbase layer. The variation in the stiffness of surface and resurfacing courses for both sections is another evidence of the distress. Finally, the over compaction of the binder and surface courses under the wheel paths is observed indication poor stiffness and shear failure of the layers. It can be noted that the variation in asphalt concrete stiffness through and along the tested sections, and the percent field compaction variation for different layers could explain the rutting phenomena.

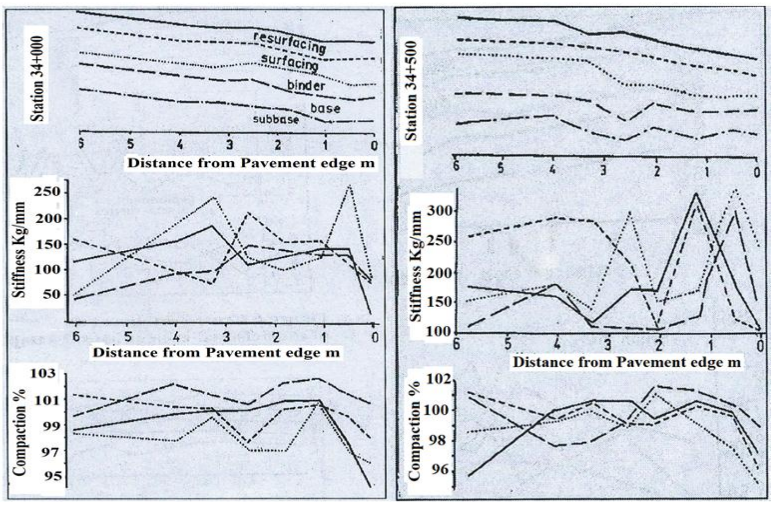

Figure-2. Typical distress at stations 34+000 to station 34+500.

Figure 3 exhibit typical distress at stations 35+500 (straight section), it can be observed that subbase layer is responsible for the shear failure of asphalt stabilized base course. Shear failure was also extended to the binder, surfacing and resurfacing courses. This finding was further supported with the great variation of the stiffness among the cross section of the surface course. This may be attributed to the possible non homogeneity of asphalt concrete material and possible segregation during the spreading process of surface course. On the other hand, over compaction could be noted for base and binder courses. It can be concluded that Surfacing and resurfacing action were not the perfect solution for that section rather than milling the underneath binder and base layers and maintaining and leveling the subbase layer.

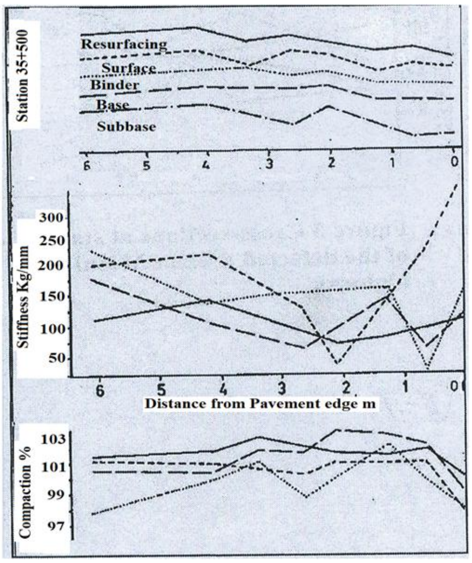

Figure-3. Typical distress at stations 35+500.

Figure 4 exhibit typical distress at stations 36+000 to Station 36+500 (straight section – fill zone), a moderate rutting could be observed at the subbase course while it was extended to the upper asphalt concrete layers. Extensive over compaction could be noted for the whole asphalt concrete layers at the starting station of the section. The quality of binder and surface courses was poor from the homogeneity through the cross-section point of view. This was further supported by the variation in the stiffness of asphalt concrete mixture throughout the section.

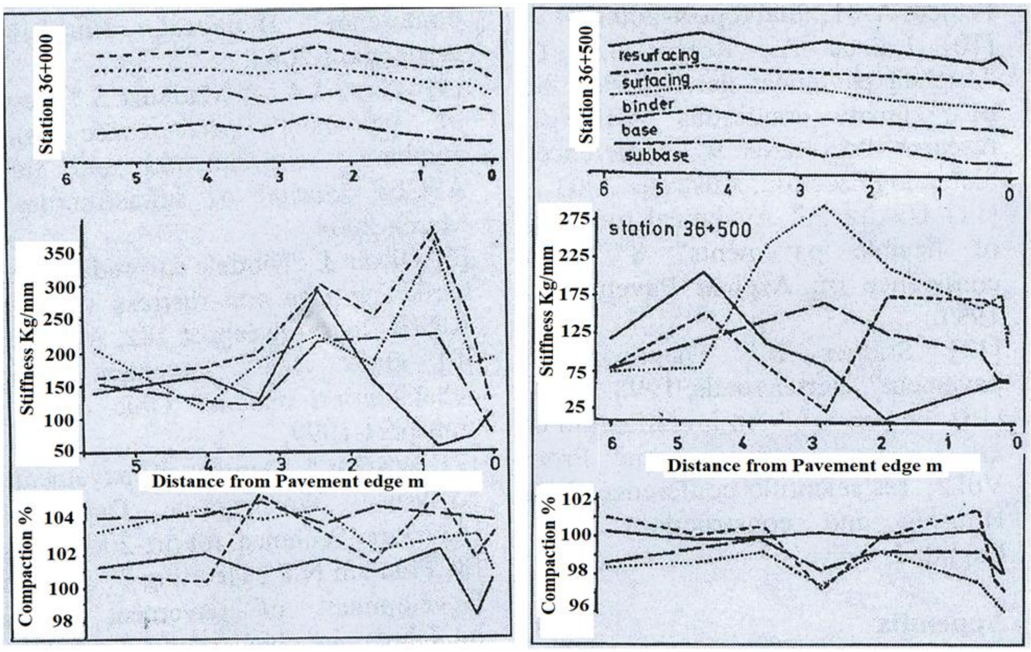

Figure-4. Typical distress at stations 36+000 to station 36+500.

Figure 5 demonstrates typical distress at stations 37+000 to Station 37+500 (horizontal curve turning right), it can be observed that the asphalt concrete surface and resurfacing courses are responsible for the rutting in the section. The great variation in the stiffness of such layers indicates poor homogeneity throughout the section. This was further supported by the over compaction exhibited at the wheel paths.

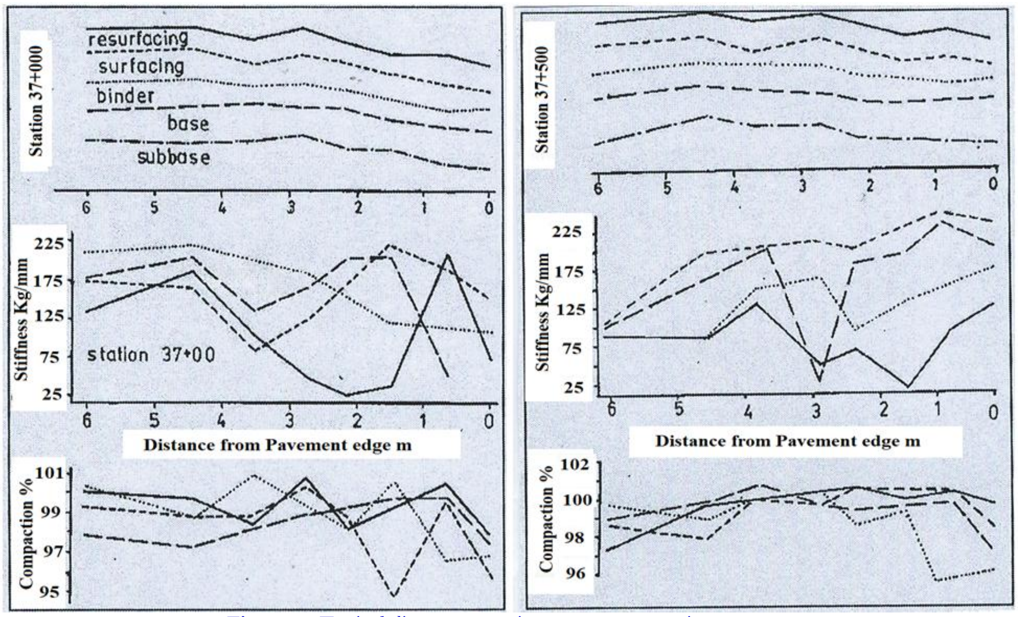

Figure-5. Typical distress at stations 37+000 to station 37+500.

Figure 6 shows typical distress at stations 38+000 to Station 38+500 (straight section – cut zone), poor subbase finished level at the start of the section could be considered as the major cause of the rutting at the overlaying asphalt concrete layers. Moreover, the great variation in the stiffness of binder and surface courses throughout the cross-section is referring to the shear failure which causes the rutting. The over compaction especially at the start of the section is another cause of the pavement distress.

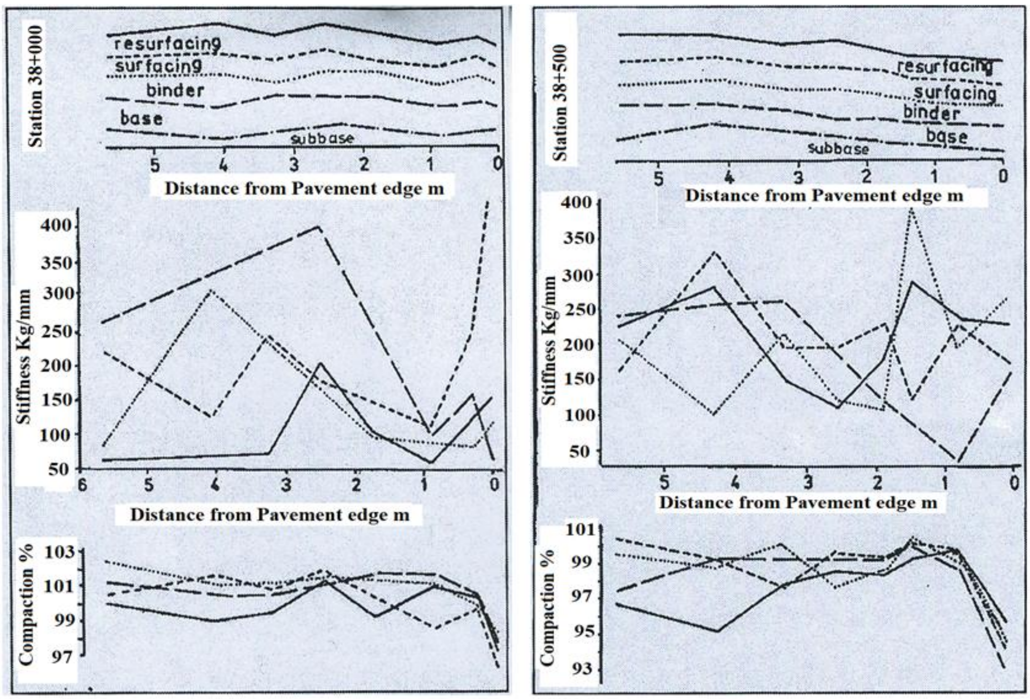

Figure-6. Typical distress at stations 38+000 to station 38+500.

The extensive rutting shown throughout the pavement sections may be attributed to the defect in asphalt concrete layers and the subbase layer. It was felt that such rutting may be responsible of the variation in asphalt concrete layers thickness as the material will flow under the extensive loading of trucks to the sides of the wheel paths due to insufficient tensile stress. Such variation could be attributed to pavement age, construction practice, and material composition, and it worth an analysis by computer to verify possible correlation. Table 1 illustrates the structural strength variables for various Asphalt Concrete layers. A mathematical model was obtained which correlates the rut depth with materials variables for the whole pavement sections, such data and the rutting model were published elsewhere, Sarsam [18]. The data were utilized in this paper to predict the deterioration models.

Table-1. Modulus and stiffness properties of asphalt concrete.

Age (month)	Modulus (MPa)	Stiffness (Kg/mm)	Layer
246	5.5 x 10^3	488	Asphalt stabilized base course
240	6.8 x 10^3	413	Binder course
150	8.9 x 10^3	408	Surface course
9	18.6 x10^3	404	Resurfacing course

4. PREDICTION AND VERIFICATION OF DETERIORATION MODEL

The pavement deterioration model utilizes the structural data obtained from Sarsam [11] the input to this model includes parameters that are similar to those used for specification of materials and quality control such as grain size distribution variables, voids and Asphalt content. Table 2 demonstrates the range of Asphalt Concrete variables studied. At the beginning, the performance of each of the four pavement layers as represented by the structural strength has been differentiated in accordance with material type and composition using linear regression analysis.

Table-2. Range of asphalt concrete variables studied.

Property	Maximum	Minimum
Marshall stiffness (Kg/mm)	792	259
Marshall stability (Kg)	2331	1004
Marshall flow(mm)	4.9	2.2
Asphalt content (%)	6.8	4
Bulk specific gravity	2.423	2.321
Vv (%)	7.1	2.2
Cu= D60/D10	75.1	26
Cc = $D30^2/D60\times D10$	5.4	0.3
Cg = D50/D80	0.6	0.2
Cgg = D80/D50	3.7	1.4
Age (months)	246	9

4.1. Model Development

The first step in the development of statistical model is the collection of data, then specifying the dependent and independent variables. The dependent variable was the structural strength of the pavement (stiffness) which was calculated from Marshall test on slab and core specimens. Marshall stiffness was calculated by dividing the Marshall stability values by the corresponding flow values. The independent variables include gradation parameters (coefficient of uniformity Cu, coefficient of curvature Cc, coefficient of gradation Cg, and coefficient of degradation Cgg); Asphalt content, Voids content, Specific gravity of the mix and the expired life of the pavement layers since construction (pavement age). The final structural models for the pavement layers were presented in Table 3.

Table-3. Predicted structural strength models.

Layer	Mathematical form of deterioration model	R^2	SEE
Asphalt stabilized base course	$Y = C_1 + a_1v_3 + b_1v_4 + c_1v_5 - d_1v_6 - e_1v_7 + f_1v_8 + g_1v_9 + h_1v_{10} - i_1v_{11}$	0.951	30.2
Binder course	$Y = C_2 + a_2v_3 - b_2v_4 - c_2v_5 - d_2v_6 - e_2v_7 + f_2v_8 - g_2v_9 + h_2v_{10} - i_2v_{11}$	0.997	24.6
Surface course	$Y = C_3 - a_3v_3 - b_3v_4 - c_3v_5 + d_3v_6 - e_3v_7 - f_3v_8 - g_3v_9 + h_3v_{10} - i_3v_{11}$	0.989	23.3
Resurfacing course	$Y = - C4 - a_4v_3 + b_4v_4 + c_4v_5 + d_4v_6 - e_4v_7 + f_4v_8 + g_4v_9 + h_4v_{10} + i_4v_{11}$	0.999	20.2

4.2. Model Selection

Selecting model of the right form to fit a set of data usually requires the use of empirical evidence in the data, some trial and error experimentation, and knowledge of the shapes for various Asphalt Concrete layers that different mathematical functions can assume. Referring to the literature, Loizos, et al. [16]; Oliver [5]; Sarsam [8] is important to have an idea about the models that have been found to work well for different applications. An Empirical model was selected which can hold substantial number of field measurements that are normalized by the model and thus tends to reflect average condition. The data from the laboratory testing of each of the four layers were combined and subjected to linear stepwise regression analysis using the SPSS-16 software. The dependent variable (stiffness) was related to more than one independent variable, and then Asphalt Concrete pavement deterioration model could be developed and is represented as given in Table 4.

Table-4. Deterioration model of asphalt concrete pavement.

Deterioration model	R^2	SEE
$Y = - C5 + a_5V_2 + b_5V_3 + c_5V_4 - d_5V_5 - e_5V_6 - f_5V_7 + g_5V_8 + h_5V_9 + i_5V_{10} - jV_{11}$	0.975	29.2

5. DISCUSSION AND LIMITATIONS OF ASPHALT CONCRETE PAVEMENT DETERIORATION MODEL

Critical pavement construction details such as adequate compaction effort may be represented in this model by voids content in each layer. It was observed that the pavement structural responses are highly dependent on modulus of Asphalt Concrete as shown in Figure 7 the rate of deterioration increases gently at the early age of pavement, and then it increases at faster rate as it reaches the expected design life of the pavement.

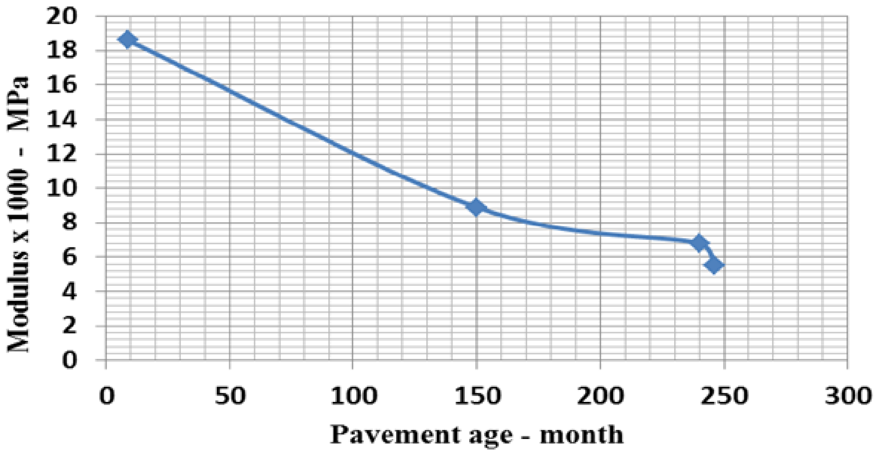

Figure-7. Structural deterioration of pavement.

The pavement material composition such as gradation and Asphalt content has a major impact on structural responses. Table 4 shows that the structural strength degradation of the pavement is highly dependent on construction practice parameters such as specific gravity and voids content. The higher constant value gives the lesser value of the squared coefficient of determination (R^2). The higher number of constant as illustrated in Table 5 provides more error to the pavement deterioration model.

Table-5. Details of mathematical models' symbols.

Variable details	Constants
Y= Marshall stiffness (Kg/mm)	C1= 6016.5
V2= Pavement age (months)	C2=3420.3
V3= Asphalt content (%)	C3= 2360.6
V4= Bulk specific gravity	C4= 4279.6
V5= Voids content (%)	C5= 186.2
V6= Coefficient of uniformity, Cu = D60/D10	
V7= Coefficient of curvature, Cc= $D30^2/D60 \times D10$	
V8= Coefficient of gradation, Cg = D50/D80	
V9= Coefficient of degradation, Cgg = D80/D50	
V10= Marshall stability (Kg)	
V11= Marshall flow (mm)	

The alphabetic symbols (a, b, c, d, e, f, g, h, I, j) in Table 6 are the regression coefficients representing the amount that dependent variable changes when the corresponding independent variables change.

Like any other deterioration model, Prozzi and Madanat [4]; Robert [3]; Pradhan, et al. [14]; Sarsam [18] and Sarsam [10] the model developed in this research is only an approximation of the actual physical phenomenon of deterioration. There could be a predicted error associated with the model; however, this error can be estimated through the experience of the estimator to assess the uncertainty in the prediction.

Table-6. The constants details of the developed models.

Constant	Constant	Constant	Constant	Constant
a1= 105.9	a2= 197.4	a3= 8.8	a4= 95.9	a5= 0.0087
b1= 2383.8	b2= 1556.5	b3= 626	b4= 1663.1	b5= 4.6
c1= 26.7	c2= 18.2	c3= 7.7	c4= 25.8	c5= 252.3
d1= 0.319	d2= 18.8	d3= 0.553	d4= 3	d5= 4.04
e1= 6.4	e2= 9.6	e3= 10.7	e4= 15	e5= 0.25
f1= 524.7	f2= 1989.9	f3= 165.2	f4= 718.9	f5= 1.05
g1= 48.2	g2= 311.5	g3= 127.7	g4= 179.8	g5= 1.89
h1= 0.286	h2= 0.497	h3= 0.273	h4= 0.154	h5= 11.98
i1= 153	i2= 194.5	i3= 117.6	i4= 8.57	i5= 0.276
j = 128.5				

The difference between the assumptions on which the theoretical models are based, and the reality of pavement materials and structure are clearly indicated in Figure 8 which shows the relation between measured and predicted Asphalt concrete stiffness.

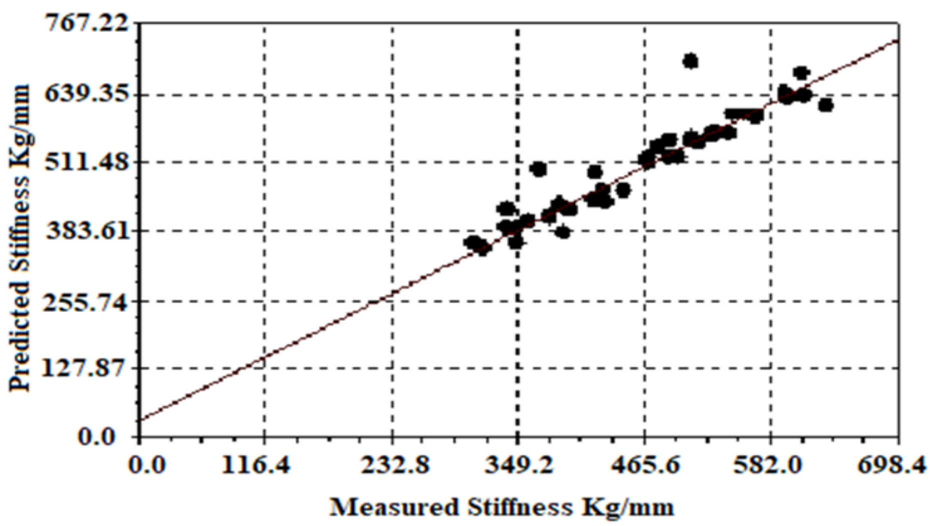

Figure-8. Measured and predicted stiffness using the developed deterioration model.

6. CONCLUSIONS

It could be concluded that the homogeneity of subbase or asphalt concrete mixture throughout the pavement section in vital in restricting the shear failure and rutting. The developed pavement deterioration model for Mosul-Baji highway is capable for predicting the degradation in Asphalt Concrete stiffness through the useful life of the pavement. Such modeling may give a correct basis for treatment planning, project evaluation, management and funding.

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