© Universiti Tun Hussein Onn Malaysia Publisher's Office

IJIE

Journal homepage: http://penerbit.uthm.edu.my/ojs/index.php/ijie
ISSN : 2229-838X e-ISSN : 2600-7916

The International Journal of Integrated Engineering

Prediction of Pavement Life Using Influence Function and Peak Influence Function for Mechanistic-Empirical Pavement Design Analysis

Rosnawati Buhari[1*], Mustafa Kamal Shamsuddin[1], Saifullizam Puteh[2], Siti Khatijah Abu Bakar[1]

[1]Smart Driving Research Centre, Faculty of Civil Engineering and Built-Environment, Universiti Tun Hussein Onn Malaysia, 86400 Parit Raja, Johor, MALAYSIA

[2] Faculty of Technical and Vocational Education, Universiti Tun Hussein Onn Malaysia, 86400 Parit Raja, Johor, MALAYSIA

*Corresponding Author

DOI: https://doi.org/10.30880/ijie.2020.12.01.023
Received 08 March 2019; Accepted 26 October 2019; Available online 16 February 2020

Abstract: The aim of this study is to predict the long-term pavement life using different method of analysis of the primary response in the pavement layer. The methods are influence function method (IFM) and Peak Influence Function Method (PIFM). A tyre-pavement interaction model was used to predict the 3-Dimensional contact stresses under static and dynamic loads. In this model, a truck tyre was loaded on a three-layer flexible pavement surface. The load primarily affects the vertical contact stress and the longitudinal contact stress. Statistical method was used to analyse the compare the differences between both methods. As a result, elastic modulus of the asphaltic material varies with asphaltic concrete thickness, vehicle speed, volume of void, volume of bitumen and temperature. Temperature was found to influence the asphaltic material modulus more than others due to the elastic modulus of the mixture's dependency on the stiffness of the bitumen. It is succeeded by vehicle speed, volume of void, volume of binder and AC thickness of which is minimal or have almost no effect on strains. In conclusion, Peak influence function is able to predict the response in pavement as a result of vehicle loads sufficiently and realistically. Low differences of strain was evidenced to reduce the differences resulting in failure.

Keywords: Pavement life, influence function, peak influence function, pavement design

1. Introduction

Increase number of axles and high tyre pressure from heavy vehicles resulted traffic-related pavement distresses. The load that produced horizontal stresses induced between the layers eventually result in crack formation and any local settlements that leading to asphalt layers cracking [1]. Other than cracking, several distresses are generally occurring due to high axle load are rutting, revelling etc. Previously in empirical pavement design, the stresses/strain or primary response analysis in pavement where it is significantly affected to the pavement life performance is not seriously considered [2]. Therefore, one step forward was having taken to amend the pavement design guide from empirical pavement design to Mechanistic-Empirical pavement design (MEPDG).

In MEPDG, evaluation of pavement response to vehicular loading is a very important consideration to predict the most accurate pavement performance as well as to guarantee optimum performance during the design life of the pavement. Three key material parameters of relevance to the analytical design or evaluation of a flexible bituminous pavement are stiffness modulus of the materials, deformation characteristics and pavement performance are considered.

*Corresponding author: rosna@uthm.edu.my

The main consideration for designing a flexible pavement is limit the tensile strain at the bottom of the asphaltic layer and compression strain on top of the subgrade layer. These two responses reflect fatigue cracking and permanent deformation of the pavement which are the most critical damage mechanisms for pavement.

There are two modes of crack origination and progression includes top-down or down-top. Conventionally, cracking of the asphaltic layers arises from the tensile strain developed in bound layers as each wheel load passes. It is therefore a function of both the magnitude of tensile strain and the number of load applications. It consists of two main phases, crack initiation and crack propagation. It is caused by tensile strains generated in the pavement by not only traffic loading but also temperature variations and construction practice [3]. Cracking that initiates at the bottom of the layer is more critical because such cracking is not visible until the crack reaches the top of the layer. For example, in thinner pavements, cracking is commonly assumed to initiate at the bottom of the bound layer where the maximum tensile stress and strain occur, and then propagate upwards under repeated load applications. Although, it is agreed by several researchers that, generally cracks originate at the top surface of the layer and propagate towards the bottom and this crack is not fit with the classical. The general relationship defining the fatigue life based on crack initiation is as follows [4]-[6]:

$$N_f = k_1 (\varepsilon_t)^{-k2} \tag{1}$$

where N_f is number of applications of load to initiate fatigue cracking, ε_t is tensile strain at the bottom of the asphaltic layer and $k_1$and k_2 are constants.

Rutting is associated with permanent deformation of all the pavement layers, and is an accumulation of the small irrecoverable part of the deformation caused by each wheel load. It is a function of the visco-elastic nature of the bitumen, the mechanical support offered by the grading of the aggregate and the supporting ability of the underlying subgrade. The general relationship defining the fatigue life based on crack initiation is as follows [4]-[6]:

$$N_r = k_3 (\varepsilon_c)^{k4} \tag{2}$$

where N_r is number of applications of load to initiate fatigue cracking, ε_c is compression strain on top of subgrade and k_3 and k_4 are constants.

2. Methodology

In this study, quarter truck model was use to simulate the static and dynamic wheel loads. Qurter-truck parameter values used is corresponds to dual axles fitted with steel suspensions loaded with 2.4 tonnes. Truck speed is kept constant at 20 m/s. The Road with IRI=2 is used [7-9]. The load applied on to the axisymmetric model is in the form of stress pulsed vertically to the centre of the model. The Influence function method (*IFM*) and Peak Influence Function Method (*PIFM*) were used to simulate the critical responses on the two layer of pavement structure consist of asphaltic concrete and granular subbase laid on the subgrade soil. The *PIFM* and *IFM* will be based on the previous study done by Buhari et al. [8]. In accordance, the response at a particular location as a function of time, *t*, based on *PIFM* can be written up as follows:

$$Y(x,t) = F(t) I(V, x - Vt) \tag{3}$$

and for IFM as follows:

$$Y(x) = F(x) I(V, x) \tag{4}$$

where *l(V,x-Vt)* is the influence function for speed, *V* and (x-Vt) is distance from the point of load applied. The analytical response analysis using IFM is far more complicated and time consuming compared to the PIFM [8]. For smooth road surface with the same pavement structure and material characteristic, via the employment of both methods, the maximum influence function vertically under the center of the load is similar along the load. However, for rough road surface, they might differ. It is known that the dynamic behavior of the vehicle will tend to be most responsive to excitation from tyre and wheel. Rougher pavement surface tends to increase the amplitude of dynamic forces. In this condition, response at a particular point might prevail over the response from the vicinity of the points [10].

The evaluation of this method with other theoretically different methods for determining stresses, strains and deflections in pavement structures was followed Ullidtz & Ekdahl [11]-[12]. It was found that the simple method based on Odemark'stransformation and Boussinesq's equation claimed to be as good as, or, better than the more sophisticated methods employing the Finite Element Method. On that same note, they also claimed that the difference between theoretical assumptions of continuum mechanics and the real conditions of particulate pavement materials are so imperative that there is no improvement to be obtained through the use of more sophisticated methods. They concluded that other than using a method based on more appropriate assumption available for pavement analysis, the simple methods are to be favored.

The elastic modulus of the bituminous binder used is based on the Van der Poel nomograph which is the simplest constitutive model for an asphalt mixture named theory of linear elasticity [10]. In this theory, materials are usually assumed to be homogeneous, isotropic and linear elastic and characterized by time-independent constants of proportionality between stress and strain. These kinds of materials are described as elastic, where the loading curve is identical to the unloading curve. Several other assumptions made in this method include: (i) all layers are assumed to be infinite in lateral extent, (ii) all layers have finite thickness except for sub-grade assumed to be infinite and, (iii) all materials are weightless [12-13]. Linear elastic theory assumes linear behavior over any stress range and that material response is non-viscous and all the deformation is recoverable. According to Van der Poel, for asphaltic materials, Young's modulus was replaced by a 'stiffness modulus' since the stiffness varied with loading durations and temperatures. Using these limitations, the stresses calculated for linear elastic theory are reasonable and may be used as the basis for pavement design. A few researchers such as Peutz et al. and Eisenmann et al. successfully applied linear elastic models in flexible pavement design as well as performance evaluation in the past [13-14]. At present, the elastic model is employed by numerous organisation, including Shell in the BISAR computer programme for pavement structure analysis. Moreover, Van Der Poel introduced the concept of 'stiffness' to describe the behaviour of pure bitumen as a function of loading time and temperature [15]. He indicated that the stiffness of bitumen at lower strains could correlate with the penetration index and softening point of the bitumen. The research resulted in the highly acclaimed Van der Poel nomograph, by means of which, the deformation of bitumen can be calculated as a function of stress, time and temperature. The significance of the temperature's effect on stiffness was reported to be similar to that of the strain effects on fatigue life and was more prominent than other material variables [16].

The pavement system was simulated using the methods introduced by Ullidtz [11]. He was produced the simplest way of describing a pavement system that is, of a half space. Material properties of each layer are homogeneous. Each layer except the bottom layer has finite thickness and is modelled as a semi-infinite solid. All layers are infinite in the horizontal direction, all layers are isotropic, which means that the material properties are the same in every direction, surface shearing forces are not present on the surface and each layer is characterised by two properties: Poisson's ratio and the elastic modulus. The Method of Equivalent Thickness (MET) principle denotes the transformation of a system consisting of layers with different moduli into an equivalent system where all layers have the same modulus.
Boussinesq's equation is plausible for application in this method.

Other than that analysis for dual layer system was using Odemark Method. For calculating stresses or strains above an interface or, within the compression of the layer over the interface, the system is treated as a half-space with a modulus of a first layer. For calculating stresses and strains at or below the interface of the layer and vertical stress or horizontal strain at the bottom of the upper layer, the interface above is transformed to an equivalent layer with modulus of second layer and Poisson's ratio but with the same stiffness as the original layer [17].

Eqs (1) and (2) were used to simulate the pavement distress occurred in the pavement. The major factors affect the constants $k1$ and $k2$ are the volumetric proportion of binder, VB and Initial Softening Point, *TRBI*. The constant values can be estimated from the equation given in Brown and Brunton as follow [17]:

$$\log k1 = 14.39 \log VB + 24.2 \log TRBI - 46.06 \tag{5}$$

$$k2 - 5.13 \log VB + 8.63 TRBI - 15.8 \tag{6}$$

Alternatively, the following equation may be used:

$$\log \varepsilon_t = \frac{14.39 \log VB + 24.2 \log SPi - k - \log N}{5.13 \log VB + 8.63 \log SPi - 15.8} \tag{7}$$

where *k* is equal to 46.82 for life to critical conditions and *k* equal to 46.06 for life to failure.

3. Results and Discussion

Various stress produced by differing vehicle loads spread throughout the pavement and the stress distribution effect on radial strains at the base of the asphaltic layer as well as vertical strain on top of the sub-grade soil as a function of distance are shown in Fig. 1. The pavement structures and material properties used in the analysis have been previously explained in the prior section. As demonstrated, tensile strain at the bottom of the asphalt layer is lesser than the compressive strain at the top of the subgrade.

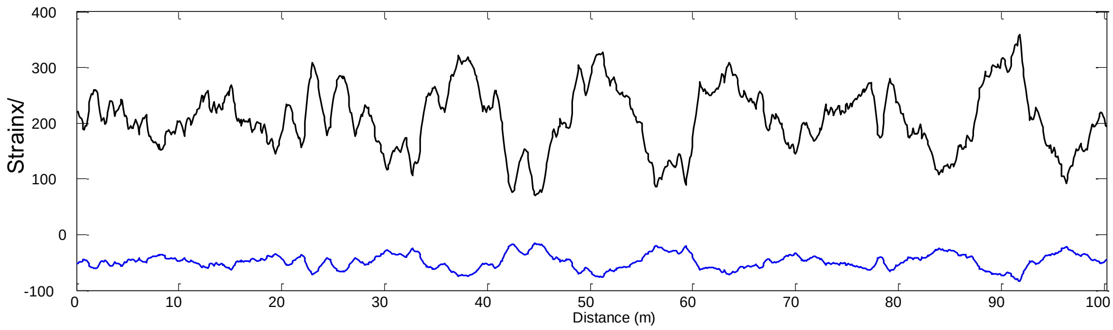


Fig. 1 - Compression strain on the top of sub-grade layer (line on top) and tensile strain at the bottom of asphaltic concrete (line at bottom) using Road 1

3.1 Critical Strain in the Pavement

Fig. 2 illustrates the critical strains on top of sub-grade using the IFM. It was found that by arranging the influence function at all points along the road then, multiplying each point with the forces generated by the vehicles travelling 20 m/s at each point. To justify; subject to the line with the square marker, the maximum point represented by (i) is the maximum strain generated by the load on point (i) at time, *t* and the strain noted as (i+20) is the strain generated influence by the load at point (i) at time *t*.

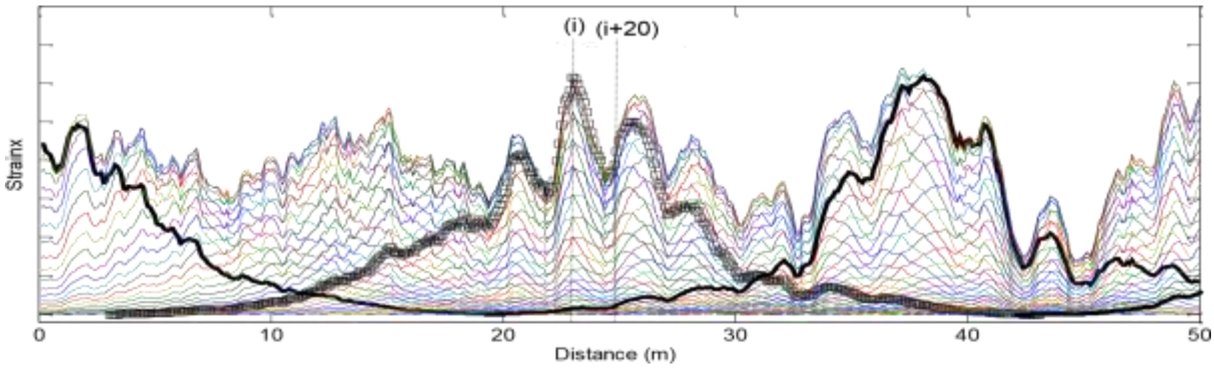


Fig. 2 - Illustration of strains at bottom of bituminous layer using *IFM* results using Road 1

Table 1 - Pavement properties variations. Road 1 using tahe values in the bracket

Pavement layer	*h* (mm)	Air void, v_v	Binder %, v_b	Modulus (GPa)
Asphaltic layer	150 (200) 250 300 350	(4), 6, 8,10, 12	4, 6, (8),10,12	E_1
Granular Sub-base	200	-	-	(E_2=0.1E_1) E_2=0.02E_1 E_2=0.01E_1
Subgrade Soil	∞	-	-	E_3 (CBR- 5%)

Comparing both the methods, the contrariety of the influence function defined using IFM and PIFM for 100 m length of road is able to be identified. The implicit values of the differences between the influence functions at vertical under load at each point along the 100 m road length that were computed in the employment of both methods were defined by the following equation.

$$P_{dif} = \left[\frac{(if_1 - if_2)}{if_1}\right] \times 100 \tag{8}$$

where P_{dif} is percentage difference between strains, if_1 is critical strain determined using IFM at subject point and if_2 is critical strain determined using PIFM at subjected point. Fig. 3 and Fig. 4 establish the comparison of strains using IFM and PIFM**.** A positive value means that at the point where there is a difference, the strain prevails by the strain at the nearest point at the right or at the left. As shown in the figure, differences of the strain at the bottom of the asphaltic concrete layer are smaller that is, the maximum difference stands at less than 30%. Almost all differentiations are lower than 10%. Although, in average, the differences along each road is lower than 20%.

Overall, with the use of both methods, differences between the strains at the bottom of asphalt concrete can be said to be lower than the difference of that on top of sub-grade. This is owed to the curve of the strain at the bottom of an asphaltic concrete layer is narrower than the influence function curve at the top of the sub-grade. Subsequently, the possibility of the influence from the load at the nearest point to overshadow the maximum influence at the subject point is lesser than the influence at the top of the sub-grade layer. This phenomenon becomes more apparent when the road surface is rougher.

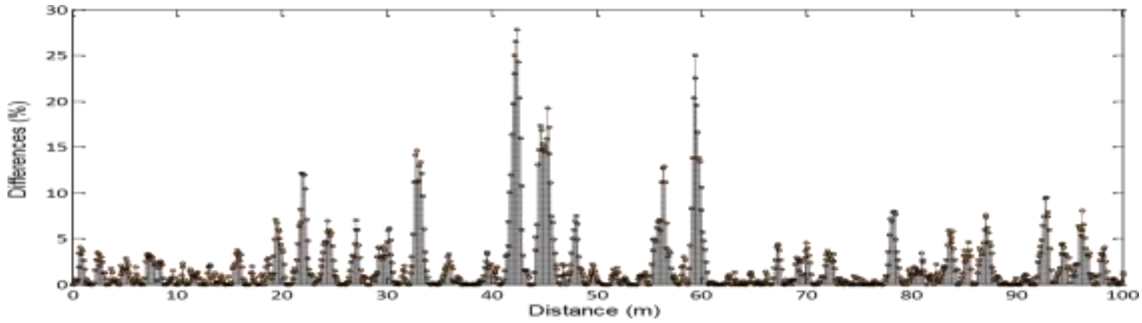


Fig. 3 - Percentage differences of tensile strain at bottom of asphaltic layer using IFM and PIFM. Quarter-truck model, a truck of single axle with steel suspension travelling on Road 1 at truck speed of 20m/s

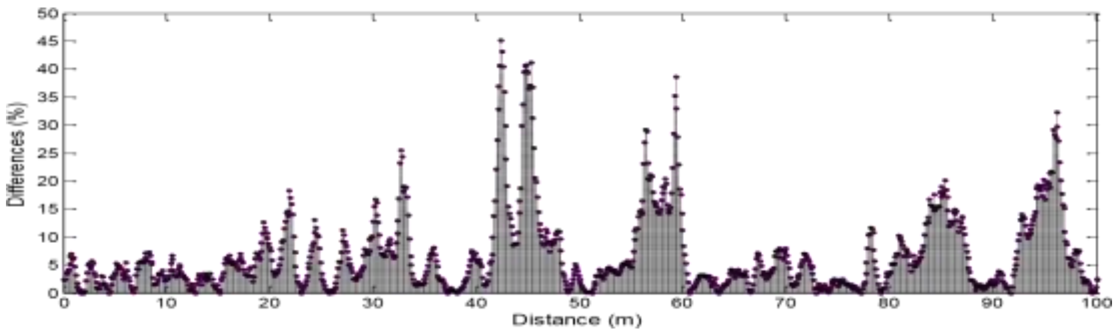


Fig. 4 - Percentage differences of compression strain on top of sub-grade layer using *IFM* and *PIFM*. Quartertruck model, a truck of single axle with steel suspension travelling on Road 1 at a speed of 20 m/s

3.2 Pavement life using IFM and PIFM

Two failure mechanisms were considered in this study that is fatigue due to tensile strain at the bottom of asphaltic layer and rutting due to compressive strain on top of subgrade layer. Eq. (1) is used in this analysis. Fig. 5(a) shows the differences in number of allowable trucks to rutting using *IFM* and *PIFM* for 20 m distance in percentages. That is considering by showing the 100 m results may not showing the gap between the lines due to very small differences. Close up the results within 20 m distance, the maximum difference is approximately 11.8% about 14.5 m from the origin. Compare to the differences in fatigue failure as shows in Fig. 5(b), the maximum differences is significantly lower than the differences in rutting failure that is only 6.5 %. This is due to the fact that the fatigue influence function curve is narrower than the rutting influence function curve. Also shown the figure, among the percentage differences, the higher differences are determined at the points that have significant increasing and decreasing slope between the points of the road or might considered as rougher part of the road.

Fig. 6(a-g) shows the percentage differences between numbers of allowable cycles to the rutting generated by dual axle with steel suspension travelling on several roads at speed 20 m/s, using both methods. As seen in the Fig. 6(a-e), comparison the differences of the number of cycles to rutting in percentage results from the travelling vehicles on the roads that have similar IRI=2, the maximum differences computes are less than 15% and the difference at almost all the points is about 5%. As expected, differences are increasing when road roughness is higher as seen in Fig. 6(a) and Fig. 6 (f-g). The maximum differences then higher than 15% and majority of the differences are almost achieving 10%.

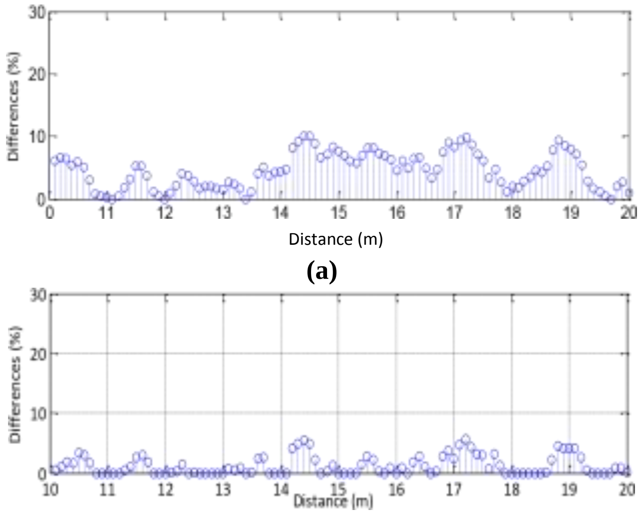
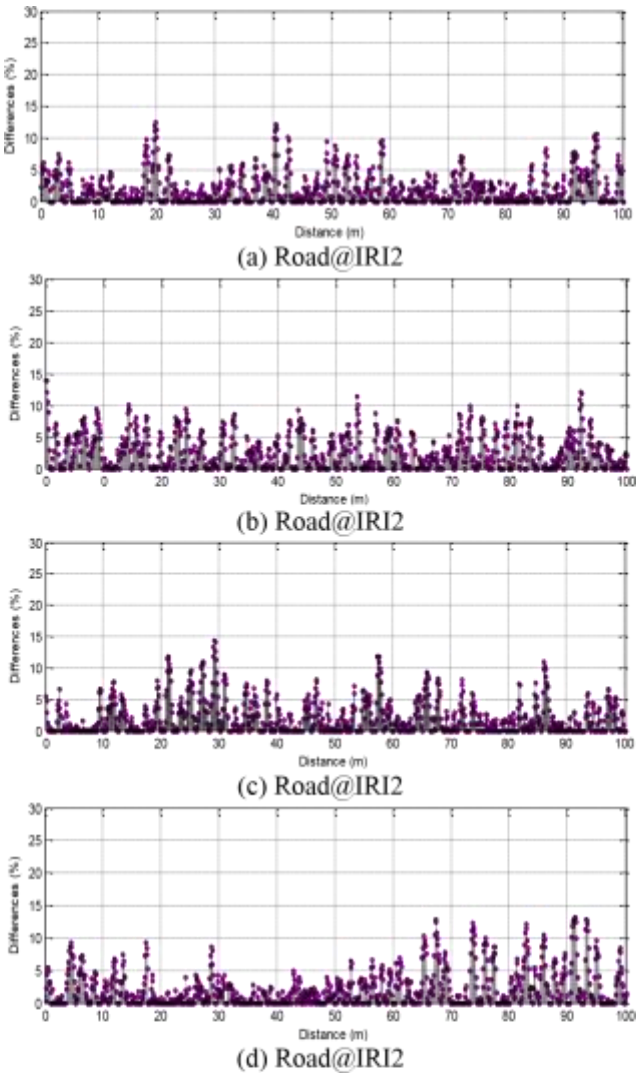
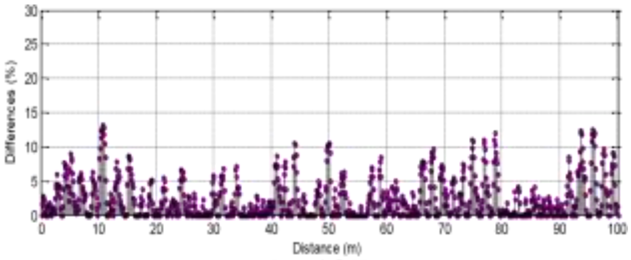
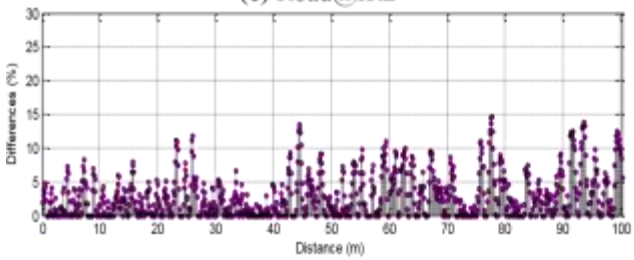


Fig. 5 - Comparison differences of number of allowable cycles to failure calculated using IFM and PIFM for 20 m road length; (a) Percentage differences of the number of allowable cycles to rutting and (b) Percentage differences of the number of allowable to fatigue on each point using both methods

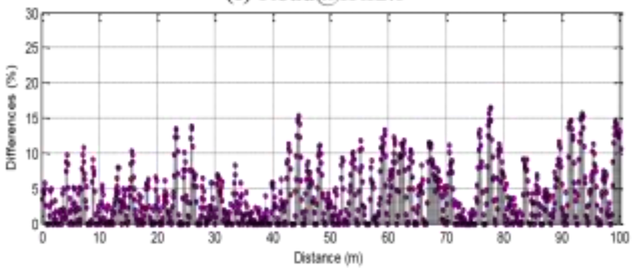




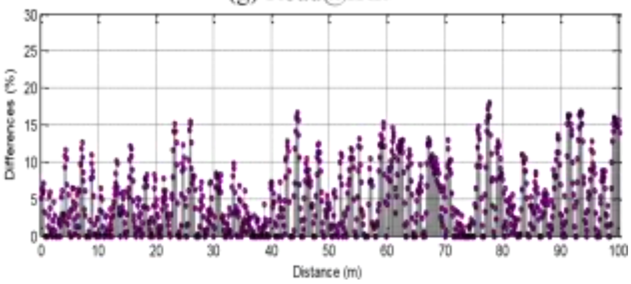
(e) Road@IR2



(f) Road@IRI2.5



(g) Road@IRI3



(h) Road@IRI3.5

Fig. 6 - Percentage differences of number of allowable to rutting generated by the vehicle travelling on several roads using IFM and PIFM

3.3 Sensitivity Study

The sensitivity analysis was performed to examine the effect of IFM used to predict pavement performance. The analysis encompasses a broad range of air temperature, vehicle speed, air void, volume of bitumen and asphaltic concrete thickness as results in Fig. 7.

It is seen from the figure that the maximum variance is lower for the higher temperature. This is because a higher compression strain caused by a higher temperature is narrower than a compression strain caused by a lower temperature. In this condition, the maximum response is dominated by peak influence function. The same findings were recorded when there were variations of volume, void, and binder. Furthermore, for a higher vehicle speed, the considerable difference of tyre force with the nearest point due to instantaneous up/down phenomenon had ability to increase the percentage of differences.

Table 2 displays the results of the maximum percentage of differences along the 100 m of different road level when the asphaltic concrete (AC) temperature, vehicle speed, AC thickness, AC void content and AC binder content are considered variables. Only one road with *IRI*=2 takes into account the difference in results among them. As expected, for all variables, the percentages of differences increase with higher road roughness. The highest value is recorded as 19.8. Amongst the different variables, the highest is AC bitumen with 7%. The volume of bitumen in the mixture and its rheological behavior has a major bearing on the susceptibility of the asphalt to cracking. Bitumen at high temperature plays a role in helping maintain the service function of the flexible pavement from cracking; namely, healing. In asphalt-concrete mixture, bitumen is responsible for the visco-elastic properties whilst the mineral skeleton influences elastic and plastic properties.

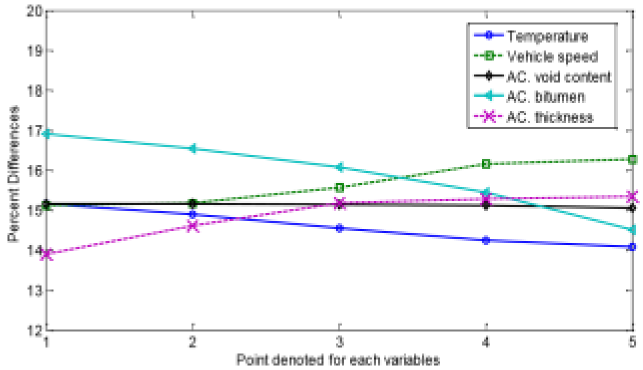


Fig. 7 - Maximum difference percentages of the number of allowable rutting calculated using IFM and PIFM as the temperature, volume of void, volume of binder, AC thickness and vehicle speed act as variables

Table 2 - Percent differences of stiffness modulus for varies parameter and road level using IFM and PIFM

Variables/Road	IRI=2	IRI=2.5	IRI=3	IRI=3.5
Temperature (‘C) 17				
	15.16	16.91	18.34	19.42
22	14.88	16.58	18.01	19.06
27	14.54	16.23	17.65	18.61
32	14.24	15.92	17.3	18.29
37	14.08	15.62	16.92	17.93
Veh. speed (m/s)				
10.6	15.12	17.55	19.18	18.98
15.7	15.17	17.91	19.24	19.01
20.6	15.56	16.86	18.34	19.01
25.6	16.15	17.56	18.48	19.04
30.6	16.28	17.66	18.52	19.06
AC. air void (%)				
4	15.16	16.85	18.34	19.40
6	15.15	16.83	18.32	19.37
8	15.13	16.83	18.31	19.35
10	15.12	16.8	18.27	19.34
12	15.06	16.78	18.27	19.31
AC. Bit. (%)				
7	16.91	18.24	19.26	19.80
8	16.55	17.96	19.08	19.74
9	16.08	17.60	18.86	19.64
10	15.44	16.29	18.50	19.48
11	14.50	16.28	17.92	19.18
AC. Thickness				
0.15	13.89	15.66	16.98	18.62
0.2	14.61	16.50	17.90	19.24
0.25	15.17	16.86	18.34	19.40
0.3	15.27	16.92	18.42	19.43
0.35	15.33	16.96	18.46	19.44

Therefore, elastic modulus of the asphaltic layer differs due to some variables examined in accordance to the relationship put forth by Brown and Brunton [17]. Initially, the dependency of the elastic modulus of asphaltic layer on the frequency of the applied load and temperature variation was examined. There are five different air temperatures, 27ºC (yearly average of temperature measurement), ±5ºC and ±10ºC of a yearly average is chosen. To differentiate the vehicle speed, five different vehicle speeds representing the real measures of heavy goods vehicles speeds on highway roads are determined. The 24-hour data was grouped into five bandwidths and the median of each bandwidth was determined for the analysis.

Fig. 8 shows the increase and decrease of stiffness modulus when the variables are changed. The differences were then determined by comparing the elastic modulus already calculated for the 2nd, 3rd, 4thand 5th value whereas elastic modulus was calculated for the 1st value. It can be seen that the curves are divisible into two separate regions positive and negative. It should also be noted that the negative difference for temperature, volume of binder and volume of void indicates a decrease in strain with increased parameter values and the difference for vehicle speed is positive that bears indication that an increase in strain is parallel to an increase of speed. A linear relationship is shown simply to illustrate the trend of the data. It is clear that over the range of variables, temperature is the one factor that influences the asphaltic material modulus more than others. This is followed by vehicle speed, volume of void, volume of binder and asphalt concrete thickness that ultimately, has minimal or almost no effect on strains.

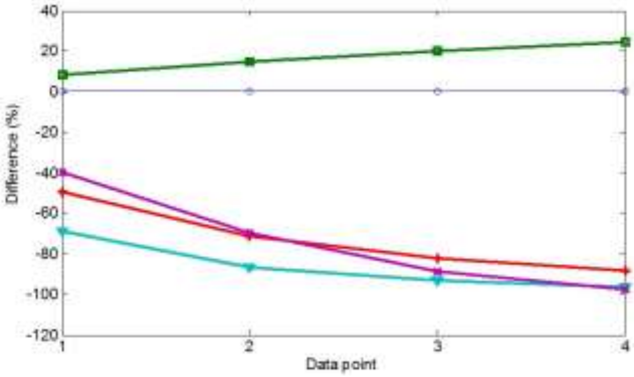


Fig. 8 - Percentage comparison of asphaltic concrete modulus, E_1 calculated using variation parameters. Data points denoted for the parameters start from the second value. Example: for parameter volumes of void, data point 1, 2, 3 and 4 are 6%, 8%, 10% and 12%, respectively

Theoretically, as viscous as liquid is in normal service temperature, bitumen has some specific features such as; its ability to still flow even at high temperature [24]. Bitumen is visco-elastic material and its behavior and stiffness varies from purely viscous to wholly elastic depending on the temperature and loading time at reference points that varies with truck speed. Consequently, the increase in temperature reduces the elastic modulus significantly due to the elastic modulus of the mixture dependency on the stiffness of the bitumen. In similarity, variations of bitumen and void content give the same patterns increasing parameter values found to reduce their elastic modulus. In contrast, the increment of elastic modulus as the vehicle speeds increase caused asphalt materials to be sensitive to vehicle speed and the frequency content of the applied loads. Higher speeds meant shorter durations of an applied axle, thus lowered effects on the material. Apart from that, there is little impact on the elastic modulus when the asphaltic layer thickness is varied approximately 0.0136%, 0.0273%, 0.0409% and 0.0545% for data point 1 to 4 respectively.

4. Summary

The following conclusions can be drawn from the prediction of pavement life using influence function and peak influence function for mechanistic-empirical pavement design analysis:

- The elastic modulus of the asphaltic material varies with asphaltic concrete thickness, vehicle speed, volume of void, volume of bitumen and temperature. Temperature was found to influence the asphaltic material modulus more than others due to the elastic modulus of the mixture's dependency on the stiffness of the bitumen. It is succeeded by vehicle speed, volume of void, volume of binder and AC thickness of which is minimal (or have almost no effect) on strains.
- Stress produced by vehicle loads spread throughout pavements at different weights. The maximum influence function is vertically under the centre of load.
- The compression influence function on top of the subgrade layer is approximately 3.5 times higher than the tension influence function at the bottom of asphaltic layer.
- Increase in the E_1/E_2 was seen to significantly increase critical stresses and strains for fatigue and rutting failure tremendously.
- Permanent deformation (rutting) predicted via primary response defined by the IFM and PIFM resulted in low differentiation (lower than 20%). Although, the value could increase or decrease as a correspondence to the pavement material and material properties.
- The differences of the tensile strain produced at the bottom of an asphaltic layer are less than that of the difference in compression strain produced on top of sub-grade. This is due to the fact that the influence function curve of tensile strain is narrower than the influence function curve of compression strain. Thus, the possibility of influence from the load at the nearest point to dominate maximum influence at the subject point is lesser than the influence at the top of the sub-grade layer.
- The road surface roughness seems to significantly increase the comparison between the fatigue life and rutting value using IFM and PIFM.
- The peak influence function is able to predict the response in pavement as a result of vehicle loads sufficiently and realistically. The low differences of strain in turn reduce the differences resulting in failure.

Acknowledgement

The authors would like to acknowledge the Universiti Tun Hussein Onn Malaysia that has funded this research through the Research University grant scheme TIER 1 Grant Vot H265 that enables this paper to be written.

References

[1] Abiola, O. S., Kupolati, W. K., Sadiku, E. R., & Ndambuki, J. M. (2014). Utilisation of natural fibre as modifier in bituminous mixes: A review. Construction and Building Materials, 54, 305–312.
[2] Park, D. Y., Buch, N. and Suh, Y. C. (2001). Development of fatigue cracking prediction model for flexible pavements, J. Of Civil Engineering, 5, 4, 397-402.
[3] Read, J. M., (1996). Fatigue cracking of bituminous paving mixtures, PhD Thesis, (Nottingham: University of Nottingham).
[4] Hardy, M. S. A and Cebon, D., (1992). Dynamic flexible pavement response models, Proc. 3rd International Symposium on Heavy Vehicle Weights and Dimensions, Vol.3, University of Cambridge, UK, 102-108.
[5] Cebon D., (1999). Handbook of vehicle –road interaction, (England: Taylor and Francis Group).
[6] Sun, L. and Hudson, W. R., (2005). Probabilistic approaches for pavement fatigue cracking Prediction Based on Cumulative Damage Using Miner's Law, J. Of Eng. Mechanics, 546-549.
[7] Buhari R., Abdullah M.E, Rohani M, (2014). Predicting Truck Load Variation Using Q-Truck Model. Applied Mechanics and Materials.Trans Tech Publication, Switzerland. 534, 105-110.
[8] Buhari R., Collop A.C., (2013). Pavement Primary Response using Influence Function and Peak Influence Function, Applied Mechanics and Materials, 256-259, 1871-1881.
[9] Buhari, R., Rohani M. M, Primary response of pavement variation with construction variables, Journal of engineering and applied science. ARPN Journal of Engineering and Applied Sciences, Vol.11, No.20, 1181511818.
[10] Van Der Poel, C., (1954). A general system describing the visco-elastic properties of bitumen and its relation to routine test data, J. of applied chemistry, 4.
[11] Ullidtz, P., (1998). Modelling flexible pavement response and performance, (Lyngby, Denmark: Polyteknisk Forlag).
[12] Ullidtz, P. and Busch C, 'Laboratory testing of full-scale pavement: the Danish road-testing machine.' Transp. Res. Rec. TRB, 715 PP 52-62, 1979.
[13] Eisenmann, J. (1975). Dynamic wheel load fluctuation-road stress. Strasse and Autobahn, 4, 127-128.
[14] Peutz, M.G.F., Van Kempen, H.P.M., and Jones, A., (1968). Layered system under normal surface loads, Highway Research Record No. 228, (Washington D.C: Highway Research Board, National Research Council), 34-45.
[15] Huang, Y. H., (1967). Stresses and displacements in viscoelastic layered systems under circular loaded areas. Proc. 2nd Int. Conf. on the Structural Design of Asphalt Pavements, pp. 225-244.
[16] Huang, Y.H., (1993). Pavement analysis and Design, (New Jersey, USA: Prentice Hall Inc).
[17] Brown, S. F. and Brunton, J. M., (1992). An introduction to the analytical design of bituminous pavements, 3rd ed., (Nottingham UK: Univ. of Nottingham)