International Journal of GEOMATE, Dec., 2019 Vol.17, Issue 64, pp. 9 - 17
ISSN: 2186-2982 (P), 2186-2990 (O), Japan, DOI: https://doi.org/10.21660/2019.64.08570
Geotechnique, Construction Materials and Environment

FLEXIBLE PAVEMENT DESIGN USING MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE IN THE PHILIPPINES

Lestelle Torio-Kaimo[1], Juan Michael Sargado[1], Daniel Peckley Jr.[1]

[1]Geotechnical Engineering Group, Institute of Civil Engineering, University of the Philippines

*Corresponding Author, Received: 03 April 2019, Revised: 04 June 2019, Accepted: 01 July. 2019

ABSTRACT: This paper presents a method of flexible pavement design and performance using a mechanistic-empirical pavement design guide (ME-PDG) as a tool for pavement design in the Philippines. The study was made using three types of traffic condition for both flexible pavements of the treated and untreated subbase. The initial design was performed using the local design guide which is based from the AASHTO 1993 design guide. The results were then evaluated using ME-PDG for its performance. The designs are then adjusted to meet with the results given by ME-PDG. All the simulation and computations in this study were performed using a program, based on the guide, created in MATLAB software. Results showed that top-down cracking failure is the most critical and greatly affects the smoothness of the pavement. It was also observed that the use of treated sub base is very effective in distributing the loads applied to the pavement and helps decrease the damage experienced by the pavement. The study concluded that ME-PDG yields more realistic and less conservative results compared to the AASHTO 1993 design guide. However, further studies should be performed to gather enough data to produce accurate and correct results.

Keywords: Mechanistic-empirical, Pavement design, Subbase

1. INTRODUCTION

Road pavement design and engineering have always been a challenge and a struggle to governments, scientists and engineers. The objective of coming up with a good road structure that can attest considerable time and damage has always raised interest anywhere in the world. In particular, flexible pavement design is initially based on experience in determining the thickness of pavement. Empirical approach played an important role until the advent of pavement design. The methods developed started from empirical methods (1929), with the consideration of soil strength test, to limiting shear failure method (1943) and regression methods (1961), which is based on pavement performance and road tests. Finally, in 1977, empirical-mechanistic methods were developed based on mechanics of materials in relation to pavement response [4]. The development of this method is still evolving up to the present aiming for better pavement design.

In 1996, in the effort of further developing the 1993 guide, a new method was recommended based on mechanistic principles. This came to the advent of the Mechanistic-Empirical Pavement Design Guide (ME-PDG) developed in the National Cooperative Highway Research Program (NHCRP 1-37A)[4]. Upon its initial release in April 2004, several reviews and studies were conducted to evaluate its performance. These resulted in ME-PDG to be the new pavement design guide for AASHTO.

In the Philippines, the Department of Public Works and Highways (DPWH) is a division in the government that is tasked to construct, maintain and develop the road network of the country. The DPWH follows the AASHTO 1993 guidelines. It is an empirical method based on the AASHTO road test performed in the late 1950s. Several adjustments and modifications were made to be able to account for uncertainties and limitations of the methods over the changing years.1

The purpose of this study is to compare flexible pavement designs and performance using empirical AASHTO 1993 pavement design guide and the mechanistic-empirical pavement design guide hereafter termed the ME-PDG. The study is limited to the principles and guidelines used in the ME-PDG Design Guide. This study assumes the applicability of the equations used in the guide to Philippine settings and that these equations are not site-specific.

2. MECHANISTIC-EMPIRICAL DESIGN GUIDE

The mechanistic-empirical approach was developed to aid in the limitations of the AASHTO 1993 design guide. This is for the reason that the results are given by AASHTO 1993 design guide were shown to have an inferior performance for places with warm temperature i.e. warmer than the AASHTO Road Test, as predicted by ME-PDG compared to those in colder regions, and AASHTO

1993 design guide performance prediction deteriorates as traffic levels increases [8].

The main objective of ME-PDG is to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures bases on mechanistic-empirical approach [4]. It provides a different design implementation which allows the designer to be fully involved in the process and have the flexibility to consider different design features and materials for a given site condition. This allows further optimization of the design and more controlled conditions and results.

The ME-PDG does not use ESALs to define traffic conditions, instead, it is given by vehicle class and load distributions in terms of traffic load spectra. The study suggests that design be initially performed using AASHTO 1993 Design Guide and then the results are evaluated using the ME-PDG for its performance.

2.1 Asphalt Concrete Pavement

Flexible pavements are multilayer system under loads generally composed of a subgrade, a drainage layer or a subbase, base course and surfacing course [7]. The traffic load stresses are spread out and are distributed to the roadbed soil. The asphalt concrete is the uppermost layer and designed to provide a skid-resistant surface. The base course distributes the traffic loading to the subbase course. The subbase course is a course of specified material and design thickness supporting the base course and distributes the traffic loads to the subgrade. Unbound aggregates or chemically treated course may be used for the base or subbase layer.

Asphalt pavements are often considered over other types due to its smoothness which provides a good rideability experience to the motorists. Smoothness affects ride quality, a major concern of the public in pavement conditions, overall durability, and performance. Asphalt binder type is another property that needs to be addressed in constructing asphalt pavement. Asphalt binders are characterized by their physical properties, which are directly related to field performance. Apart from asphalt binder viscosity grading, it is also tested against rutting, fatigue cracking and thermal cracking.

In the Philippines, flexible pavements usually consist of sub-base course, base course and surface course. Material properties and specifications for each course are specified in the DPWH Design Guidelines, Criteria and Standards. The major parameters required for the structural design include serviceability index, traffic parameters in terms of equivalent standard axle loads and structural number and layer coefficient for strength calculation. For the detailed design, TRL Overseas Road Note 31 – A Guide to the Structural Design of Bitumen Surfaced Roads in Tropical and Sub-Tropical Countries is another reference aside from AASHTO 1993 Guide [11].

3. METHODOLOGY

The M-E PDG is an iterative process in which predicted the performance of selected pavement structure is compared against the design criteria. The parameters are then adjusted until the user is satisfied with the design. The design process for asphalt pavements include the following:

1. Define site-specific trial design with subgrade support, material properties, traffic loading and environmental conditions;
2. Establish design criteria for acceptable pavement performance at the end of the design period (i.e. fatigue cracking, rutting, thermal cracking, and IRI);
3. Select the level of reliability for every applicable performance indicators;
4. Data processing for seasonal variations of materials, traffic and climatic inputs required in the evaluation;
5. Compute structural responses using multilayer elastic theory or finite element based pavement response models for each load and time step throughout the design period;
6. Calculate predicted distresses and/or damage at the end of each analysis period;
7. Evaluate the predicted performance of the trial design against the specified reliability level. Check if it satisfies the initial performance criteria. If doesn't, modify the design and repeat the process until the design is acceptable.

4. DESIGN PARAMETERS

4.1 Design Inputs

ME-PDG is capable of considering a wide range of structural sections as trial designs. These designs are analyzed and modified until it satisfies all performance criteria over the analysis period. Figure 1 shows some of the design that can be used in the analysis.

Apart from the trial design structure, other parameters must be provided such as subgrade properties, traffic, climatic data and other inputs related to construction. Since MEPDG is a parameter intensive procedure, it utilizes design input levels such that the designer can base the analysis on the level of data quality available. Level 1 data are site and/or material-specific inputs for the project that are obtained through direct testing or measurements. Level 2 data uses correlations to determine the required inputs. Level 3 data include national or regional default values to define the inputs. The input levels can vary from one parameter to the other. The input level selection depends on several factors such as the sensitivity of the pavement performance to a given input, criticality of the project, available information and available time and resources [4].

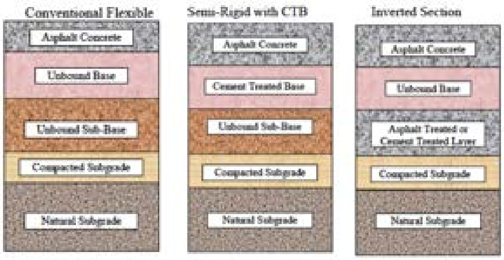

Fig.1 Possible asphalt layered systems [7]

The assumed layer thicknesses for each loading conditions, for both unbounded subbase (w/o CTSB) and cement-treated subbase (CTSB), are summarized in Table 1 to Table 3. Each thickness is predetermined using AASHTO 1993 design guide and it is designed in a way that it can withstand loading and damage up to its design life, at the same time, practical and economical. Additionally, traffic was assumed to increase at 4% annually. It is further assumed that the number of vehicles passing the road is constant for all months of the year. Three types of traffic conditions were considered. They are rural highways traffic having 20 million ESAL, urban roads traffic with assumed ESAL of 50 million, and heavily-trafficked highways with 130 million ESAL [11].

Table 1 ACP layer thickness used in the analysis

Traffic Condition	w/o CTSB	w/ CTSB
Heavily Trafficked Roads	25- 250 mm	25-250 mm
Urban Roads	25- 250 mm	25- 250 mm
Rural Roads	25-250 mm	25-250 mm

Table 2 Base layer thickness used in the analysis

Traffic Condition	w/o CTSB	w/ CTSB
Heavily Trafficked Roads	200 mm	200 mm
Urban Roads	200 mm	200 mm
Rural Roads	200 mm	200 mm

Analysis inputs such as traffic, material and climatic inputs like temperatures within the material structure and average moduli values of the layers for each analysis period should also be defined.

Table 3 Subbase layer thickness

Traffic Condition	w/o CTSB	w/ CTSB
Heavily Trafficked Roads	350 mm	250 mm
Urban Roads	350 mm	225 mm
Rural Roads	350 mm	225 mm

Table 4 and 5 present the material properties used in the analysis. This include regression intercept (A), regression slope of viscosity-temperature susceptibility (VTS), air void content (V_a), effective binder content (V_{beff}), cumulative % retained on the n-sieve (ρ_n), dynamic modulus (E), and viscosity (η). Then, analysis parameters for acceptable performance design criteria such as initial IRI, allowable values for fatigue cracking, thermal cracking, rutting and smoothness.

Table 4 Material Prop. for Pavement w/o CTSB

Property	Asphalt Concrete[1]	Base	Subbase	Subgrade
A	10.6508	-	-	-
VTS	-3.5537	-	-	-
V_a	3.8	-	-	-
V_{beff}	5.71	-	-	-
$\rho_{3/4}$	1.8	-	-	-
$\rho_{3/8}$	26.5	-	-	-
ρ_4	44.5	-	-	-
ρ_{200}	6.2	-	-	-
E (psi)	-	20000	15000	10000
η	0.35	0.3	0.4	0.4

Table 5 Material Prop. for Pavement w/ CTSB

Property	Asphalt Concrete[1]	Base	Subbase[2]	Subgrade
A	10.6508	-	-	-
VTS	-3.5537	-	-	-
V_a	3.8	-	-	-
V_{beff}	5.71	-	-	-
$\rho_{3/4}$	1.8	-	-	-
$\rho_{3/8}$	26.5	-	-	-
ρ_4	44.5	-	-	-
ρ_{200}	6.2	-	-	-
E (psi)	-	20000	500000	10000
υ	0.35	0.3	0.2	0.4

[1]Based on DFID Project on Stabilized Sub-bases for Heavily Trafficked Roads
[2]Based on Concrete Airport Pavement Workshop

on Cement-Stabilized Base Courses

Mean monthly temperature and the monthly relative humidity (RH) data (Table 6) were obtained from the World Meteorological Organization (WMO) World Weather Information database for the Philippines as reported by PAGASA.

Table 6 Average Monthly Temperature in the Philippines

Month	Ave. Min. Temp. (°C)	Ave. Max. Temp. (°C)
January	23.50	29.50
February	23.78	30.50
March	24.89	32.11
April	26.22	33.50
May	26.72	33.22
June	26.22	32.22
July	25.78	31.11
August	25.50	30.61
September	25.50	30.89
October	25.50	30.89
November	24.89	30.72
December	23.89	29.72

Truck loading was assumed to be a 3-axle truck. According to DPWH standards, the 3-axle truck has the highest axle load when distributed. In a 3-axle truck, the load distribution for each axle is as shown in figure 2.

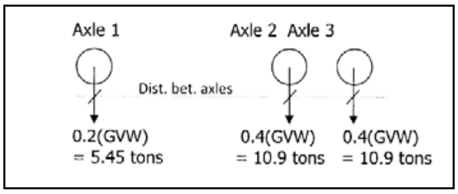

Fig.2 Load distribution for each 3-axle [11]

The corresponding ESAL of a single 3-axle truck pass is 6.44. It is calculated using the equation from DPWH DO-22[11].

Table 7 Truck Specifications [11]

Truck Specifications	Values
Number of Wheels	10
Total Number of Axle	3
Standard Axle Load	8200 kg
Gross Vehicle Weight	27250 kg
Tire Contact Radius	4 in

4.2 Pavement Response Model

Pavement response models are used to determine the structural response of the system caused by traffic loads and environmental influences. The analysis yields stresses, strains and displacement in the pavement layers which is required in the design procedure. Depending on the material and loading of the pavement, multilayer elastic theory and/or finite element method can be used. Both methods considered in the Design Guide.

Inputs to the flexible pavement response models include pavement geometry (i.e. layer thickness), environment (i.e. temperature vs. depth for each season, moisture vs. depth for each season), material properties including elastic and non-linear properties (adjusted for environmental and other effects, as necessary), and traffic, which includes load spectrum and tire contact pressure and distribution.

4.3 Analysis Location

Upon the analysis of the structural response of the system, each pavement response variable must be evaluated at the critical location where it is at its extreme value. It is important to define the locations in the pavement where maximum damage will occur over the design period. The design guide defines the analysis locations where the maximum damage is to be expected. Analysis location shown in figure 3 can be used both for multilayer elastic analysis and finite element method.

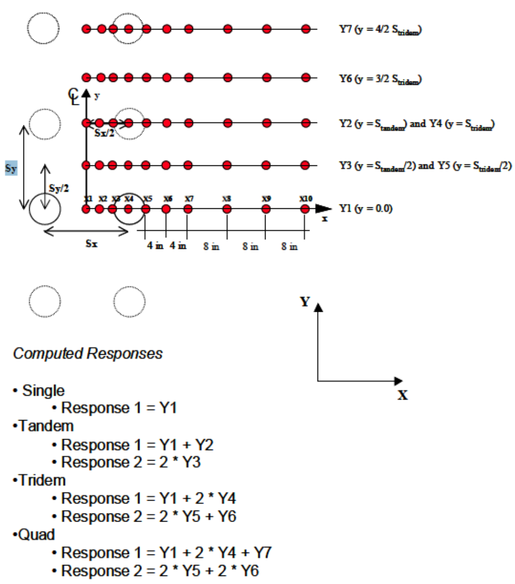

Fig.3 Schematic for horizontal analysis locations

These guides are locations only in the x-y view plane. Critical responses are determined at several depth locations beneath each of the locations above, depending on the distress type. Apart from these, fatigue cracking depth locations and rutting depth locations should also be evaluated. Fatigue cracking depth locations include (1) surface of the pavement (z = 0); (2) 0.5 inches from the surface (z = 0.5); (3) bottom of each bound or stabilized layer. Also, rutting depth locations would involve (1) mid-depth of each structural layer/sub-layer; (2) top of the subgrade; (3) six inches below the top of the subgrade.

4.4 Asphalt Concrete Pavement Distresses and Critical Responses

The design guide is based upon the accumulation of damage as a function of time and traffic. The primary distress considered in the design is rutting, fatigue cracking and thermal cracking. Pavement smoothness (IRI) will also be predicted based on the given factors.

Rutting is a surface deformation caused by inelastic or plastic deformations in any or all of the pavement layers and subgrade. There are three distinct stages for the permanent deformation behavior of pavement materials. These three stages include the primary stage, where there is high initial level of rutting, with a decreasing rate of plastic deformations, predominantly associated with volumetric change, the secondary stage, where small rate of rutting exhibiting a constant rate of change of rutting that is also associated with volumetric changes, and the tertiary stage, wherein there is high level of rutting predominantly associated with plastic(shear) deformations under no volume change conditions.

The design guide only takes into account the primary (Level 1) and secondary stages (Level 2). The tertiary stage (Level 3) is not accounted for in the design method. At the same time, the design guide does not take into account permanent deformation for chemically stabilized materials, bedrock, and PCC fractured slab materials to the total permanent deformation of the pavement system. Rutting is estimated for each sub-season at the mid-depth of each sublayer. The overall permanent deformation is the sum of permanent deformation for each individual layer. Rutting prediction in the asphalt mixture is based upon a field calibrated statistical analysis of laboratory repeated load permanent deformation tests.

Fatigue cracking is determined in terms of damage index, which represents the load-associated damage within the pavement structure. Fatigue cracking is determined by predicting damage using Miner's Law. Fatigue cracking is classified into two types: bottom-up or alligator cracking and surface-down or longitudinal cracking [4].

Bottom-up cracking is cracking that originate at the bottom of the asphalt layer and propagate to the surface due to repeated loading. Some factors that may cause high tensile strain and stress at the bottom of the asphalt layer include thin or weak asphalt layer, higher loads and tire pressure and weak aggregate base/subbase layers.

Surface-down fatigue cracking or longitudinal cracking is load-related cracks that start from the surface and propagate downward. Although there are no conclusive data to prove the cause of this type of distress, some of the suggested reasons include wheel load tensile stress and strain occurring at the surface combined with aging of the asphalt layer, shearing of asphalt layer from radial tires with high contact pressure and severe aging of asphalt layer combined with high contact pressures. Figure 4 and 5 shows the mechanism of bottom-up and surface-down cracking.

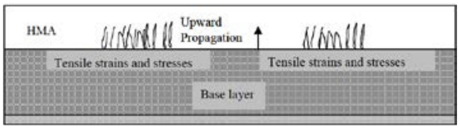

Fig. 4 Bottom-up Fatigue Cracking

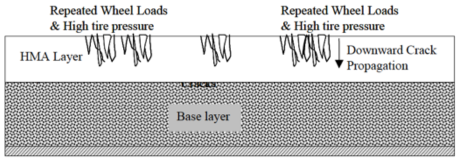

Fig. 5 Surface-down Fatigue Cracking

Thermal cracks are due to extreme temperature changes in the pavement layers and are most common to regions with extreme cold weather. There are two types of non-load related thermal cracks: transverse cracking, which occurs first, and block cracking, which occurs after transverse cracking and is due to the aging of the asphalt that makes it brittle over time. For this type of cracking, crack propagation is computed using Paris' law.

The IRI of the pavement at the end of the design period depends upon the initial IRI from the initial as-constructed condition of the pavement and the subsequent developed distresses over the design time. The distresses include rutting, fatigue cracking and thermal cracking. Prediction of roughness over time, taken from the model, involves initial IRI, predicted distresses, and site factors such as subgrade and climatic factors. IRI is estimated incrementally over the entire design period on a monthly basis.

4.5 Numerical Modelling

MATLAB software was used to simulate the process of ME-PDG including the pavement structural response model, FEM. Linear elastic FEM (2D axisymmetric formulation) is used to calculate the stresses arising from the distributed load due to one wheel. The total stresses due to the simultaneous action of four wheels corresponding to one side of the tandem axle are then calculated via superposition of stresses taken at appropriate locations in the FEM solution. The total damage due to one vehicle is calculated as the sum of damage arising from one pass of the front axle and the damage arising from one pass of the tandem axle. Thermal cracks were not included in the final calculations since the damage contributed by this distress is very negligible compared to rutting and fatigue cracking. This is due to the fact that the Philippines do not experience extreme temperature change because of its tropic climate.

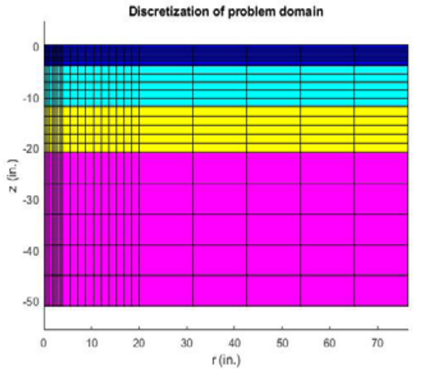

Fig. 6 Sample pavement model with discretization

5. RESULTS AND DISCUSSION

5.1 Heavily Trafficked Roads

Analyses are performed for a design period of 10 years for this study. The result for bottom-up cracking (Figure 7) for heavily trafficked roads with untreated subbase shows that the most critical location is at coordinates (2,0) and (6,0). The maximum value of bottom-up cracking that it experienced is 18.94% with a thickness of 1 inch. The least critical location is at coordinate (42,0) with the highest value of bottom-up cracking experienced for asphalt layers with a thickness between 5 to 6 inches. For cement-treated subbase, asphalt layers of 5 inches and up experienced almost the same magnitude of distress for the least critical and most critical location, unlike the untreated pavement. This shows that the treated subbase increases the ability of the pavement to distribute the load evenly to the pavement layers, which help lessen the incurred damage.

Top-down cracking (Figure 8) shows varying values for the asphalt layer thickness of 6 inches and below and converges for asphalt thickness of between 6 to 10 inches. The most critical location is at (2,0) and the least still critical is at (6,0). The maximum value of top-down cracking experienced is at 1-inch thickness of asphalt layer with a value 10560 m/km. Similar trend was observed for the top-down cracking of the treated and untreated subbase. However, the damage for the treated base shows lower values compared to that of untreated base, which makes cement treated base effective in decreasing the stresses in the pavement.

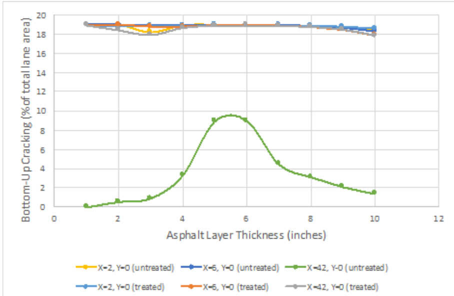

Fig.7 Result for bottom-up cracking at the most critical and least critical locations for heavily trafficked roads

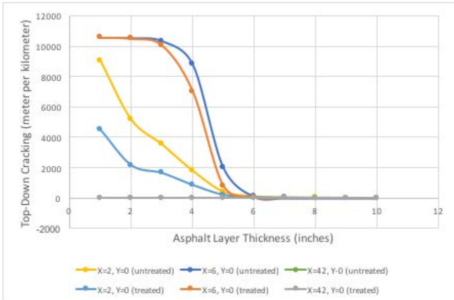

Fig.8 Result for top-down cracking at the most critical and least critical locations for heavily trafficked roads

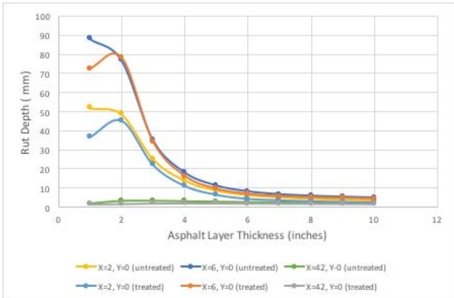

Fig.9 Result for rut depth at the most critical and least critical locations for heavily trafficked roads

Rutting (Figure 9) is maximum at location (6,0) with the maximum rut depth experience of 88.24 mm. Values of rutting for the section converges towards the least critical values as the asphalt layer becomes thicker. Rutting in treated subbase displayed significantly lower values as compared to the untreated base. A slight increase on rut depth is evident at 2-inch thickness.

The resulting IRI has the most critical location at coordinate (6,0) with maximum IRI at a value of 16.29 for 1-inch depth (Figure 10). An acceptable value of IRI can be considered for thickness between 4 to 6 inches. For layer thickness greater than 6 inches, the value of the IRI of the least and most critical locations are almost equivalent. Calculated rut depth is not taken into account for the calculation of final IRI due to the initial assumption that rutting is assumed to be equal to 20%. This means that the least value of IRI for the untreated base is 4. Smoothness values of the pavement decreased for each of the asphalt thickness for the treated subbase. This should be expected since most of the values that evaluate the final IRI of the pavement also decreased as shown from the results. IRI of asphalt layer between 4 inches and 6 inches can be seen to decrease abruptly from 8.94 to 0.62.

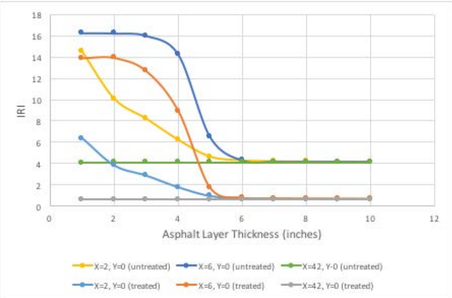

Fig.10 Result for IRI at the most critical and least critical locations for heavily trafficked roads

5.2 Urban Roads

The analysis for urban roads yields similar behavior as that of the results for heavily trafficked roads. For the bottom-up cracking, the least critical is still at coordinate (42,0) and the most critical are still equally experienced at coordinates (2,0) and (6,0). A slight increase in distress was observed for thickness between 4 inches and 6 inches. This may be due to the stresses experienced by the pavement with these thicknesses. On the other hand, bottom-up cracking for the treated base has a maximum value of 18.94% both for locations (2,0) and (6,0) at the 1-inch thickness and the minimum value of 15.5703% for coordinate (42,0) at 10-inch thickness. Values for thickness 4 inches to 8 inches are almost the same for all coordinates making it a good design given that the whole section experience almost the same magnitude of distresses.

Top-down cracking is still maximum at coordinate (6,0) with the value of 10559 for 1-inch depth. The values of top-down cracking converge to zero for the thickness of 5 inches and above. The asphalt layer experiences no damage for top-down cracking if the asphalt layer is set equal to or greater than 6 inches. For the top-down cracking for the treated base, the damage is still highest at coordinate (6,0) with a magnitude of 10558 m/km. It is also evident in top-down cracking approaches to zero as the asphalt layer becomes thicker.

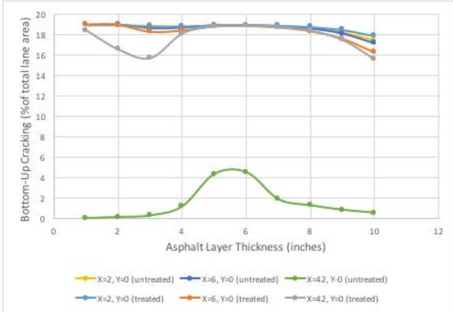

Fig.11 Result for bottom-up cracking at the most critical and least critical locations for urban roads

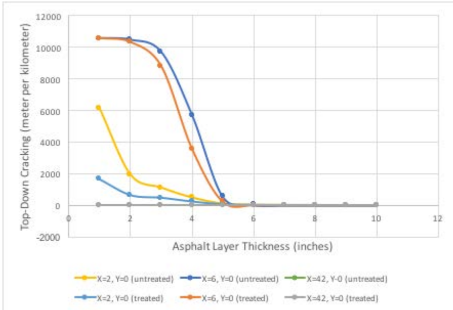

Fig.12 Result for top-down cracking at the most critical and least critical locations for urban roads

Rutting has a maximum value of 68.615 at the location (6,0) and a minimum of 1.8136 mm at (42,0), both at a 1-inch depth. Rut depth converges to a value of 2 mm, for all coordinates, as the asphalt layer becomes thicker. The same behavior is true for treated subbase as it is observed to converge to a value of rut depth equal to 1.

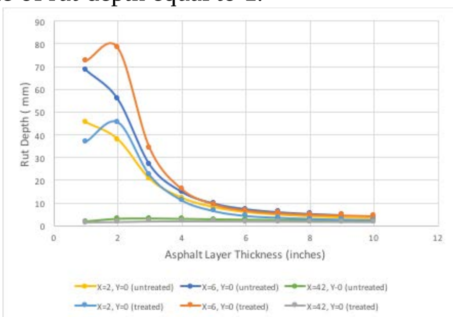

Fig.13 Result for rut depth at the most critical and least critical locations for urban roads

For IRI results for urban roads with untreated base, calculated rut depth is again not taken into account for the calculation of final IRI due to the initial assumption that rutting is assumed to be equal to 20%. The least value of IRI for the untreated base will still be 4. Maximum IRI is still experienced at 1-inch depth and the critical location IRI values converge to 4 as the asphalt layer becomes thicker.

Considering the behavior of distresses for urban roads an acceptable value of IRI can be achieved for the asphalt layer equal to 5 inches or more after the design period of 10 years.

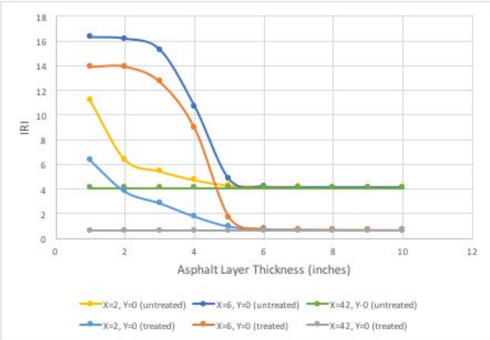

Fig.14 Result for IRI at the most critical and least critical locations for urban roads

5.3. Rural Roads

For rural roads with untreated subbase, similar behavior is observed for bottom-up cracking. As with urban and heavily trafficked roads, the values of the most critical and least critical locations vary greatly in magnitude, showing an uneven distribution of stresses to the pavement. The least critical location shows little to no sign of bottom-up cracking while the most critical location experienced a high magnitude of distress. Bottom-up cracking for the treated base has a maximum value of 18.94% both for locations (2,0) and (6,0) at the 1-inch thickness and the minimum value of 10.94% for coordinate (42,0) at 3-inch thickness. Values for thicknesses 4 inches to 8 inches are almost the same for all coordinates making it a good design given that the whole section experienced almost the same magnitude of distress.

Similar behavior was observed for top-down cracking and rutting for untreated subbase as with the results of urban and heavily-trafficked loads. The result for top-down cracking for the least and most critical location converges to a value of zero for thicker asphalt layers i.e. 5 inches and above. On the other hand, results of rut depth for the least and most critical locations converge to a value of 2 mm. For the treated subbase, damage due to top-down cracking is highest at coordinate (6,0) with a magnitude of 10553 m/km. It is observed that the result converges to zero as the asphalt layer becomes thicker. Rutting is still highest at 1-inch thickness with the maximum value equal to 42.68 mm and decreases as the asphalt layer becomes thicker.

The least value of IRI for the untreated base is again equal to 4. Maximum IRI is still experienced at 1-inch depth and the critical location IRI values converge to 4 as the asphalt layer becomes thicker.

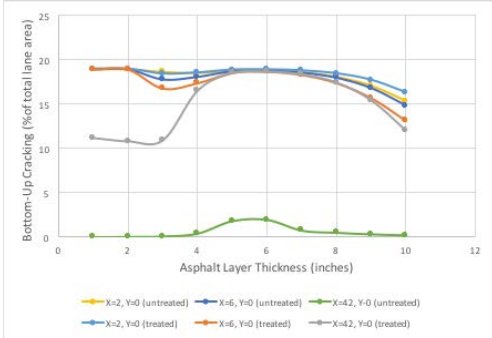

Fig.15 Result for bottom-up cracking at the most critical and least critical locations for rural roads

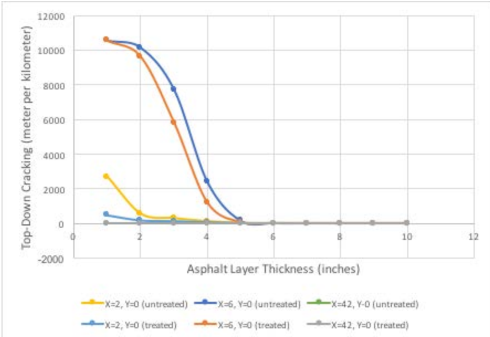

Fig.16 Result for top-down cracking at the most critical and least critical locations for rural roads

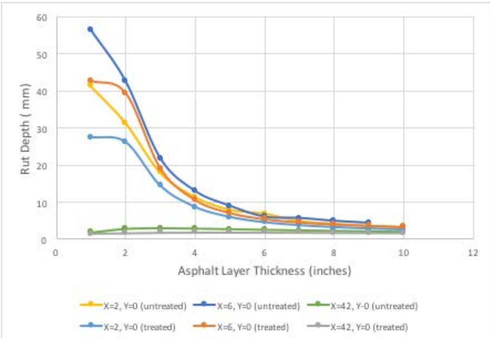

Fig.17 Result for rut depth at the most critical and least critical locations for rural roads

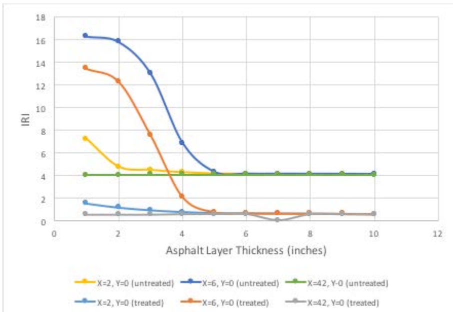

Fig.18 Result for IRI at the most critical and least critical locations for rural roads

Considering the behavior of distresses for rural roads, the value of IRI for treated subbase converges to 0.5 for all points in the pavement, giving a better design for the pavement. A decent value of IRI can be obtained for asphalt thickness of 4 inches and above.

6. CONCLUSIONS

The main objective of this study is to compare flexible pavement design and performance between AASHTO 1993 pavement design guide and the mechanistic-empirical pavement design guide as a tool for pavement design in the Philippines. This attempt of applying the mechanistic-empirical approach for pavement design in the Philippines resulted in several conclusions and recommendations that would help address future studies produce better results and more reliable design solutions.

The results of the analysis showed that top-down cracking failure is the most critical and greatly affects the smoothness of the pavement. For the untreated subbase, although the rut depth value was calculated in the analysis, the coefficient of variation for rutting depths was still assumed to be equal to 20% as recommended in the design guide. This does not give a realistic trend to the IRI and sets the minimum value of IRI for untreated subbase to 4. Thermal cracking was found to yield a very small value of failure compared to rutting and fatigue cracking. This show that ME-PDG for thermal cracking only applies to regions with extreme changes in temperature.

The use of treated subbase is very effective in distributing the loads applied to the pavement and helps decrease the damage experienced by the pavement. The results show that the magnitude of distresses for most points of the pavement are comparable.

Although the results show that ME-PDG yields more realistic and less conservative results than AASHTO 1993 guide, further study should be conducted to be able to gather more data that would produce more accurate results. The advantage of ME-PDG for being a site-specific, parameter-intensive method becomes a disadvantage in cases when the necessary data are not available for the user. This forces the designer to make assumptions that add to the inaccuracy of the results. Additional testing should be performed to be able to acquire the data needed in analyzing pavements using ME-PDG.

Analysis using ME-PDG gives better and quicker results if the AASHTO 1993 guide is used to predict the initial design for the analysis.

A thorough study should be made to further understand how ME-PDG was formulated for better application of the method to other cases. It is highly recommended to perform experiments verifying the relationships used in the method and its applicability to Philippine conditions.

7. ACKNOWLEDGMENT

The authors acknowledge the financial assistance of Holcim Philippines in this research project.

8. REFERENCES

[1] F. Y. Abdo, Portland Cement Association, [Online].Available: http://www.pavementse.com/airports/11-09%20Presentations/1%20Cement%20Treated%20Bases_Abdo.pdf.

[2] American Association od State Highways and Transportation Officials (AASHTO), "AASHTO Guide for Design of Pavement Structures," Washington, D.C., 1993.

[3] Department of Works and Highways, "Minimum Pavement Thickness and Width of National Roads," 2011.

[4] National Highway Institute, "Introduction to Mechanistic-Empirical Pavement Design of New and Rehabilitated Pavements," Washington DC, 2002.

[5] California Department of Transportation, "California Department of Transportation," [Online]. Available: http://www.dot.ca.gov/hq/oppd/hdm/pdf/chp0630.pdf.

[6] Florida Department Of Transportation, Flexible Pavement Design Manual, Florida Department Of Transportation Pavement Management Offic , 2008.

[7] J. P. Guyer, 2009. [Online]. Available: http://www.cedengineering.com/courseoutline.asp?cid=313.

[8] C. W. Schwartz and R. L. Carvalho, " Evaluation of Mechanistic-Empirical Design Procedure," *Implementation of the NCHRP 1-37A Design Guide* , vol. 2, February 2007

[9] D. E. Newcomb and D. H. TImm, "Mechanistic Pavement Design," *Hot MIx Asphalt Technology*, January 2002.

[10] O. Ali, "Evaluation of the Mechanistic Empirical Pavement Design Guide," 2005.

[11] Department of Public Works and Highways (DPWH), "DPWH Standard Specifications for Public Works and Highways, 2004 Edition: Volume II, Highways, Bridges and Airports," vol. 2, Manila, 2004, p. 438.