COMPARATIVE STUDY OF NON-DESTRUCTIVE FIELD TESTING DEVICES ON BSM-EMULSION

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ABSTRACT

In recent years, an increase in the number of non-destructive testing devices (NDT) commonly utilized in field characterization of pavement systems as well as in assessment of construction quality control of road pavement layers has been witnessed. These devices utilize either static or dynamic loading mechanisms. Most of these devices impart an impact or vibratory load to the road surface with the applied force and the induced pavement surface motion being simultaneously monitored. These devices are not only rapid, cheap and non-destructive, but also offer the opportunity to measure the pavement's in-situ response under a load closely simulating actual moving traffic loads. Different factors affect the measurements taken with these devices. On the other hand, the measurements of each non-destructive device differ from others in terms of magnitude and unit and in most cases there is no direct comparison between these different devices. This paper reports on the statistical correlation study done on measurements taken with the Clegg Impact Soil Tester (CIST) also known as "Clegg Hammer", the Rapid Compaction Control Device (RCCD), the Light weight deflectometer (LWD), the Falling Weight Deflectometer (FWD) and the Portable Seismic Pavement Analyzer (PSPA) during the monitoring of the curing process of the Soil Treated with Emulsion (commonly known as Bitumen Stabilised Material, BSM-emulsion) research sections in Mozambique. In the past, no correlations between these devices have been developed and only correlations with CBR are reported. The finding of this study was that there is a reasonably good correlation between the CIV values, which is the CIST measurement, and the RCCD penetration although the RCCD measures the in situ shear resistance characteristics while the CIV measures elastic properties of materials. Fair to good correlations were found between the LWD and FWD bowl parameters of similar nature. The correlations between the PSPA with other devices were in general poor. The conclusions drawn in this study are specific to the BSM-emulsion and no moisture control was carried out during the testing which limits wider use of the correlations developed.

1 INTRODUCTION

In recent years there has been an increase in the number of Non-Destructive Testing (NDT) tools utilized in field characterization of structural pavement systems. These non-destructive devices utilize either static or dynamic loading mechanisms and they are not only rapid, cheap and non-destructive, but also offer the opportunity to measure the pavement's in-situ response under a load closely simulating actual moving traffic loads (Rohde, 1994). Most of these devices impart an impact or vibratory load to the road

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surface with the applied force and the induced pavement surface motion being simultaneously monitored (Hoffmann *et al*., 2004).

Some of these devices were used during the monitoring of the curing process of Soil Treated with Emulsion (BSM-emulsion) research sections built in Marracuene area, north of Maputo in Mozambique. These were the Clegg Impact Soil Tester (CIST), Rapid Compaction Control Device (RCCD), Light Falling Weight Deflectometer (LWD), the conventional Falling Weight Deflectometer (FWD) and the Portable Seismic Pavement Analyzer (PSPA). These testing devices can be divided into three categories as follows:

a) Devices that measure the Shear Strength of the pavement layer,
- RCCD

b) Devices that measure the effective elastic modulus of the pavement layer,
- LWD
- FWD
- CIST

c) Devices that measures the Seismic modulus of the pavement layer,
- PSPA

Thus the various non-destructive in-situ testing devices measure different parameters. The difference is in terms of magnitude of the measurements and unit, and in most cases there is no direct comparison between these different devices.

This paper reports on the statistical correlation study done on measurements taken with these devices. To achieve this, an extensive series of tests on BSM-emulsion were carried out at regular intervals of 50 m, in the middle of each traffic lane. The testing was conducted as per schedule indicated in Table 1 but it was not always possible to have all of the equipment on site. Special attention is called to the fact that during the execution of these tests no moisture control was performed.

Table 1: Testing schedule of the BSM-emulsion sections

Device	Test setup	Frequency	Time after construction
CIST	4.5 kg hammer, five blows at each station	50 m	24 hrs, 2 days, 1 week, 2 weeks, 4 weeks, 3 months and 6 months
RCCD	Single blow at each station	50 m	24 hrs, 2 days, 1 week, 2 weeks, 4 weeks, 3 months and 6 months
LWD	10 kg and 200 mm plate diameter setup, three readings at each station	50 m	24 hrs, 2 days, 1 week, 2 weeks, 4 weeks, 3 months and 6 months
FWD	40 kN and 300 mm plate diameter setup, two readings at each station	50 m	Every six months
PSPA	Four readings at each station at 0º, 90º, 180º and 270º	50 m	Every three months

2 PREVIOUS CORRELATION STUDIES

Few correlation studies between the different devices indicated above are reported. Most of these devices were correlated with the CBR of the sand soil material used. This may be due to the fact that current criteria for evaluating and designing pavements rely on characterizing soils and unbound pavement structural layers with either California Bearing Ratio (CBR) values or effective elastic modulus.

2.1 Rapid Compaction Control Device

The research carried out on labour-based projects in the sub-Saharan region by Paige-Green (1998) pointed out that the RCCD penetration can be correlated with CBR through Equation (1) for material complying with specifications indicated in Table 2 below:

$$DN_{RCCD} = 0.0735 * \left(e^{-1.33 \frac{CMC}{OMC}} \right) \left(CBR^{0.46} \right)^{-0.775} \qquad (1)$$

Table 2: Material specifications for unpaved rural roads (CSRA, 1990; Paige-Green, 1998)

Maximum size (mm)	37.5
Oversize Index (I_o)	5 %
Shrinkage product (S_p)	100 – 365
Grading coefficient (G_c)	16 – 34
Soaked CBR (%)	15 at 95 % Modified AASHTO density
Treton Impact value (%)	20 – 65

The laboratory study conducted by De Beer *et al.* (1993) to define the relationship between the RCCD and the well known DCP/CBR relationship by Kleyn (1975) resulted in the Equation (2).

$$CBR = 804*(DN_{RCCD})^{-1.29} \qquad (2)$$

2.2 LWD and FWD

Fleming *et al.*, (2000) conducted field tests to correlate the moduli determined with the LWD with that of the FWD. Their results showed that the resilient surface modulus, $E_{(FWD)}$ correlated well with resilient surface moduli obtained from the LWD as illustrated by Equation (3). However they found that the correlation coefficients are LWD instrument specific and should first be established before being used with confidence. Fleming (2001) reported that a number of factors influence the measured moduli of the LWD including differences in mass, transducer type and software analysis (which records the maximum deflection as that at the time of the peak force).

$$E_{(FWD)} = 1.031*E_{(LWD)} \qquad (3)$$

Nazzal (2003) found that the best model to predict the FWD back-calculated resilient surface moduli, $E_{(FWD)}$ in MPa from the LWD surface modulus, $E_{(LWD)}$ in MPa is:

$$E_{(FWD)} = 0.97*E_{(LWD)} \quad \text{for } 12.5 \text{ MPa} < E_{(LWD)} < 865 \text{ MPa} \qquad (4)$$

With R^2 = 0.94, significance level < 99.9% and standard error = 33.1

In comparing his studies with those of Fleming (2000), Nazzal (2003) found that his correlations agreed with those of Fleming (2000) for a variety of material types.

According to Rahimzadeh *et al.*, (2004) the relationship between FWD and LWD was found to be material type and thickness dependent. The FWD is regarded as the most appropriate device for setting the standard, because not only is the loading most representative of real traffic loading, but it can also be used for assessment of all pavement layers as construction proceeds. Either the FWD or the LWD can be used for measurement of stiffness as long as the same plate rigidity factor is assumed ($\pi/2$ for a flexible plate). If the LWD default setting (rigid plate, rigidity factor of 2) is assumed, then a correction factor must be applied, such that:

$$E(\mathrm{LWD}) = 1.273 * E(\mathrm{FWD}) \tag{5}$$

Steinert *et al.* (2005) concluded that LWD surface moduli were comparable with surface moduli derived from the traditional FWD. Regression analyses comparing surface moduli from both devices yielded correlation coefficients ranging from 0.34 to 0.95. In general, LWD surface moduli were slightly less than FWD surface moduli. Correlation coefficients tended to increase with decreasing pavement thickness. The paved roads showed a strong correlation between LWD and FWD derived surface moduli with a regression coefficient of 0.81. The regression coefficient increased when the pavement thickness decreased. Regression analyses comparing surface and subbase moduli yielded correlation coefficients ranging from 0.16 to 0.81. Again, correlation coefficients tended to increase as pavement thickness decreased. The LWD had a reasonable correlation with FWD derived subbase moduli, suggesting that the LWD surface moduli are influenced in part by the subbase layer.

A pilot study with various combinations of FWD and LWD settings was carried out by Horak and Khumalo (2006). Although the sample size was small, correlation of the FWD surface moduli and LWD surface moduli showed that $E(\mathrm{FWD})$ with average contact pressure of 566 kPa and the $E(\mathrm{LWD})$, with average contact pressure of 535 kPa had the best correlation. The regression model was as follows:

$$E(\mathrm{FWD}) = 1.29 * E(\mathrm{LWD}) - 42.0 \quad \text{with } R^2 = 0.84 \tag{6}$$

The study also yielded the regression models between six of deflection bowl parameters defined in Table 3. The obtained regression models are summarised in Table 4 with a significance level of 95% for the deflection bowl parameters measured with LWD and FWD on granular base pavement road.

Table 3: Deflection Bowl Parameters (Horak et al, 1989)

Parameter	Formula	Structural indicator
1. Maximum deflection	D_0 or Ymax as measured	D_0 gives an indication of all structural layers with about 70% contribution by the subgrade
2. Radius of Curvature (RoC)	$RoC= (200)^2/[2D_0 (1- D_{200}/D_0)]$	RoC gives an indication of the structural condition of the surfacing and base condition
3. Base Layer Index (BLI)	$BLI=D_0-D_{300}$	BLI gives an indication of primarily the base layer structural condition
4. Middle Layer Index (MLI)	$MLI=D_{300}-D_{600}$	MLI gives an indication of the subbase and probably selected layer structural condition
5. Lower Layer Index (LLI)	$LLI=D_{600}-D_{900}$	LLI gives an indication of the lower structural layers like the selected and the subgrade layers
6. Spreadability, S	$S=\{[(D_0 +D_1 +D_2+D_3)/5]100\}/D_0$ Where D_1, D_2, D_3 spaced at 300mm	Supposed to reflect the structural response of the whole pavement structure, but with weak correlations
7. Area, A	$A=6[1+2(D_1/D_0) +2(D_2/D_0) + D_3/D_0]$	The same as above
8. Shape factors	$F1=(D_0-D_2)/D_1$ $F2=(D_1-D_3)/D_2$	The F2 shape factor seemed to give better correlations with subgrade moduli while F1 gave weak correlations
9. Slope of Deflection	$SD= \tan^{-1}(D_0-D_{600})/600$	Weak correlations observed

Table 4: Deflection Bowl Parameters regression summary (Horak and Khumalo, 2006)

Deflection bowl parameter	FWD settings	LWD settings	Correlation formula	Correlation Coefficient (R^2)
Ymax	40 kN	20 kg, 200 mm	Ymax (FWD) = -127.43 + 2.08 Ymax (LWD)	0.58
MLI	25 kN	10 kg, 200 mm	MLI (FWD) = 22.62 + 1.58 MLI (LWD)	0.97
BLI	25 kN	20 kg, 200 mm	BLI (FWD) = 103.1 + 0.3 BLI (LWD)	0.28
RoC	25 kN	20 kg, 200 mm	RoC(FWD) = 76.2 + 0.55 RoC (LWD)	0.78
SD	40 kN	20 kg, 200 mm	SD (FWD) = 0.82 + 1.3 SD (LWD)	0.37
F1	25 kN	20 kg, 200 mm	F1 (FWD) = 0.4 + 0.15 F1 (LWD)	0.96

2.3 Clegg Impact Soil Tester

There has been considerable interest in correlating the Clegg Impact Value, (CIV), determined with the Clegg Impact Soil Tester (CIST), with the CBR of the material. Clegg (1980) presented the first correlation, Equation (7), which was based on tests carried out in the laboratory. With availability of more data from different sources (Australia, New Zealand and United Kingdom) which covered a wider range of soils for both laboratory and in-situ conditions, the Equation (7) was confirmed but slightly adjusted yielding Equation (8) (Clegg, 1986).

$$CBR = 0.07*(CIV)^2 \quad (7)$$

$$CBR = 0.06*(CIV)^2 + 0.52*(CIV) + 1, \; R^2 = 0.92 \quad (8)$$

Mathur and Coghlan (1987) reported the following relationship:

$$CBR = 0.11*(CIV)^{1.86}, R^2 = 0.79 \quad (9)$$

Al-Amoudi *et al*. (2002) conducted a study on a typical eastern Saudi Arabian calcareous soil that consisted firstly of laboratory tests over a wide range of density, moisture content, and compactive effort and secondly focused on the performance of in situ CBR and CIST tests on various types of soils. The data developed from both the laboratory and field tests were combined to arrive at the best statistically reliable model which could predict the CBR values from CIST results. Equations (10) and (11) show the best fitting model yielded from the laboratory and field respectively.

$$CBR = 0.1977*(CIV)^{1.535} , R^2 = 0.81 \quad (10)$$

$$CBR = 1.349*(CIV)^{1.012}, R^2 = 0.85. \quad (11)$$

Al-Amoudi *et al*. (2002) went further by analyzing statistically all of the data developed in the laboratory and the field as well as the data reported by Clegg and Mathur and Coghlan (1987) simultaneously which resulted in the following general best-fit model:

$$CBR = 0.1691*(CIV)^{1.695}, R^2 = 0.85 \quad (12)$$

Both Clegg (1986) and Al-Amoudi *et al*. (2002) indicated that although Equations (8) and (12) were developed from many different types of soils, it is recommended to conduct a few trial CBR-CIST tests to verify the reliability of these models for any proposed soil to be used in construction.

2.4 Portable Seismic Pavement Analyzer

The Portable Seismic Pavement Analyzer (PSPA) is still a relatively new instrument in pavement engineering, therefore no correlation study between this device with other NDT has been reported. The study carried out by Nazarian *et al.*, (2002) focussed on the relationship between the seismic modulus and design modulus. The research on three dozen specimens with a large variation in stiffness and material type (from clayey subgrade to high quality base) resulted in the relationship illustrated in Figure 1. This relationship offers a several advantages, being first a mean of estimating the resilient modulus from the seismic modulus. In that manner, the seismic modulus can be readily converted to the design modulus. In addition, the need for extensive resilient modulus testing is substantially reduced. The quality control can be carried out much more rapidly as well (Nazarian *et al.*, 2002).

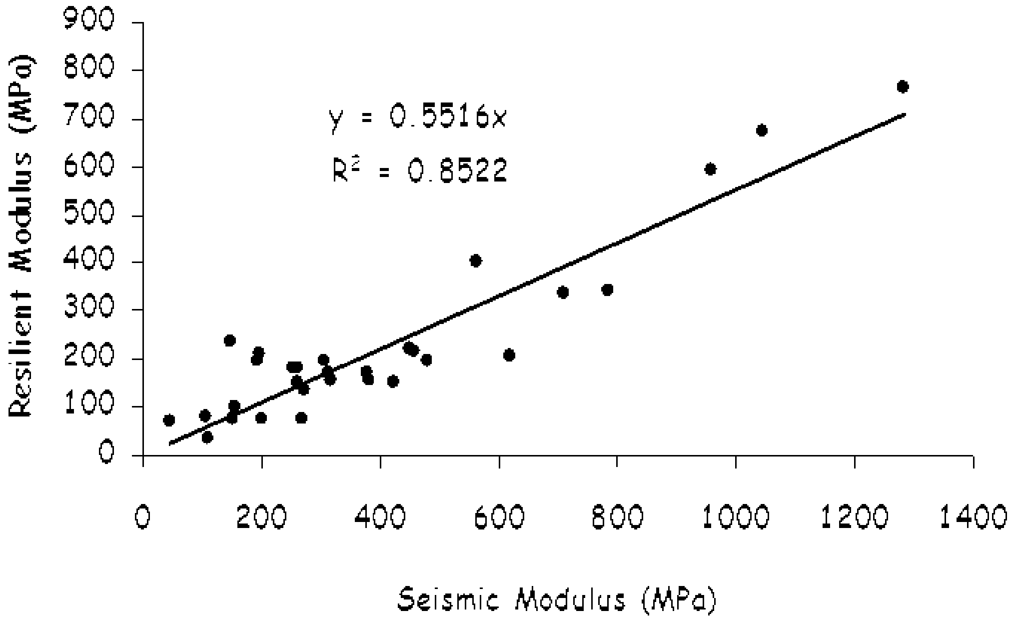

Figure 1: Relationship between Seismic and Resilient Moduli (Nazarian *et al*., 2002)

2.5 BSM – emulsion derived correlations

The correlations were developed based on the measurements taken on the same day. The best fit was based on the best coefficient of determination (R^2) and Standard Error of Estimate (SEE). A RAG (red, amber, green) system was used to rate the regressions correlations. For R^2, the red colour was used for values of 0 to 0.5, amber for values of 0.51 to 0.8 and green for values of 0.81 to 1. For SEE, the red colour was used for values of 10.01 upward, amber for values of 1.01 to 10.00 and green for values of 0 to 1.

2.5.1 *CIST and RCCD*

Reasonably good correlation between the CIST and the RCCD was found as shown in Table 5. Although the RCCD measures the in situ shear resistance characteristics while the CIST measures elastic properties of materials, both devices have a shallow depth of influence. The derived power model matches with the expected model from the combination of Equation (2) with Equations (7) to (11).

Table 5: Correlations between CIST and RCCD

Parameters		**Regression Equation**	**R^2**	**SEE**	**Best fit type**
y [CIST]	**x [RCCD]**				
CIV	RCCD pen (mm/blow)	$y = 551.072x^{-1.004}$	0.705	0.25	Power

2.5.2 *LWD and FWD*

For the LWD, only deflection measurements and bowl parameters were used to develop the correlations with others devices while for the FWD, maximum deflection (Ymax) and three bowl parameters (BLI, MLI and LLI) were used. Table 6 summarizes the correlation models found.

Table 6: Correlations between LWD and FWD

Parameters		Regression Equation	R^2	SEE	Best fit type
Y [LWD]	X [FWD]				
D_0 (μm)	Ymax (μm)	$y = 0.361x^{0.983}$	0.619	0.27	Power
	BLI (μm)	$y = 1.617x^{0.823}$	0.609	0.27	Power
	MLI (μm)	$y = 2.450x^{0.888}$	0.569	0.29	Power
	LLI (μm)	$y = 10.19x^{0.735}$	0.307	0.36	Power
D_{300} (μm)	Ymax (μm)	$y = 0.158x^{0.928}$	0.817	0.15	Power
	BLI (μm)	$y = 1.104x^{0.683}$	0.622	0.22	Power
	MLI (μm)	$y = 0.667x^{0.916}$	0.897	0.11	Power
	LLI (μm)	$y = 1.129x + 1.963$	0.849	6.06	Linear
D_{600} (μm)	Ymax (μm)	$y = 0.235x^{0.742}$	0.666	0.18	Power
	BLI (μm)	$y = 1.377x^{0.508}$	0.438	0.24	Power
	MLI (μm)	$y = 0.911x^{0.690}$	0.647	0.19	Power
	LLI (μm)	$y = 0.873x^{0.874}$	0.818	0.13	Power
RoC (m)	Ymax (μm)	$y = -553,579\ln(x) + 3.770,280$	0.564	169.81	Logarithmic
	BLI (μm)	$y = -472,625\ln(x) + 2.976,631$	0.576	167.44	Logarithmic
	MLI (μm)	$y = -487,031\ln(x) + 2.628,986$	0.490	183.47	Logarithmic
	LLI (μm)	$y = -332,566\ln(x) + 1.579,526$	0.180	232.73	Logarithmic
BLI (μm)	Ymax (μm)	$y = 0.164x^{1.045}$	0.459	0.39	Power
	BLI (μm)	$y = 0.609x^{0.925}$	0.505	0.38	Power
	MLI (μm)	$y = 1.498x^{0.907}$	0.390	0.42	Power
	LLI (μm)	$y = 10.17x^{0.630}$	0.148	0.50	Power
MLI (μm)	Ymax (μm)	$y = 0.026x^{1.109}$	0.769	0.21	Power
	BLI (μm)	$y = 0.221x^{0.855}$	0.641	0.26	Power
	MLI (μm)	$y = 0.126x^{1.131}$	0.900	0.14	Power
	LLI (μm)	$y = 0.373x^{1.130}$	0.707	0.24	Power

The maximum deflections determined with LWD (D_0) and the FWD (Ymax) yielded a fair correlation. This can be ascribed to the difference in contact pressure, the shallow depth of influence of the lighter LWD weight as well as the thin BSM-emulsion layer.

The LWD deflection at 300mm (D_{300}) shows good correlations with the FWD set of deflection bowl parameters. The correlations between deflection bowl parameters of the same nature of LWD and FWD were found to be fair to good, and the correlations were power model instead of linear as expected and proposed by Horak and Khumalo (2006) on a light granular base pavement.

2.5.3 *RCCD, LWD and FWD*

Poor correlations between the RCCD and LWD were found as shown on Table 7 below. This is to be expected as the RCCD measures the in situ shear resistance characteristics while the LWD measure elastic properties of materials.

Table 7: Correlations between RCCD and LWD

Parameters		Regression Equation	R^2	SEE	Best fit type
Y [RCCD]	X [LWD]				
RCCD pen (mm/blow)	D_0 (μm)	y = 0.049x + 10.32	0.198	8.86	Linear
	D_{300} (μm)	$y = 10.433x^{0.141}$	0.01	0.38	Power
	D_{600} (μm)	$y = 6.853x^{0.295}$	0.036	0.38	Power
	RoC (m)	y = -6,49ln(x) + 56,15	0.133	9.22	Logarithmic
	BLI (μm)	y = 0.058x + 12.09	0.227	8.70	Linear
	MLI (μm)	$y = 16.92045x^{0.02633}$	0.001	0.39	Power

Due to its shallow depth of influence, the correlation between RCCD penetration and D_0, RoC and BLI did show some improvement in coefficient of determination, R^2. A similar pattern was found between the RCCD and FWD.

2.5.4 *CIST, LWD and FWD*

Poor correlations between the CIST and LWD were found as shown in Table 8 below. This is similar to the weak correlation found with the RCCD, due to its shallow depth of influence. The correlation between CIV and D_0, RoC and BLI did show some improvement in coefficient of determination, R^2. A similar pattern was found between the CIST and FWD with significant improvement of SEE values.

Table 8: Correlations between CIST and LWD

Parameters		Regression Equation	R^2	SEE	Best fit type
y [CIST]	X [LWD]				
CIV	D_0 (μm)	y = -15.604ln(x) + 113.193	0.307	10.20	Logarithmic
	D_{300} (μm)	y = -13.8ln(x) + 88.59	0.096	11.65	Logarithmic
	D_{600} (μm)	y = -14.4ln(x) + 80.70	0.085	11.72	Logarithmic
	RoC (m)	y = 12,59ln(x) - 37,58	0.326	10.06	Logarithmic
	BLI (μm)	y = -12.6ln(x) + 92.28	0.326	10.06	Logarithmic
	MLI (μm)	y = -10.1ln(x) + 66.12	0.081	11.75	Logarithmic

2.5.5 *PSPA with other devices*

The PSPA was only available twice after construction of part of the BSM-emulsion sections for measurements. As shown in Tables 9 and 10, no significant correlations were obtained.

Table 9: Correlations between PSPA, CIST and RCCD

Parameters		Regression Equation	R^2	SEE	Best fit type
y [PSPA]	x [Others]				
E (MPa)	CIV	y = 22.57x + 1909.281	0.074	655.22	Linear
	RCCD pen (mm/blow)	y = -905.192ln(x) + 5351.763	0.049	654.82	Logarithmic

Table 10: Correlations between PSPA and LWD

Parameters		Regression Equation	R^2	SEE	Best fit type
y [PSPA]	X [LWD]				
E (MPa)	D_0 (μm)	$y = 2403.846e^{0.001x}$	0.027	0.24	Exponential
	D_{300} (μm)	$y = 1375.269x^{0.173}$	0.04	0.24	Power
	D_{600} (μm)	$y = 1148.072x^{0.261}$	0.05	0.24	Power
	RoC (m)	$y = 0.585x + 2532$	0.022	663.59	Linear
	BLI (μm)	$y = 2527.911e^{0.001x}$	0.016	0.24	Exponential
	MLI (μm)	$y = 1889.436x^{0.115}$	0.031	0.24	Power

This lack of correlation between the two devices may be due to the limited number of tests performed or it may be a result of the lack of a clear testing protocol for PSPA at the time the tests were carried out.

3 CONCLUSIONS AND RECOMMENDATIONS

The conclusions drawn in this study are specific to the BSM-emulsion material. Note that no moisture control was carried out during the testing. Previous studies reported good correlations between elastic modulus determined with the LWD and the FWD but such correlations are material type and pavement structure dependent. This study focused on correlations between the LWD deflection bowl parameters with that of the FWD. Fair to good correlations were found between the LWD and the FWD deflection bowl parameters of the similar nature, although the expected models were linear, it was found a power model in general gave the best correlation. Reasonably good correlation was determined between the CIV values, which is the CIST measurement, and the RCCD penetration. The RCCD measures the in situ shear resistance characteristics while the CIV measures elastic properties of materials which may explain the low correlation. Correlations between the PSPA and other devices were found to be generally poor.

Further investigation should be carried out, including pavement structures with a wider stiffness modulus range, to confirm whether the findings of this study are not material specific. Additionally in situ parameters such as temperature and moisture should be monitored to confirm and quantify their influence on the correlations that have been developed.

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