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Research Paper

Evaluation of Pavement Structural Number and Resilient Modulus in a Schistose Quartzite/Quartzite Environment using Dynamic Cone Penetration Test Data: Consequence for flexible pavement construction

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Abstract

In-situ dynamic cone penetration test was deplored in this study to determine and correlate important geotechnical attributes of the foundation soil in quartzite and schistose quartzite dominated rocks. The DCPT penetrative indexes at refusal ranged from 0.1 to 6.4 mm/blow, with average cumulative number of blows of 97. The average CBR value of the subgrade and subbase/base are 33 % and 66 % respectively. The average strength coefficients for subbase (0.08) and base (0.097) showed that the soil has higher strength for base layer than subbase course due to obtained CBR value. The estimated Structural Number is 3.59, which is capable of sustaining any imposed traffic load, more than 30 msa (ESAL). The average subgrade moduli and elastic moduli of the soil are 439 MPa and 1167 MPa accordingly, with soil derived from schist showing better subgrade strength properties in terms of M_R/E_S, of 458/1258MPa than quartzite (387/924MPa) based on its mineralogical composition. Regression model showed exponentially weak positive correlation coefficients for M_R and E_S of 0.0165 and 0.0276 for quartzite, and schistose quartzite respectively. Consequently, the soil parameters in terms of subgrade and elastic modulus can sustain and support flexible pavement construction in the study area.

Introduction

Pavement evaluation is a crucial pre-construction test used to ascertain or appraise a roadway section's structural problems in order to plan remedial action or conduct routine monitoring

(Wright, 1986; Falowo & Dahunsi, 2020). The pavement's structural capability is the focus of the pavement structural condition. Although several techniques are employed to determine these crucial structural characteristics, the non-destructive test methodology is the most highly recommended way for assessing the structural capability of flexible pavement (Bell, 2007). A dynamic cone penetrometer test was performed at Rufus Giwa Polytechnic in Owo, Ondo State, Nigeria (Figure 1) in an effort to assess the subsurface's structural capability and correlate modulus parameters for pavement construction (Ubido et al., 2021). The study area lacked baseline information on soil index properties, pertinent for pavement design and construction. In addition, recent failures of existing roads in the campus, necessitated this study, as information from failures of those roads attributed them to poor design and construction processes, arising from insufficient data on the soil characteristics. Hence, to solve the problem, this study utilized existing subgrade CBR and modulus models to derive the elastic and subgrade modulus; and the UK DCP 3.1 software to calculate the in-situ California bearing ratio (CBR). The dynamic cone penetration (DCP) is a method that uses structural estimators such as moduli and structural number to assess the state of the pavement and how long it will continue to function effectively. The condition of structural strength is expressed in terms of the Structural Number (SN). DCP is a useful tool for managing and assessing pavement conditions (Ubido et al., 2021; Transport and Road Research Laboratory, 1990; Thach Nguyen and Mohajerani, 2015; Shankar et al., 2009; Siekmeier et al., 2000). Similar to the cone penetrometer test, dynamic cone penetration test (DCPT) involves driving the cone into the soil, rather than pushing it at a steady pace. The number of blows required to advance the cone in 6-inch increments is recorded. Usually, there are two increments in a single test. Testing can be done at predetermined intervals using a retractable cone and moving the hole forward with an auger or other tool in between tests, or it can be done continuously to the desired depth using an expendable cone that is left in the ground when the drill rod is withdrawn. In general, material type and relative density may be determined using blow counts. For pavement structure modeling, this method is simple, affordable, and needs fundamental material characteristics (Quansah et al., 2017; Paige-Green and Van Zyl, 2019; Rolt and Pinard, 2016; Falowo, 2023a). The test which is shallow (less than 1.0 m in depth) was performed in accordance with ASTM D 6951. The practical significance of DCP index and CBR value are very important in predicting the serviceability performance of a pavement, because high CBR values and low penetrative index are indications of competent, stiff soil material.

Subsequently, this study presents important index properties and parameters correlation of soil in the study area, for flexible pavement construction, using DCPT. It encompasses characterization of the pavement foundation, and identification and propose treatment of special subsurface condition requiring improvement and strengthening. Thus, from the analysis of the DCP test data, important properties such as CBR, strength coefficient, structural contribution of the soil as base, subbase, and subgrade soil, subgrade modulus, and elastic modulus were derived. These parameters were investigated in both quartzite and schistose quartzite environment, since these rocks differ in terms of composition, texture, geological history, and susceptibility to weathering. It is imperative to develop an empirical models for parameters in both geological environment for pavement design, instead of using generalized parameters obtained from the vicinity of the study area. One of the unique aspect of this study is that the research was done to support the determination of the structural condition or subsoil via the SN – M_R approximation with the help of the DCP. The DCP was chosen because, it is easier, less expensive and quicker method compared to the other older empirically based methods of obtaining information about the pavement structural response and associated strength. It also provides additional in situ shear strength characteristics in depth of a flexible pavement and increased the confidence or correlation coefficients for the derived SN values for flexible pavements. Meanwhile, subgrade layer depends on the properties and stiffness of the soil material and M_R is the absolute measure of subgrade bearing capacity for pavement design, hence the roles of SN and M_R cannot be overemphasized in pavement design, especially in areas where baseline information on these parameters are unavailable.

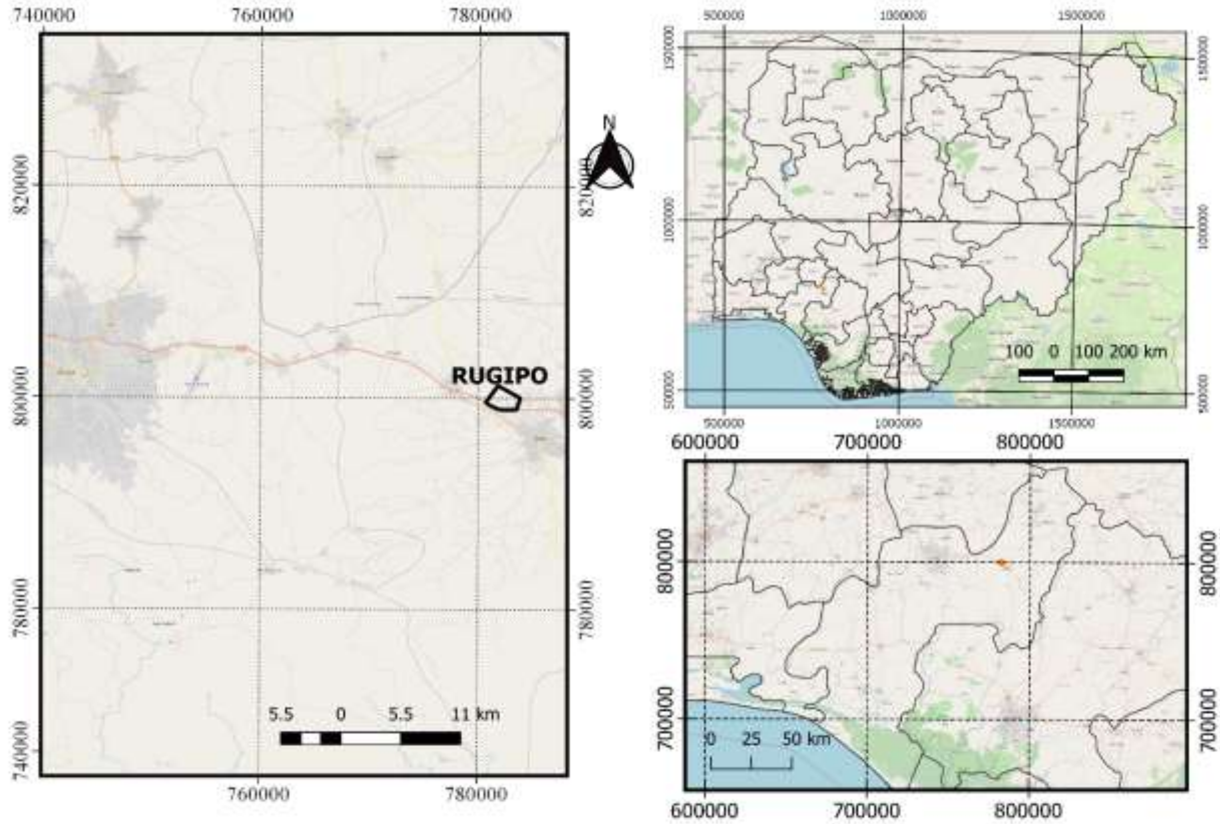
Materials and Methods

Study Area

Rufus Giwa Polytechnic in Owo, southwest Nigeria, is the research location (Figure 1). The institution is situated between the Universal Traverse Mercator (UTM) coordinates of Northing 798500 – 801500 m and Easting 781000 – 784000 m in the northeastern part of Ondo State (Figure 1). The area has temperatures between 24 and 28°C and more than 1500 mm of rainfall on average each year (Federal Meteorological Survey, 1982; Iloeje, 1981; Falowo and Imeokparia, 2015). Figure 2 shows that the area's digital elevation model (DEM) ranged from 300 to 347 meters. The institution's most common rock types are granite, gneisses, quartz-schist, and quartzite (Figure 3). In many locations, the schistose quartzite/quartzite rocks are underlain by the granite and gneisses, which are often found to be low lying. They do, however, also appear as ridges in the institution's eastern and central regions. There was noticeable schistocity in the schistose-quartzite rock. Joints, faults, folds, and foliation of gneisses rocks—which are marked by an alternation of light and dark minerals—are further tectonic imprints that may be seen. Hence, the study area was selected based on its broader representation of all the rocks in Owo town and environs. Therefore, most information from this study is applicable to many areas or soil associations in Owo and environs.

Method

This study employed the DCPT, which was administered at thirty-three (33) locations around the institution (Figure 3). The rationale for selecting these 33 test locations was based on the proposed site of the roads, geology, and accessibility. The DCPT is a straightforward mechanical device that can provide 45.5 Joules of energy and is used for quick in-situ strength assessment of roadway structural material, particularly the subgrade and other unbound layers. When a standard force is applied, it gauges how deeply a standard cone penetrates (Kadyali and Lal, 2008; Nam et al., 2016). Along with the number of blows and depth of penetration, the penetrative index (pen rate) in millimeters per blow of the standard hammer is recorded. The typical steel cone utilized in this investigation had a diameter of 20 mm and an angle of 60°. Additionally, a typical 8 kg hammer was used, which makes contact with the anvil to induce penetration by sliding over a 16 mm diameter steel rod with a fall height of 575 mm (Wu and Sargand, 2007; Vandre et al., 1998; Vazirani and Chandola, 2009). The obtained data was analyzed and interpreted using the UK DCP 3.1 software, written in visual basic language and uses a microsoft access database to store the data. The software is good in analyzing DCP data. The two primary purposes of the software are to analyze the data and utilize the findings to build sections of sealed roads that will be used as spot enhancements on routes with little or low trafficked roads (Done and Samuel, 2006; Deepika and Chakravarthi, 2012). It is crucial for design purposes that the DCP testing be done when the pavement is at its weakest, or with the maximum moisture content (MMC). The MMC was controlled during testing by making sure all sampled site were tested within one hour of starting the in-situ testing. As indicated in Table 1, the data collected at each location was adjusted for moisture content, in order to have uniform moisture condition, so as to prevent error when determining the CBR using the TRL (Transport and Road Research Laboratory, 1990) relationship. From Site No. 1 to Site No. 33, each test site was assigned a serial number. The limitations of DCPT that have to do with depth, granular soil, seasonal moisture fluctuations, surcharge loading phenomenon, difficulty in penetrating stabilized layers or granular material with large particles, and sticking of the cone into sampled soil, were all managed and taken into cognizance during testing.



(a)

(b)

Fig. 1. Research Location of the study area, showing aerial distribution of the existing roads/access roads and general geographic setting of the area, on (a) map of Ondo State and Nigeria, (b) Google map

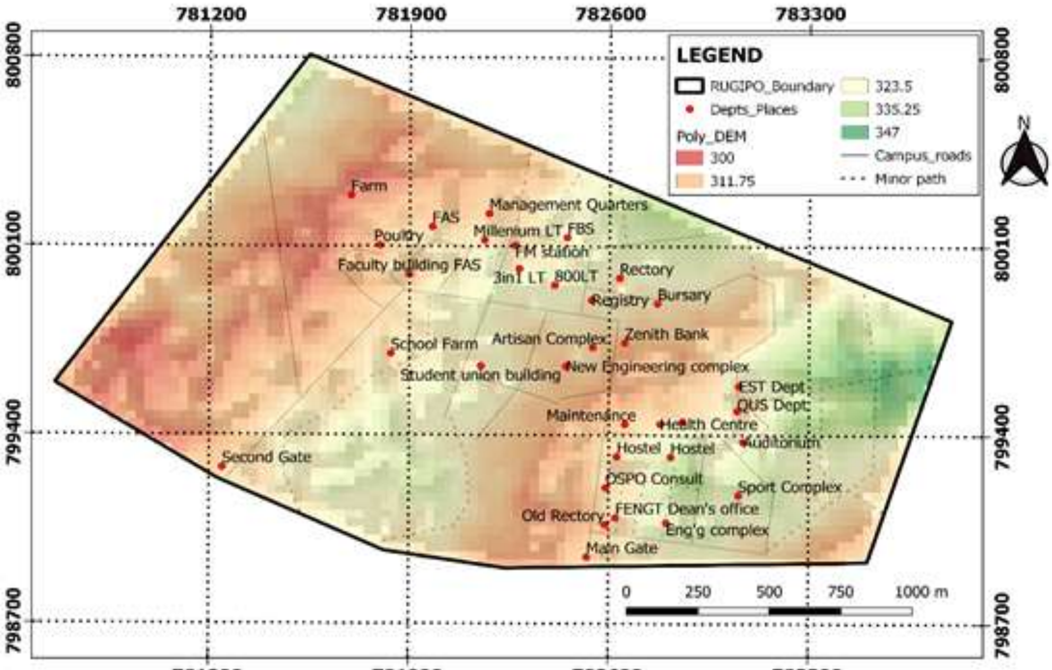


Fig. 2. DEM image of the study area showing predominant elevation variation of 311 to 335 m above sea level

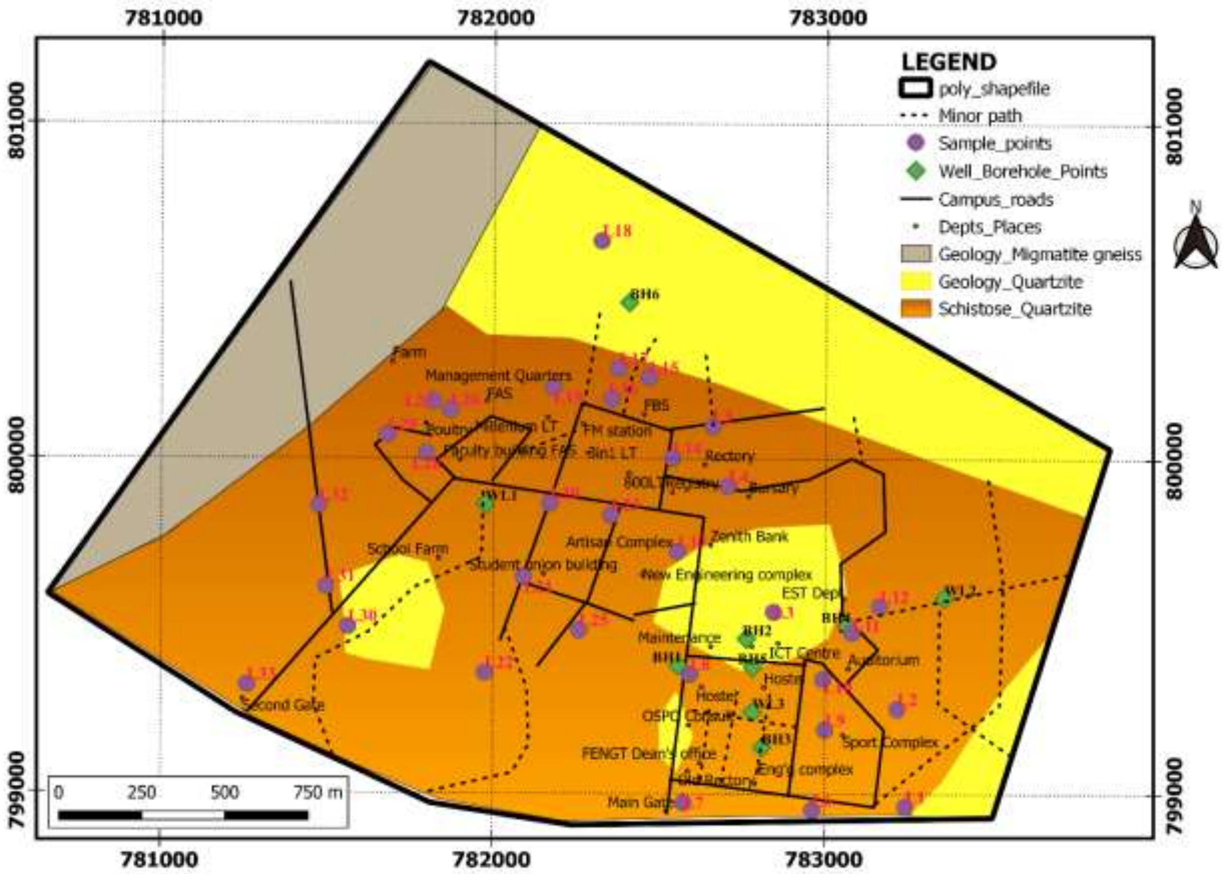


Fig. 3. Geological map of the Campus showing predominant schistose quartzite formation (overlay are DCPT points) across which many of the existing roads are founded

Table 1. CBR Adjustment Factor (Transport and Road Research Laboratory, 1990)

Surface moisture	Ratio of in-situ moisture to OMC (modified AASHTO)	Default CBR Adjustment Factor
Wet	1	1
Moderate	0.75	0.71
Dry	0.5	0.51
Very dry	0.25	0.37
Unknown (not assessed or difficult to assess	-	0.5

In order to determine the strength coefficient of the test sites, the penetration rate was converted to the CBR value, followed by the strength coefficient and lastly the structural number (SN), also known as the modified structural number (SNC or SNP). SNP indicates how much each pavement layer contributes to the depth-adjusted SNP. SN and SNC have the same surface and base values since this change only affects the sub-base and subgrade. Equation 1 (TRL equation) was used for the CBR computation. Using equations 2 (for base) and 3 (for subbase), the strength coefficient "a" of the subsoil suitable for use as the base and subbase layers is determined.

$$\mathrm{Log}_{10}^{(\mathrm{CBR})} = 2.48 - 1.057\ \mathrm{Log}_{10}^{(\mathrm{pen\ rate})} \quad (1)$$

$$a_{base} = 0.0001[29.14\ (\mathrm{CBR}) - 0.1977\ (\mathrm{CBR})^2\ + 0.00045\ (\mathrm{CBR})^3 \quad (2)$$

$$a_{subbase} = 0.184\ \mathrm{Log}_{10}^{(\mathrm{CBR})} - 0.0444\ \left(\mathrm{Log}_{10}^{(\mathrm{CBR})^2}\right) - 0.075 \quad (3)$$

Resilient modulus (M_R), in addition to CBR, is calculated to evaluate the pavement sub-grade's performance. The subgrade resilient modulus can be anticipated directly from the DCP findings or indirectly from the relationship between the sub-grade modulus (M_R), elastic modulus (E_S), and CBR. The established model correlation used in this case is given by the following relationship according to Carter and Bentley (1991) for subgrade modulus and modulus of elasticity in equations 4 and 5:

$$\mathrm{M_R} = 338\ \times\ \mathrm{DCPI}^{-0.39} \quad (4)$$

$$\mathrm{E}_s = 664.67\ \times DCPI^{-0.7168} \quad (5)$$

Results and Discussion

DCPT

The summary of the DCPT investigation, showing the CBR of the subgrade, subbase/base, strength coefficient and corresponding depth/thickness of the layers, at every points sampled is presented in Tables 2 and 3. The penetrative indexes at refusal ranged from 0.1 (TP-4) to 6.4 mm/blow (TP-3), with cumulative number of blows of 54 – 132 (avg. 97), which drove the steel rod to refusal depths ranging from 234 – 874 mm. This depths represent zone the soil is grading stiffly to hardpan lateritic sand, and commonly found in schistose quartzite environment. The CBR value of the subgrade ranged from 6 (TP-7) – 92 % (TP-6) with an average (avg.) of 33 %, the subbase/base layer ranged from 6 (TP-12 & 27) to 596 % (TP-9) with average of 66 %. The high variability in CBR values across the area is a function of degree of compaction, rock weather-ability, soil composition, and textural characteristics of the soil. Hence, the influence of the variability of the CBR values on the stability and performance of the pavement will be high. Thus, extreme caution must be taken when designing the thickness of the pavement using the CBR method.

Layers Strength Characteristics

The strength coefficients varied from 0.02 to 0.14 (avg. 0.08) and 0.04 to 0.14 (avg. 0.097) for subbase and base courses respectively, even though some degree of overlapping exists in the strength coefficients. The subbase layer shows higher strength (a little) than the base because high CBR more than 80 % are required for base layer, while the obtained average of 33 % is sufficient for a subbase course. Hence the foundation material showed higher strength for base layer than a subbase course. The depth and thickness of the layers at refusal ranged from 89 – 874 mm (463 mm) and 18 – 744 mm (287 mm). These thickness/depth are adequate to bear traffic load impact on the proposed pavement of more than 20 msa. The expected structural contribution of each of the layers to the structural number for the pavement is showed Table 3. The total contributions for SN ranging from 1.81 – 5.45 (avg. 3.59), SNC (with subgrade inclusion) is between 3.07 – 6.68 (5.09), and SNP varying from 2.93 – 5.88 (avg. 4.66). The average SN for the base and subbase courses are 1.56 and 2.03. The average SNC recorded for base, subbase and subgrade are 1.56, 2.03, and 1.49 respectively, while 1.56, 1.61, and 1.49 were recorded as average SNP contributions for base, subbase, and subgrade respectively. Since, in most flexible pavement design the SN is mostly used in determination of structural thickness of pavement (AASHTO, 1993; Falowo, 2023a). Hence, the average SN of 3.59 is above 3.0 minimum (AASHTO, 1993) threshold of pavement of this caliber.

Table 2. Summary of the DCPT results, in terms of CBR, Refusal depth, delineated layers and their corresponding thicknesses and strength coefficients

Test Point	Coordinates		Elev. (m)	Layer	CBR (%)		Thickness	Depth	Strength coefficient	
	E (m)	N (m)			Base/Subbase	Subgrade	(mm)	(mm)	Base	Subbase
1	0783243	0798963	319	1	58	50	338	338	0.11	0.11
				2	420	50	18	356	0.14	0.12
2	0783218	0799254	325	1	23	23	402	402	0.06	0.07
				2	142	50	69	471	0.14	0.14
3	0782845	0799543	319	1	10	10	116	116	0.03	0.08
				2	44	44	610	726	0.09	0.11
4	0782706	0799918	326	1	13	13	731	731	0.04	0.08
				2	220	50	63	794	0.14	0.12
5	0782659	0800097	332	1	18	18	251	251	0.05	0.09
				2	89	50	222	473	0.13	0.11
6	0782963	0798952	330	1	11	11	491	491	0.03	0.07
				2	92	92	177	668	0.14	0.12
7	0782575	0798973	312	1	9	9	193	193	0.02	0.06
				2	129	50	194	387	0.14	0.12
8	0782593	0799356	315	1	37	37	290	290	0.08	0.10
				2	158	50	121	411	0.12	0.12
9	0783000	0799192	337	1	18	18	670	670	0.05	0.09
				2	596	50	24	694	0.14	0.12
10	0782994	0799342	336	1	18	18	623	623	0.05	0.09
				2	95	50	136	759	0.14	0.12
11	0783081	0799481	323	1	14	14	582	582	0.04	0.08
				2	85	50	166	748	0.13	0.11
12	0783164	0799560	328	1	6	6	89	89	0.02	0.04
				2	108	50	282	371	0.14	0.12
13	0782555	0799723	320	1	8	8	597	597	0.02	0.06
				2	29	29	260	857	0.07	0.10
14	0782538	0800002	328	1	13	13	485	485	0.03	0.07
				2	48	48	305	790	0.10	0.11
15	0782467	0800242	327	1	8	8	480	480	0.02	0.05
				2	47	47	394	874	0.10	0.11
16	0782355	0800179	324	1	10	10	457	457	0.03	0.07
				2	106	50	131	588	0.14	0.12
17	0782376	0800269	328	1	9	9	398	398	0.02	0.06
				2	47	47	268	666	0.10	0.11
18	0782323	0800650	327	1	15	15	398	398	0.04	0.08
				2	50	50	212	610	0.10	0.11
19	0782179	0800212	326	1	13	13	352	352	0.04	0.08
				2	104	50	127	479	0.14	0.12
20	0782169	0799866	329	1	15	15	407	407	0.04	0.08
				2	105	50	188	595	0.14	0.12
21	0782091	0799647	330	1	13	13	477	477	0.03	0.08
				2	79	50	220	697	0.13	0.11
22	0781975	0799360	327	1	25	25	299	299	0.06	0.10
				2	102	50	139	438	0.14	0.12
23	0782353	0799831	327	1	9	9	288	288	0.03	0.06
				2	34	34	553	841	0.08	0.10
24	0788275	0799577	324	1	17	17	467	467	0.04	0.09
				2	78	50	223	690	0.13	0.11
25	0782259	0799488	326	1	19	19	239	239	0.05	0.09
				2	119	50	127	366	0.14	0.12
26	0781869	0800145	323	1	20	20	116	116	0.05	0.09

				2	95	50	118	234	0.14	0.12
27	0781818	0800170	323	1	6	6	182	182	0.02	0.04
				2	49	49	226	408	0.10	0.11
28	0781797	0800015	330	1	8	8	109	109	0.02	0.06
				2	31	31	744	853	0.07	0.10
29	0781680	0800070	321	1	16	16	441	441	0.04	0.08
				2	80	50	183	624	0.14	0.11
30	0781563	0799497	334	1	23	23	119	119	0.06	0.09
				2	97	50	239	358	0.14	0.12
31	0781497	0799617	326	1	17	17	145	145	0.04	0.08
				2	107	50	176	321	0.14	0.12
32	0781474	0799859	320	1	23	23	101	101	0,06	0.09
				2	120	50	250	351	0.14	0.12
33	0781260	0799322	324	1	34	34	292	292	0.08	0.10
				2	97	50	158	450	0.14	0.12

Soil Modulus Properties

Pavement evaluation relies mainly on information on the stiffness (i.e. resistance of soil material to stress-induced deformation) of pavement layers, and the modulus of subgrades as references, in addition to supplementary data on density and moisture content (Falowo, 2023b, Falowo et al., 2023; Amer et al., 2014). It is often required to estimate the subgrade-stiffness or modulus of the pavements, before and after their construction as part of the quality-control measures, and also for quality assurance (Amosun et al., 2018). The subgrade moduli of the soil ranged from 141 - 830 MPa (avg. 439 MPa), while elastic moduli varied from 133 to 3463 MPa (avg. 1167 MPa). The quartzite is characterized with M_R and E_S values ranging from 164 – 633 MPa (avg. 387 MPa) and 176 – 2107 MPa (924 MPa), while schistose quartzite ranged from 141 – 830 MPa (avg. 458 MPa) and 133 – 3463 MPa (1258 MPa). The shows that schistose derived soil has better subgrade strength properties than quartzite, due to the fact that predominant mineral in quartzite is quartz, which in most places lack sufficient binding materials which can enhances its induration (hardening) process during lithification and diagenesis, unlike schistose quartzite which are very rich clay/mafic minerals.

Regression Models

Table 4 showed the relationship or correlation of subgrade modulus (M_R) and elastic modulus (E_S) of quartzite (taking as dependent variable) and schistose quartzite (as independent variable) for different trend lines. The logarithmic and polynomial equations give the best weak positive correlation coefficients (r^2) for M_R parameter of 0.0991/0.1496, while exponential and polynomial showed the best weak trend lines (correlation coefficients (r^2) for E_S (Tables 5 and 6). The weak correlations for these parameters is attributed to geological composition, textures and structures, since both rocks are under the same weather condition, with the same geomorphological attributes. However, limited testing points for quartzite could also be a factor responsible for the weak correlations.

Table 3. Summary of the DCPT results, in terms of the structural contribution of each layers

Point No.	PI (mm/blow) At Refusal	No. of blows	Depth (mm)	Structural Contribution Layer	SN	SNC	SNP
1	0.30	122	356	Base	1.58	1.58	1.58
				Subase	1.56	1.56	1.52
				Subgrade		2.08	2.08
2	1.80	85	471	Base	1.29	1.29	1.29
				Subase	1.79	1.79	1.59
				Subgrade		1.77	1.77
3	6.40	123	726	Base	2.37	2.37	2.37
				Subase	2.87	2.87	2.07
				Subgrade		1.44	1.44
4	0.10	96	794	Base	1.37	1.37	1.37
				Subase	2.46	2.46	1.71
				Subgrade		1.44	1.44
5	0.30	108	473	Base	1.63	1.63	1.63
				Subase	1.86	1.86	1.62
				Subgrade		1.64	1.64
6	0.70	93	668	Base	1.54	1.54	1.54
				Subase	2.15	2.15	2.11
				Subgrade		1.33	1.33
7	0.30	117	387	Base	1.28	1.28	1.28
				Subase	1.33	1.33	1.22
				Subgrade		1.12	1.12
8	0.20	132	411	Base	1.63	1.63	1.63
				Subase	1.74	1.74	1.62
				Subgrade		1.98	1.98
9	0.50	107	694	Base	1.36	1.36	1.36
				Subase	2.38	2.38	1.80
				Subgrade		1.64	1.64
10	0.60	102	759	Base	1.84	1.84	1.84

				Subase	2.71	2.71	1.91
				Subgrade		1.62	1.62
11	0.40	94	748	Base	1.70	1.70	1.70
				Subase	2.51	2.51	1.76
				Subgrade		1.47	1.47
12	0.40	94	748	Base	1.62	1.62	1.62
				Subase	1.43	1.43	1.35
				Subgrade		1.52	1.52
13	0.80	54	857	Base	1.21	1.21	1.21
				Subase	2.27	2.27	1.39
				Subgrade		1.01	1.01
14	0.50	91	790	Base	1.86	1.86	1.86
				Subase	2.73	2.73	1.83
				Subgrade		1.42	1.42
15	1.30	93	874	Base	1.92	1.92	1.92
				Subase	2.69	2.69	1.58
				Subgrade		1.02	1.02
16	0.40	80	588	Base	1.23	1.23	1.23
				Subase	1.79	1.79	1.40
				Subgrade		1.26	1.26
17	1.0	71	666	Base	1.41	1.41	1.41
				Subase	2.07	2.07	1.49
				Subgrade		1.13	1.13
18	0.80	75	610	Base	1.47	1.47	1.47
				Subase	2.17	2.17	1.70
				Subgrade		1.53	1.53
19	0.20	79	479	Base	1.19	1.19	1.19
				Subase	1.63	1.63	1.41
				Subgrade		1.45	1.45
20	0.20	110	595	Base	1.66	1.66	1.66
				Subase	2.13	2.13	1.68
				Subgrade		1.51	1.51
21	0.40	101	697	Base	1.77	1.77	1.77
				Subase	2.40	2.40	1.73

				Subgrade		1.43	1.43
22	0.40	97	438	Base	1.48	1.48	1.48
				Subase	1.76	1.76	1.59
				Subgrade		1.82	1.82
23	1.60	94	841	Base	1.99	1.99	1.99
				Subase	2.93	2.93	1.87
				Subgrade		1.17	1.17
24	0.40	108	690	Base	1.92	1.92	1.92
				Subase	2.53	2.53	1.86
				Subgrade		1.59	1.59
25	0.20	88	366	Base	1.17	1.17	1.17
				Subase	1.40	1.40	1.33
				Subgrade		1.67	1.67
26	0.20	63	234	Base	0.87	0.87	0.87
				Subase	0.94	0.94	0.97
				Subgrade		1.70	1.70
27	0.80	56	408	Base	1.01	1.01	1.01
				Subase	1.27	1.27	1.13
				Subgrade		0.79	0.79
28	1.70	107	853	Base	2.25	2.25	2.25
				Subase	3.20	3.20	2.11
				Subgrade		1.09	1.09
29	0.60	94	624	Base	1.64	1.64	1.64
				Subase	2.24	2.24	1.73
				Subgrade		1.55	1.55
30	1.90	117	358	Base	1.56	1.56	1.56
				Subase	1.52	1.52	1.46
				Subgrade		1.77	1.77
31	0.50	97	321	Base	1.23	1.23	1.23
				Subase	1.28	1.28	1.21
				Subgrade		1.61	1.61
32	1.20	125	335	Base	1.63	1.63	1.63
				Subase	1.51	1.51	1.51
				Subgrade		1.77	1.77

33	0.50	114	450	Base	1.76	1.76	1.76
				Subase	1.90	1.90	1.72
				Subgrade		1.96	1.96

Table 4. Result summary of the elastic modulus (E_S) and subgrade modulus M_R of the soil

DCP Points	PI (mm/blow)	M_R (MPa)	E_S (MPa)	Geology
1	0.3	541	1575	Schistose quartzite
2	1.8	269	436	Schistose quartzite
3	6.4	164	176	Quartzite
4	0.1	830	3463	Schistose quartzite
5	0.3	541	1575	Schistose quartzite
6	0.7	388	858	Schistose quartzite
7	0.3	541	1575	Schistose quartzite
8	0.2	633	2107	Quartzite
9	0.5	443	1092	Schistose quartzite
10	0.6	413	959	Schistose quartzite
11	0.4	483	1282	Quartzite
12	9.4	141	133	Schistose quartzite
13	0.8	369	780	Quartzite
14	0.5	443	1092	Schistose quartzite
15	1.3	305	551	Schistose quartzite
16	0.4	483	1282	Schistose quartzite
17	1	338	665	Schistose quartzite
18	0.8	369	780	Quartzite
19	0.2	633	2107	Schistose quartzite
20	0.2	633	2107	Schistose quartzite
21	0.4	483	1282	Schistose quartzite
22	0.4	483	1282	Schistose quartzite
23	1.6	281	475	Schistose quartzite
24	0.4	483	1282	Schistose quartzite
25	0.2	633	2107	Schistose quartzite
26	0.2	633	2107	Schistose quartzite
27	0.8	369	780	Schistose quartzite
28	1.7	275	454	Schistose quartzite

29	0.6	413	959	Schistose quartzite
30	1.9	263	420	Quartzite
31	0.5	443	1092	Quartzite
32	1.2	315	583	Quartzite
33	0.5	443	1092	Quartzite

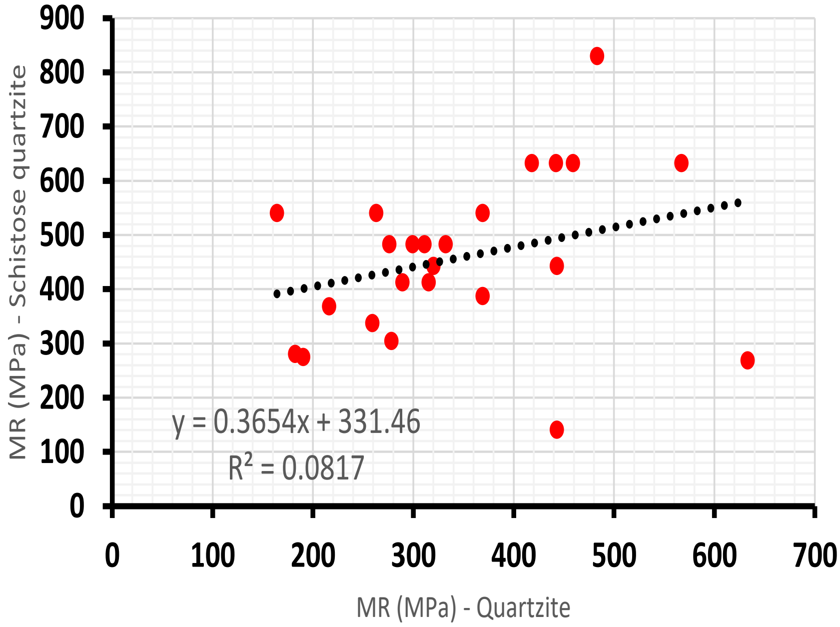


Fig. 4. Graph of the relationship or regression model M_R (between quartzite and schistose quartzite)

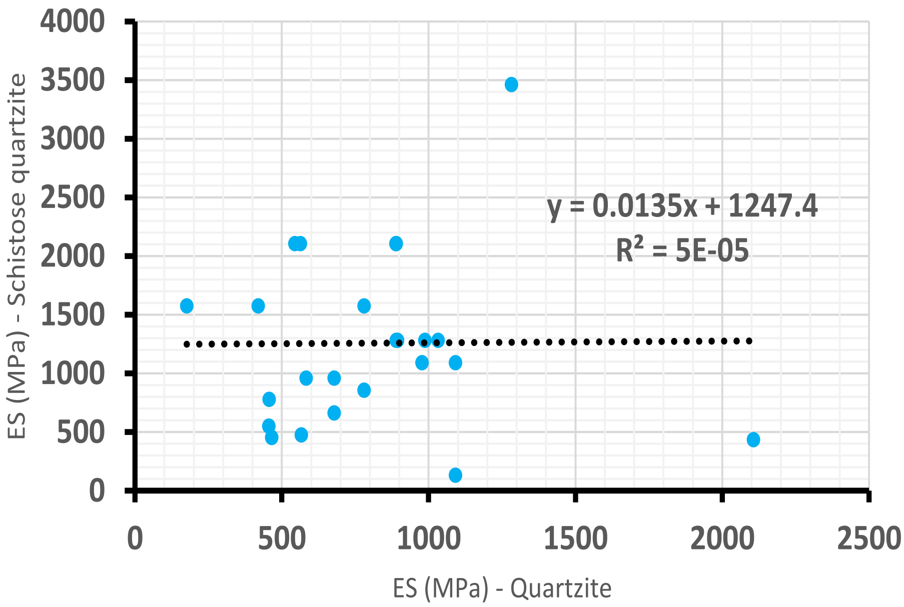


Fig. 5. Graph of the relationship or regression model E_S (between quartzite and schistose quartzite)

Table 5. M_R Empirical relationship between quartzite and schistose quartzite

Equation Type	Equation	Correlation coefficient
Exponential	$y = 359.1e^{0.0005x}$	0.0276
Linear	$y = 0.3654x + 331.46$	0.0817
Logarithmic	$y = 137.24In(x) - 336.56$	0.0991
Polynomial order 2	$y = -0.0023x^2 + 2.1395x + 28.923$	0.1496

Table 6. E_S empirical relationship between quartzite and schistose quartzite

Equation Type	Equation	Correlation coefficient
Exponential	$y = 1250.9e^{-2E-04x}$	0.0165
Linear	$y = 0.0135x + 1247.4$	-0.00005
Logarithmic	$y = 56.759In(x) + 884.61$	0.0014
Polynomial order 2	$y = -0.0006x^2 + 1.4043x + 634.43$	0.0527

Conclusion

The soil material showed higher strength and competence for subbase layer than a base course. The expected average depth of excavation of surficial weak soil is 463 mm. This superficial soil must be removed to this depth, so that the proposed pavement can be founded on competent foundation subgrade. The expected average structural contribution of each of the layers to the structural number for the pavement is 3.59, while the base and subbase courses give 1.56 and 2.03 respectively. This structural number (SN) is adequate to bear traffic load impact on the proposed pavement, since it is above minimum of 3.0 for 30 msa traffic load (ESAL). In addition, the schistose derived soil has better subgrade strength properties than quartzite, due to the fact that predominant mineral in quartzite is quartz, which in most places lack sufficient binding materials which can enhances its induration process, unlike schistose quartzite which are very rich clay/mafic minerals. Based on the test results, the schistose quartzite derived soil showed better stiffness or resistance to penetration than quartzite. Both the subgrade and elastic modulus of the derived soils generally showed exponentially weak positive correlation coefficients for M_R and E_S of 0.0165 for quartzite, and 0.0276 for schistose quartzite derived soils, and also for most equations tested. Even though, more sample testing is needed to get a more consistent correlation.

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Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

Declaration Statement

The work is original and have not been submitted in part or whole for publication elsewhere

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