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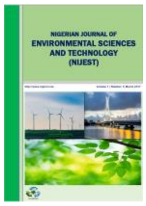

Geotechnical Evaluation of Crushed Sandstone Waste Materials from Amasiri Quarries, Southeastern Nigeria for Use in Civil Engineering Projects

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ABSTRACT

This paper presents the study of crushed sandstones wastes (CSW) aiming at their utilization as main materials for the base layer of flexible pavements with series of geotechnical laboratory tests contained in American Society for Testing and Materials adopted for their evaluations. The tests conducted included particle size distribution, Atterberg limits, compaction characteristics, California bearing ratio, unconsolidated undrained compressive strength, free swell index and one-dimensional consolidation tests. Statistical analyses using proprietary computer software were used to investigate the spatial variations of the geotechnical data, and degrees of relationships that exist among the tested parameters. From the results, the Atterberg limits tests revealed intermediate plasticity with less than 35% fines (percentage passing No. 200 British Standard sieve) in majority of the samples; an indication of good highway materials. The relatively high maximum dry density, low optimum moisture content and high shear strength recorded by the tested CSW materials proved their usability in most road projects such as sub-base and subgrade. The strength indicators adopted to evaluate the validity of the CSW as pavement materials showed relatively high shear strength as the undrained cohesion values satisfy the requirement of greater than 103 kN/m^2 values specified for base course materials. The empirical correlations analyses verified the influence of moisture on the strength parameters and gave a general overview of the geotechnical behaviors of the examined quarry wastes. Therefore, CSW materials are adjudge suitable but in order to achieve long term strength gain and stability especially in high volume applications, stabilizations will be required.

Keywords: Amasiri, Highway, Quarry waste, Subgrade, Moisture

1.0. Introduction

One major challenge in highway engineering is the economic and ecological use of soil materials. Conventional road construction materials of good quality with majority as natural resources are becoming scare due to the booming population, urbanization and industrial developments in developing tropical countries. In Nigeria, massive road reconstructions and rehabilitation have increased in Southeastern states of Nigeria and other neighboring Northern states and the demand for good sub-base, base and subgrade materials are increasingly high which had ultimately affected the construction cost. When failure of any structural element occurs, many factors often considered responsible included: (i) improper foundation investigation, (ii) poor building design, (iii) poor materials and (iv) inexperience of the handlers (Cosenza et al., 2006; Adewuyi and Philips, 2018). The use of poor materials characterized by high plasticity, weak strength, and high sensitivity to the variation in water content as substitute to known quality materials by construction companies and road engineers in most cases had resulted in most of the road failures earlier recorded in Southeastern Nigeria (Nwankwoala et al., 2014; Adiat et al., 2017). Repairs of these failed roads had become huge financial burdens to governments as huge amounts of monies are often required for reconstruction and rehabilitation of these failed roads. Therefore, it is important for precise evaluations of the geotechnical properties of any material considered for use as subbase or subgrade materials to ensure proper design and successful construction of any structure. Several researchers (Kar-Winn et al., 2001; Leong et al., 2002; Aghamelu et al., 2011) had investigated many natural materials used as substitute for use as subbase and subgrade materials and there is need to identify other suitable substitutes for use as pavement materials close to the points of constructions. However, in various quarrying operations, rocks when crushed into various sizes generate significant amounts of wastes which are often referred as quarry wastes. Zhang et al. (2019) considered the quarry wastes, as by-product of crushed aggregates for various engineering purposes. The wastes have been used in different engineering projects such as building materials, road development materials and ceramics industries. By 2050, the world is expected to generate 3.40 billion tons of wastes annually, increasing drastically from today's 2.01 billion tons (Kaza, 2018). Understanding of the geotechnical characteristics of the quarry wastes will be required for design and construction of foundations and engineering works. This is due to the facts that proper utilization of quarry wastes would conveniently decrease the demand of natural sand and would help in reducing the rate of depletion of resources and further provide some time for the resources to be replenished.

Amasiri region, southeastern Nigeria is gifted with good source of natural rocks such as sandstone, dolerite and limestone for aggregate production, thus the motivation to use the recycled materials within the region have been low in the past. During the crushing of sandstone into aggregates, huge quantities of crushed sandstones wastes (CSW) are usually generated on daily basis and the wastes considered as leftovers are greatly unutilized. Quarry waste build-up has become yet another environmental issue with its health threat. Solution to this threat lies in its utilization into useful product. Therefore, several researches on the innovative uses of waste materials are continuously advancing (Rezende and Carvalho, 2003; Nweke and Okogbue, 2017).

Many highway agencies, private organizations and individuals have completed varieties of research projects concerning the feasibility, environmental suitability and performance of different types of wastes in highway constructions (Rezende and Carvalho, 2003; Soosan et al., 2005; Nweke and Okogbue, 2017; Zhang et al., 2019). Proper utilization of these waste materials can eliminate their disposal expense and in some cases creates profitable products. In the case of the study CSW materials, only few local contractors use these waste materials for moulding road kerbs and their utilization are probably due to their cheap availability. Interestingly, their rates of production within Amasiri quarries and their surroundings increases and exceed their rate of utilization. Presently, the spaces required for waste disposal of these unutilized dust materials had become serious problem facing the quarry industries. Unfortunately, these "leftover" quarry dust materials when not appropriately used, may led to environmental and economic problems in society. Under this circumstance, an effective way of utilizing the quarry fine materials in large volume are needed, and research activities regarding the characterization of these materials to validate the promising applications become necessary.

Presently, there is no existing published research on the geotechnical properties of the Amasiri sandstone quarry wastes hence the need to explore the potentials of these waste materials in construction works has becomes very important. This paper, therefore, presents the study of crushed sandstones wastes (CSW) aiming at their utilization as main materials for the base layer of flexible pavements, especially in high volume applications. This concept of replacement of pavement materials with quarry dusts can boost the consumption of the quarried waste materials.

2.0. Methodology

2.1 Study area.

Amasiri and its suburbs are located geographically in southeastern part of Nigeria with co-ordinate of latitudes 5º53^{1}N and 5º55^{1}N and longitudes 7º54^{1}E and 7º57^{1}E (see Fig. 1). It is bounded in the north by Akpoha and Okposi, in the east by Ibii and Afikpo, in the west by Oso and Edda and in the south by Amangwu and part of Edda. The area can easily be accessed due to the good road networks such as Amasiri-Afikpo road and Amasiri-Abakaliki road as well as Amasiri-Edda road. There are numerous footpaths leading to communal farmlands and river sections. Amasiri region is characterized by an undulating relief due to the alternation of sandstone and shale and differential weathering and erosion which gave rise to the undulating land form (Igwe et al., 2013; Akana and Didei, 2017). The prevailing climate was typically tropical, with an average temperature of 25ºC and an average rainfall of about 200 and 250 cm. The ridges, however, have good drainage systems which become poor as the area tends towards the low lands. The major drainage patterns in the area were mainly trellis drainage pattern which had resulted from alternation of sandstone and shale**.**

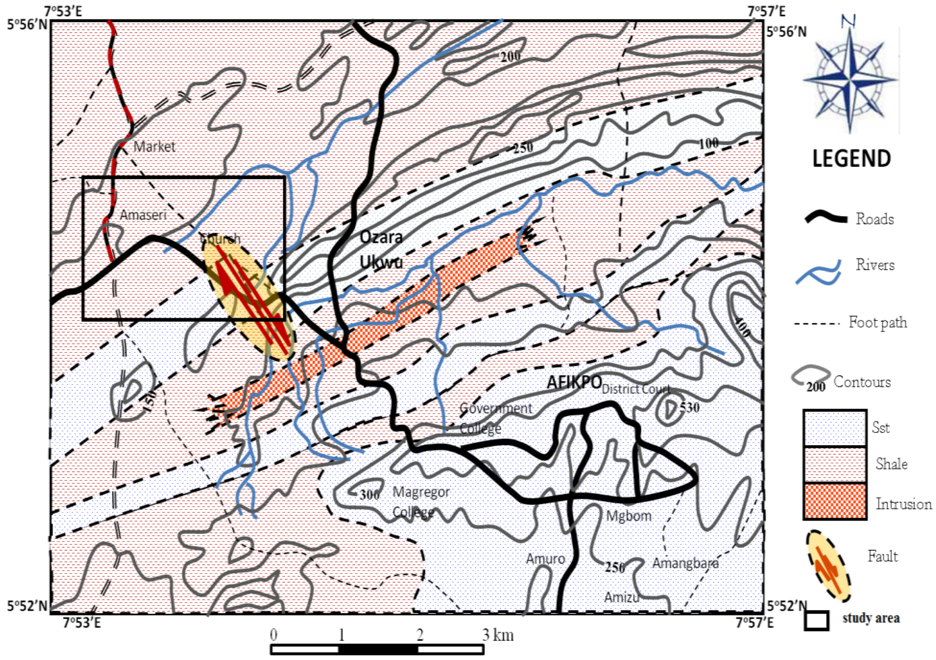

Figure 1: Map of Study Area

2.2 Sampling and preparation of the quarry waste samples

The quarry wastes are by-products of sandstone extracted from different active quarries during quarrying operation in Amasiri area. The remains of the crushed rocks after quarry operations formed the fines quarry dust. About 20-25% of the total materials crushed in a crusher units for extraction of aggregates were left as fine dusts and were considered to be waste. The samples from different locations showed no differences in color, texture and quality. A polyethylene sack of CSW materials were collected for each of the different 14 quarries, labeled and stored in dry places. The samples are herein designated as CSW1 to CSW14. To ensure that the moisture content change and other alteration processes did not affect the test results, polyethylene sacks used were sun-dried 48 hours before use and firmly sealed the moment they contained samples. Laboratory testing commenced in all cases within 48 hours of sampling at the laboratory at National Steel Raw Materials Exploration Agency, Kaduna, Nigeria.

2.3 Laboratory tests

The quality standards adopted for sample preparation and testing were as contained in American Society for Testing and Materials (ASTM, 2000) and American Association of State Highway and Transportation Official (AASHTO 1961) and local code namely the Nigerian General Specifications of Federal Ministry of Works and Housing (FMWH, 1997) for Roads and Bridge Materials as well as other established specifications in published literatures for civil engineering and geotechnical works. The particle size distribution test (wet sieving) was conducted in accordance with BS 1377 (1975) standards and the Atterberg limits tests followed the procedures of British Standard BS 1377 test 3 (1990) while linear shrinkage test followed BS 1377:1975 test 5. In linear shrinkage tests, about 150 g of air-dried soils passing 0.425 mm sieve were used. The mould was cleaned and dried and a thin film of silicone grease was applied to the inner surface to prevent soil sticking to the mould. The soil samples were placed on a glass plate and mixed properly using distilled water for about 10 minutes until a homogenous paste of about the liquid limit was achieved. The paste was then placed in the mould and tapped to eliminate air bubbles. The mould was slightly over filled and leveled off along the top edge of the mould using a spatula. The mould was exposed to air so as to achieve a slow drying process. When the soil samples shrunk away from the mould, they were transferred to the oven to dry at 60-65ºC and then at 105-110ºC. After drying, the soil samples were allowed to cool in a

desiccator. The length of the bar of soil was measured using a venire caliper, both top and bottom surfaces. The mean of the two lengths was taken as the dry length.
The linear shrinkage (LS) was calculated using equation 1:

$$LS = (1 - Ld/Lo) \times 100 \quad (1)$$

Lo = original length (14 cm).
Ld = length of dry specimen.
The natural moisture content test followed simple method outlined in accordance with BS 1377 (1975) while specific gravity test was carried out in accordance with BS 1377 (1975). An air-dried soil passed through a 4.75 mm sieve was used for the Proctor compaction tests. The Proctor compaction tests were performed as per ASTM 2000, D698a. The California bearing ratio tests were performed on the soil samples in both unsoaked and soaked conditions and they followed BS 1377 test 16 (1990) procedures. However, soaking was done overnight (24 hours) in a water-filled bathtub, as reported by Okagbue and Ochulor (2007) and Mallo and Akuboh (2012). Samples for the unconsolidated undrained triaxial compression tests were prepared and tested in accordance with ASTM 2850 (2000). An oven-dried CSW samples passing 425 mm sieve were used to perform free swell index (FSI) tests according to ASTM D5890-02 and Phanikumar and Ramanjaneya Raju (2020). The reference liquid used for all the tests was kerosene. The procedure involved taking two 20 g oven dried soil samples passing through 425 µm sieve. Each sample was placed separately in two 100 ml graduated cylinders, one containing distilled water and the other kerosene (non-polar liquid) filled to the 100 ml mark. The final volume of soil was read after 24 hours to calculate the free swell index. The Free swell index (FSI), according to Holtz and Gibbs (1956), is calculated using equation 2,

$$FSI\ (\%) = V/Vo \times 100 \quad (2)$$

Where
V= final volume (volume in water)
Vo= initial volume (volume in kerosene)
Free swell index test was performed in accordance with Indian Standard IS: 2720 while one dimensional consolidation test was carried out in accordance with BS 1377 (1975) standard. One dimensional consolidation test is one dimensional because with a metal ring confining the sample, no lateral soil or water movement takes place. The test was carried out for three load increments starting with a load that gave a stress of 100 kN/m^2 on the sample and then subsequently doubled after each compression stage. The applied stress for this study was 100, 200 and 400 kN/m^2. After the first loading, the beam support was wound down and at the same time a clock was started and the readings on the settlement measuring dial gauge was taken at intervals of 0.125, 0.25, 0.5, 1, 2, 4, 8, 15, 30 minutes and after 1, 2, 4, 8 and 24 hours. After 24 hours, decision was taken to apply the next loading. Readings were taken for all the loading stages. The bulk density before and after the test were taken. The coefficients of consolidation (CC) were determined by plotting settlement versus the square root of time for each of the samples. Statistical (descriptive and correlation) analyses using proprietary computer software (GenStat Discovery, edition 3) were used to investigate the spatial variations of the geotechnical data, as well as to evaluate the degrees of relationships that exist among the tested soil parameters.

3.0. Results and Analysis

3.1. Particle size distribution and Atterberg limits

The results of the particle size distribution tests conducted on the CSW samples as presented in Table 1 revealed that the sand size fractions ranged from 46 to 60% while percentage passing No. 200 BS sieve (fine fraction) ranged from 30 to 42%. Oyediran and Williams (2010) expect soils with amounts of fines less than 35 % to possess better engineering properties when compared with those with amounts of fines greater than 35 % which are expected to pose field compaction problems when used either as sub-base or sub-grade materials. According to Unified Soil Classification System (USCS) and AASHTO classification system, majority of the tested samples showed values < 35% finer passing and are categorized under A-1 and A-2; an indication of an excellent and good materials for highway construction. Laboratory analyses results indicating the percentage of fines range from 29% - 42%, have CSW samples (CSW3, CSW4, CSW5, CSW7, CSW8, CSW9, CSW10 and CSW13) with values within the range of a maximum of 35.0% recommended by Federal Ministry of Works and Housing (FMWH, 1997) for a foundation material; an indication that most of the tested CSW samples would be rated as fair to good sub-grade foundation material and may not pose danger to the stability of the road structure.
Table 1: Results of index properties and specific gravity of sandstone quarry wastes

	Particle size distribution			Atterberg limits				Natural moisture content (%)	Specific gravity
Sample ID	Gravel (%) (> 4.75 mm)	Sand (%) (4.75- 0.075 mm)	Fines (%) (< 0.075 mm)	LL %	PL %	PI %	SL %	W_n	SG
CSW1	12	46	42	32	21	11	12	9.7	2.59
CSW2	16	51	37	37	24	13	14	12.0	2.49
CSW3	10	56	34	33	23	10	10	8.9	2.61
CSW4	10	60	30	33	22	11	12	10.7	2.60
CSW5	15	56	29	33	25	8	14	5.9	2.64
CSW6	18	48	36	38	25	13	16	10.0	2.59
CSW7	18	48	34	32	26	6	10	8.5	2.60
CSW8	12	56	32	32	21	11	12	10.1	2.59
CSW9	12	54	34	37	24	13	14	12.0	2.56
CSW10	14	53	33	33	23	10	12	8.9	2.58
CSW11	10	50	40	33	22	11	16	9.02	2.62
CSW12	12	48	38	33	25	8	14	11.2	2.54
CSW13	10	58	32	38	25	13	16	9.77	2.59
CSW14	12	48	40	32	24	8	12	10.4	2.59

The results of the Atterberg limit tests carried out on CSW materials as presented in Table 1 indicate that the liquid limit (LL) ranges from 32 to 38% while the plasticity index (PI) ranges from 6 to 13%. According to Mallo and Akuboh (2012), Atterberg limits tests are usually carried out to establish and describe the consistency thereby providing useful information regarding their strength, behavior and stability. However, according to the guideline of Nigerian Federal Ministry of Works and Housing (FMWH, 1997), the LL should not exceed 35% for the materials to be considered suitable for use as sub-base or subgrade materials. The BS (1975) specified that materials with LL ranging from 40 to 60% are typical of clay soils and they exhibit high plasticity and compressibility while those ranging from 25 to 50% are typical silty soils and they exhibit low to medium plasticity. According to Holtz & Gibbs (1956) and Ikeagwuani & Nwonu (2021), any soil that possesses a PI that exceeds 17% can be regarded as an extremely high plastic soil. A highly plastic soil usually has the ability to retain appreciable amount of total moisture in the diffuse double layer, especially by means of absorption (Aghamelu et al., 2011). According to the Unified Soil Classification System, CSW samples are inorganic clays of intermediate plasticity and intermediate compressible inorganic silt and organic clayey soils. The FMWH (1997) recommended PI of 13% maximum for sub-base and base materials. Soils with intermediate plasticity index (0–20%), according to Adiat et al. (2017) would make better engineering properties and thus all tested samples from the study area would make good engineering materials. However, all the tested CSW materials fall within the low plasticity thereby satisfying the standard recommendations of FMWH (1997) for materials suitable for use as base and sub-base course materials. The low PI values imply that there would be no or little tendency of the CSW materials to swell when they absorb water and this may not likely to pose a serious threat to the performance of the roads.

The linear shrinkage (SL) for the tested CSW materials ranged from 10 to 16%. The degree of expansion classification based on SL show that less than 5%, 5% to 8% and greater than 8% indicate non- critical, marginal and critical degree of expansion respectively. Soils with SL values > 8%, are considered active and likely to have critical swelling potentials and are not good foundation materials. The FMWH (1997) recommended SL of 8% maximum for various highway construction materials. Considering Nigerian General Specifications for Roads and Bridge Materials specifications, all the studies samples from Amasiri quarries have unsatisfactory SL values and would be rated as poor and critical in its degree of expansion and may be considered unsuitable for use as subbase and base materials unless improved.

3.2 Natural moisture content and specific gravity

The results of the natural moisture content (Wn) of the quarry waste from Amasiri quarries as presented in Table 1 ranged from 4.9 - 12.0% with an average value of 9.7%. Natural moisture content of any material is a measure of the water-holding ability of the material, usually reflecting clay content and type of the material (Aghamelu et al., 2011). The little variations in the natural moisture content can be attributed to the varying soil texture, and rainfall intensity. Natural moisture content influences the strength behavior of the soils and has the tendency to increase with increasing rainfall. Soil samples with high moisture content are expected to greatly reduce the shear strengths of soil. These may likely cause incessant failures of the overlying and would probably not suitable for road construction. The subgrade, according to Bell (2007) refers to the soil immediately beneath the sub-base and much they carry the load of the road structure plus that of the traffic. The strength of the subgrade materials, however, does not remain the same throughout its life. Changes in its strength are brought about by changes in its moisture content, by repeated wheel loading, and in some parts of the world by seasonal changes as in the tropical region. Soils with natural moisture content of 5%-15% are suitable for most engineering projects while those soils with Wn values ranging from 20 to 35% are considered unfavorable as engineering materials. The Wn values obtained for the tested CSW samples range from 4.9 - 12.0% are low and according to Underwood recommendation, are suitable for various engineering road projects.

The results of the specific gravity (SG) tests are also presented in Table 1. From the table, the SG ranges from 2.49 to 2.64. The SG of materials used as construction materials appears to be a very useful parameter in relating these materials to their degree of weathering (Oyediran and Williams, 2010). Specific gravity is known to correlate with mechanical strength of aggregates and may be used as a basis for selecting suitable highway pavement construction (Owoyemi & Adeyemi, 2012; Daramola et al., 2018). Good materials used in highway construction, according to Bell (2007) should have SG ranging from 2.5 to 2.75. British standard recommended that the SG should range from 2.60 to 2.80. The results obtained from the soils are indicative of a high degree of laterization of the soil. On the basis of the SG, the tested samples with an average value of 2.59 may perform marginally as they fairly satisfy the BS standard range between 2.60 and 2.80.

3.3 Compaction characteristics and California bearing ratio

The results of the compaction test as presented in Table 2 indicate that the maximum dry density (MDD) values ranged from 1.78 to 1.98 Mg/m^3 with an average value of 1.80 Mg/m3 and the corresponding optimum moisture content (OMC) values ranged from 9.0 to 20% with an average value of 12.4%. Soil samples with high MDD value with corresponding low OMC values are best suitable for use as sub-base and sub-grade materials. However, the recommended MDD and OMC standard, however, depends on their specific use. For instance, the Nigerian General Specification (FMWH, 1997) recommends that for materials to be used as fills, they should possess MDD > 0.047 Mg/m3 and OMC < 18 %. The Nigerian Specification for Road and Bridge Materials further specified OMC < 18% for both sub-base and subgrade materials. It was only sample CSW 6 that has OMC value > 18%. The increase in the OMC was due to finer particles, depending on their contents which increased the total particle surface area of the samples.

Table 2: Results of density and strength properties conducted on the sandstone quarry wastes

	Compaction characteristics		**California bearing ratio**		**Strength properties**		**Free swell index %**	**Coefficient of consolidation $M^2/yr(10^{-2})$**
Sample ID	MDD Mg/m^3	OMC %	Un-soaked CBR (%)	Soaked CBR (%)	Internal friction (ϕ)°	Cohesion (KN/m^2)	FSI	CC
CSW1	1.82	12	81.0	54.4	15	138	17.4	3.21
CSW2	1.78	18	51.2	33.4	15	130	28.0	4.51
CSW3	1.95	12	87.5	62.5	20	146	17.0	1.72
CSW4	1.92	14	82.0	57.4	16	139	20.8	4.81
CSW5	1.85	10	84.5	52.5	16	136	30.8	1.82
CSW6	1.88	20	60.2	35.4	22	140	16.7	5.61
CSW7	1.78	10	60.3	39.3	20	132	12.0	1.82
CSW8	1.89	12	81.0	56.4	15	120	17.4	3.61
CSW9	1.90	14	51.2	33.4	15	134	27.0	4.51
CSW10	1.79	10	87.2	62.0	25	142	17.0	2.62
CSW11	1.98	11	82.0	54.4	15	141	20.8	3.81
CSW12	1.87	12	84.5	52.5	18	138	28.8	1.72
CSW13	1.88	10	64.2	40.6	20	136	17.7	4.41
CSW14	1.79	12	60.3	39.3	15	132	12.0	1.82

The increase in finer particle content increased the OMC because the compaction parameters were controlled to a great extent by the index properties of soils (Adekalu and Osunbitan, 2001; Kalkan and Bayraktutan, 2008). Based on these specifications, the tested dust materials are considered suitable for use as both sub-base and subgrade materials and as filling and embankment materials as they satisfy the Nigerian General Specification for Road and Bridge Materials.

The CBR test is a penetration test used for the determination of the mechanical strength of the highway base materials. It is the ratio (expressed as percentage) of the actual load required to produced a 2.5 mm deflection to that required to produce the same deflection in a certain standard crushed stone. The results of CBR tests conducted on the CSW materials for unsoaked and soaked samples at zero curing as presented in Table 2 revealed that the CBR values ranged from 51.2 to 87.5% and 33.4 to 62.5%, for unsoaked and soaked samples, respectively. CBR value of any construction material is regarded as a benchmark in the assessment of its strength for pavement design (Mannering and Kilareski, 1998; Wignall et al., 1999). The recommended CBR standard, however, depends on specific use. For instance, FMWH (1997) recommended that for a material to be used as generally as fills it should possess soaked CBR values greater than 5%. Nigerian General Specifications for Roads and Bridge Materials further recommended that for unsoaked soils, the subgrade, subbase and base materials should be ≥ 10%, ≥ 30% and ≥ 80% respectively. Reports according to Nweke and Okogbue (2017) had it that if a sub-grade has a CBR value less than 10%, the sub-base material will deflect under traffic loadings in the same manner as the sub-grade and cause pavement deterioration. The Nigerian General Specifications for Roads and Bridge Materials recommended a minimum CBR of 8% for sub-grade/fill, a minimum CBR of 20% for sub-base type 2" material and the "subbase type 1" material shall have a minimum CBR of 30% after at least 24 hours soaking while Asphalt Institute (1962) recommended CBR value of less than 3% for sub-grade and 3 to 7% for sub-base. The test results showed that all tested samples have CBR values greater than 20% and are therefore suitable for use as subbase type 1 and sub-base type 2 materials in road construction and as subgrade materials for both soaked and unsoaked conditions. The reduction observed in CBR in soaked condition suggest that moisture influx would be detrimental to the quarry dust especially when used as subgrade materials. CBR has been correlated with pavement performance as well as used to establish design curves for pavement thickness (Mannering and Kilareski, 1998). Based on the local standards, the CSW materials met the FMWH (1997) recommendations for unsoaked materials, indicating that the subgrade, subbase and base materials should be ≥ 10%, ≥ 30% and ≥ 80% for CBR specifications.

3.4 Strength, swelling potentials and coefficient of consolidation

Shear strength of a material denotes the ability of such material to resist shearing deformational stresses. Strength parameters, cohesion and internal friction angle of the undisturbed soils tested at their natural moisture content were determined by unconsolidated undrained triaxial compression testing. The results of the strength tests of the CSW materials are also presented in Table 2. The internal friction (ϕ) ranged from 16 to 25º with an average value of 20º while the cohesion (*c*) ranged from 120 to 142 KN/m^2 with an average value of 136 KN/m^2. As shown in the table, the values of internal friction and cohesion decrease with corresponding increase in water content (or increase in the degree of saturation) compared with results of natural moisture content (see Tables 1). Apart from the fines content, the type of clay minerals and in situ bond strength are also important factors that affect the effective cohesion of the material. Punmia and Jain (2005) remarked that soil materials that are non cemented, most often, record very low values of internal friction (close to 10°). Reasonable values of *c* recorded denoted a cohesive material, similar to consolidated clay and other materials with high clay content. The fact that values of c were significantly greater than 10° and *c* reasonably high would suggest that the tested materials when used as subgrade and construction materials would have considerable strength, and are likely to withstand shear stresses. Generally, the cohesion and the angle of internal friction values recorded implying low plasticity, high permeability, shear strength and bearing capacity. The implication of the low value of the cohesion is that the stress needed to destroy the cohesion which binds the individual grains and give the rock its strength is small. The undrained cohesion values satisfy the requirement of greater than 103 kN/m^2 values specified for base course materials (FMWH, 1997), thus the materials are considered suitable as base course materials for road pavement materials.

The results of the free swell test as presented in Table 2 ranged from 12 to 30.8% with an average value of 20.2%. Soil with high PI according to Mitchell (1993), are expected to exhibit higher swelling potential. This is due to the fact that more plastic the material the higher the swell potential as they likely have higher percentage of clays. Soils with PI exceeding 35%, according to Bell (2007), are expected to display excessive shrinkage and settlement. The dual problem of swelling and shrinking has caused damage to many lightly loaded civil engineering structures. In this study, the free swell index recorded for the CSW materials are considered low which imply low plasticity materials (PI generally < 20%, see Table 1). The coefficient of consolidation (CC) tests displayed by the tested CSW samples as presented in Table 2 ranged from 1.72×10^{-2} m^2/yr to 5.61×10^{-2} m^2/yr with mean value of 3.36 $\times 10^{-2}$ m^2/yr and reveal non critical compression characteristics. The lesser the value of coefficient of consolidation obtained in a soil, the lesser the grain size hence the higher permeability. The CC relates to how long it will take for an amount of consolidation to take place. The CC values are governed by two factors; the amount of water squeezed out, and the rate at which that water can flow out. Soil samples compacted on the dry side OMC showed high CC for all the pressure increments. This can be attributed to the fact that soil compacted at dry of OMC have high void ratio and permeability. This implies that structures founded on the soils may undergo differential settlement of small magnitude, which varies unpredictably. However, on the basis of coefficient of consolidation, the soils from the area under study are adjudged suitable for sub grade, sub base and base course materials as they satisfy all requirements specified by Nigerian General Specifications.

3.5 Statistical analyses

Statistical analyses on the tested index properties indicate strong degree of positive relationships, expressed in terms of correlation coefficient at ($P<0.01$) and at the 95% confidence level (see Table 3) between strength and unsoaked CBR($r= 0.970$, $p<0.001$), percent fines and PI ($r= 0.869$, $p<0.001$), percent fines and LL ($r= 0.865$, $p<0.001$) and percent fines and OMC ($r= 0.717$, $p<0.001$). It was established that there existed a relationship between CBR and index properties and a strong correlation exist between them. Also, negative correlation exists between gravel and fines ($r= -0.978$, $p<0.001$) of the tested samples. Good correlations exist between linear shrinkage and percentage fines. Also plots of c against SG gave poor (0.331, $p<0.001$) correlation, indicating predominance of probably lightweight minerals with weak cohesion (Table 2), while negative correlation exists between CBR and W_n (-0.387, $p<0.001$) which implies that moisture, although weakly, influenced the CBR results of the samples. The internal friction angle, on the other hand, has a poor correlation with SG. Plasticity and gradation data of the soils studied do not show any appreciable correlations. The concluded empirical correlations verified the influence of these parameters and gave a general overview of the engineering behaviors of the examined Amasiri sandstone quarry wastes from southeastern Nigeria.

Table 3: Statistical analyses of physical and geotechnical parameters of the sandstone quarry wastes

		Correlations																	
		W_n	P	Gs	Gavel	Sand	Fine	LL	PL	PI	SL	MDD	OMC	Un- CBR	S-	CC	FSI	(Ø)0	C
W_n	Pearson Correlation	1																	
	P-value																		
P	Pearson Correlation	.260	1																
	P-value	.369																	
Gs	Pearson Correlation	-.572*	-.217	1															
	P-value	.032	.455																
Gavel	Pearson Correlation	-.601*	-.248	.328	1														
	P-value	.023	.393	.253															
Sand	Pearson Correlation	.455	.266	.236	-.610*	1													
	P-value	.102	.357	.416	.020														
Fine	Pearson Correlation	.564*	.212	-.435	-.978**	.432	1												
	P-value	.036	.467	.120	.000	.123													
LL	Pearson Correlation	.307	.285	-.414	-.794**	.148	.865**	1											
	P-value	.285	.324	.141	.001	.613	.000												
PL	Pearson Correlation	-.153	.157	-.132	.013	-.262	.054	.408	1										
	P-value	.601	.592	.653	.964	.366	.855	.148											
PI	Pearson Correlation	.432	.187	-.340	-.843**	.343	.869**	.758**	-.287	1									
	P-value	.123	.523	.235	.000	.230	.000	.002	.320										
SL	Pearson Correlation	.473	.151	-.125	-.687**	.446	.665**	.675**	.209	.559*	1								
	P-value	.088	.605	.670	.007	.110	.009	.008	.473	.038									
MDD	Pearson Correlation	-.009	.456	.392	-.211	.472	.116	.081	-.338	.327	.291	1							
	P-value	.976	.101	.165	.470	.088	.694	.782	.237	.254	.313								
OMC	Pearson Correlation	.529	-.095	-.533*	-.668**	.169	.717**	.596*	.084	.565*	.418	-.093	1						
	P-value	.052	.746	.050	.009	.565	.004	.024	.775	.035	.137	.752							
Un- CBR	Pearson Correlation	-.457	-.194	.489	.579*	-.118	-.629*	-.633*	-.482	-.320	-.245	.365	-.556*	1					
	P-value	.100	.505	.076	.030	.688	.016	.015	.081	.265	.399	.199	.039						
S-	Pearson Correlation	-.443	-.151	.448	.592*	-.144	-.636*	-.661*	-.596*	-.267	-.383	.353	-.576*	.970**	1				
	P-value	.112	.606	.108	.026	.624	.014	.010	.024	.357	.176	.216	.031	.000					
CC	Pearson Correlation	.570*	.170	-.252	-.916**	.560*	.896**	.713**	-.180	.876**	.577*	.274	.720**	-.417	-.387	1			
	P-value	.033	.561	.384	.000	.037	.000	.004	.537	.000	.031	.344	.004	.137	.171				
FSI	Pearson Correlation	.156	.334	-.337	-.192	-.148	.258	.267	.140	.180	.423	.159	.194	.048	-.053	.089	1		
	P-value	.594	.243	.238	.510	.614	.373	.356	.632	.539	.132	.588	.506	.871	.856	.762			
(Ø)0	Pearson Correlation	-.387	-.141	.095	.249	-.406	-.177	.181	.367	-.072	-.073	-.161	-.111	.148	.149	-.079	-.352	1	
	P-value	.171	.630	.748	.391	.150	.546	.536	.197	.806	.804	.584	.707	.615	.612	.788	.217		
C	Pearson Correlation	-.327	.154	.281	.227	-.129	-.225	.052	.061	.011	.072	.340	-.157	.386	.355	-.086	.002	.475	1
	P-value	.254	.598	.331	.435	.661	.440	.861	.837	.971	.808	.234	.593	.173	.212	.769	.994	.086	

*. Correlation is significant at the 0.05 level (2-tailed). Correlation Rating: > 0.91 = very Strong; 0.90 – 0.81 = Strong; 0.80 – 0.31 = moderate; < 0.30 = weak

**. Correlation is significant at the 0.01 level (2-tailed).

3.6 Suitability of Amasiri crushed sandstone wastes as road materials

The results of particle size distribution and Atterberg limits tests indicate that the tested CSW materials are predominated mainly by sand and fine size particles with intermediate plasticity. The sand and fine size proportions of the CSW materials may likely be responsible for stability that would be achieved when the waste materials are considered for use as flexible pavement materials. The recommended MDD, OMC and CBR standard, however, depends on specific use. For instance, the Nigerian General Specification (1997) for Road and Bridge Materials recommends that for a material to be used as generally as fills it should possess MDD > 0.047 Mg/m^3, OMC < 18% and soaked CBR values > 5%. With these c and ϕ values, the materials would likely have high bearing capacity and would be good foundation materials for heavy structures. They would also have fairly high slope stabilities and will therefore be useful in the construction of embankments. Comparison of the sandstone quarry dusts with charnockite quarry dust from Cochin, India and indurated shale quarry dust from Abakaliki indicate materials with good engineering properties as they showed many similarities in the geotechnical properties (See Table 4).

Table 4: Comparison of the sandstone quarry wastes with charnockite quarry dust from Cochin, India and indurated shale quarry dust from Abakaliki

Properties	**Charnockite quarry dust from Cochin, India**[a]	**Shale quarry dust from Abakaliki**[b]	**Quarry waste from Amasiri sandstone**[c]	
	Ave.	Ave.	Range	Ave.
Particle size distribution				
Gravel % (> 4.75 mm)	8	30	10- 18	12.9
Sand % (4.75 – 0.075 mm)	34	56	46- 60	52.2
Fines % (< 0.075 mm)	58	14	30- 42	35.1
Specific gravity	2.8	2.62		
Compaction characteristics				
MDD (Mg/m^3)	1.91	1.79	1.78 - 1.98	1.86
OMC (%)	12.6	16	9 – 20	12.4
California bearing ratio				
Unsoaked CBR at modified AASHTO and OMC (%)	24.8	10	51.2 - 87.5	72.6
CBR (24 hrs soaked) BS (%)	23.2	12	33.4 - 62.5	48.1

[a] Adapted from Soosan et al. (2005) ; [b]Nweke and Okogbue (2017); [c]This study

Based on the strength requirements for materials used in pavement structural layers, the sandstone quarry dust materials can qualify for high-volume civil engineering applications especially as sub-base and subgrade materials, as long as they are well designed and compacted to enough degrees having satisfied Nigeria General Specifications for Road and Bridge Materials (see Table 5). The materials indicate low swelling capacity on moisture influx, high compressibility, high bearing capacity and high permeability (i.e., high drainage). These properties, most probably, would affect the waste materials positively, like high MDD and relatively high shear strength parameters, recorded by the materials and ultimately render it suitable for use for most construction purposes, even when moisture influx cannot be effectively controlled.

Table 5: Comparison of the sandstone quarry wastes from Amasiri quarries with Nigerian General Specifications for road and bridge materials.

Properties of pavement materials	**Nigerian General Specifications (FMWH 1997)**	**Waste from Amasiri sandstone quarries**[b]		**Remarks**
General filling and embankment		Range value	Average value	
Liquid limit %	< 40	32 – 38	34	The CSW will
Plasticity index %	< 20	6 – 13	10	perform satisfactory
MDD (Mg/m^3)	> 0.047	1.78 - 1.98	1.86	as general filling
OMC (%)	< 18	9 – 20	12.4	and embankment
% passing No. 200 (%)	< 35	30- 42	35.1	materials
CBR (24 hrs soaked) BS (%)		33.4 - 62.5	48.1	
Liquid limit %	< 35	32 – 38	34	The CSW will
Plasticity index %	< 16	6- 13	10	perform satisfactory
CBR (24 hrs soaked) at West Africa Standard and OMC (%)	≥ 25	33.4 - 62.5	48.1	Sub-base course materials
Liquid limit %	≤ 30	32 – 38	34	The CSW
Plasticity index %	≤ 13	6 – 13	10	performance will be
% Passing sieve No. 200 (%)	5-15	30 – 42	35.1	marginal as base
Unsoaked CBR at Modified AASHTO and OMC (%)	≥ 80	51.2 - 87.5	72.6	course materials but need stabilization
Unconfined Compressive Strength (kN/m^2)	> 103	-	-	
Subgrade				The CSW
LL	< 35	32 – 38	34	performance will be
PI	≤ 13	6- 13	10	satisfactory as
MDD (Mg/m^3)	> 0.047	1.78 - 1.98	1.86	subgrade
OMC (%)	< 18	9 – 20	12.4	
% passing sieve No. 200 (%)	< 35	30 – 42	35.1	

([a] Adapted from Okagbue and Ochulor, 2007)

Moisture influx, however, would likely deteriorate their constituent minerals, especially iron minerals, resulting in strength reduction and perhaps, bearing capacity loss, during the service life of such projects. The coefficient of consolidation further implies that any structures founded on the soils may undergo differential settlement of small magnitude, which varies unpredictably. Based on Nigerian General Specifications for Roads and Bridge Materials specifications, improvements on the quality of waste dust maybe required for their excellent use in The quarry waste material evaluated in this study seems to be more appropriate to low-volume roads in pavement constructions, hence, stabilization techniques are highly recommended.

4.0 Conclusion

Laboratory investigation of the performance of sandstone quarry wastes from Amasiri region as road materials enabled the following conclusions to be drawn;

1. Geotechnical investigations of the sandstone quarry dust showed materials with low plasticity and low proportions of fines. The materials indicate low swelling capacity on moisture influx, high bearing capacity with intermediate permeability.
2. The high MDD, low OMC and relatively high shear strength parameters recorded by the materials ultimately proved the materials suitable for use in road project. The cohesion values satisfy the requirement of greater than 103 kN/m^2 values specified for base course materials and are thus considered suitable as base course materials for road pavements.
3. Based on the strength requirements for materials used in pavement structural layers, the sandstone quarry waste materials can qualify for high-volume civil engineering applications as long as they are well designed and compacted to enough degrees having satisfied most of Nigeria General Specifications for Road and Bridge Materials.
4. This concept of replacement of natural fine aggregate with quarry dust as highlighted in the investigation will greatly improve the utilization of this generated quarry dusts from the region, earlier regarded as wastes. Thus, this effort will help to conserve the scarcely available natural sand for sustainable development.
5. The studied quarry waste material can be used as construction material for flexible pavements, replacing natural materials traditionally used.

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