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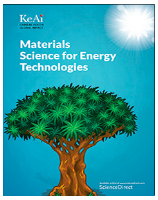

Scheffe optimization of swelling, California bearing ratio, compressive strength, and durability potentials of quarry dust stabilized soft clay soil

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ABSTRACT

Scheffe's second degree polynomial was used to formulate models for predicting the swelling potential, California bearing ratio, unconfined compressive strength and loss of strength on immersion durability of quarry dust treated soil. These models could predict the swelling potential, California bearing ratio, unconfined compressive strength and loss of strength on immersion durability of treated soil if the mix ratios are known and vice versa. The response predicted by the models are in good agreement with the corresponding experimentally observed results. The result of these tests shows the feasibility of using quarry dust in soil stabilization. The student *t*-test and the analysis of variance (ANOVA) test were used to check the adequacy of the models, and the models were found to be adequate at 95% confidence level. With the optimized equations, the properties' design, behaviour, and performance of treated soft clay soil as a pavement subgrade material will be appropriated and monitored. This will be for any possible volume changes, shear failures, strength failures and durability failures when the material used as a hydraulically bound material is in contact with moisture beyond its optimum and subjected to dynamic load beyond its design value.

1. Introduction

The method of optimization that will be used for the investigation is scheffe's simplex Lattice method for mixtures, where the property studied depends on the component ratios only. Firstly, a simplex is defined as a convex polyhedron with (k + 1) vertices produced by k intersecting hyper planes in k-dimensional space [1]. Any co-ordinate system above 3-dimensions are referred to as hyper planes, such planes are not orthogonal. A 2-dimensional regular simplex is, therefore, an equilateral triangle, while a 3-dimensional regular simplex is a regular tetrahedron. Scheffe [2,3] used a regular (q – 1) – simplex to represent a factor space needed to describe a response surface for mixtures consisting of several components. If the number of components is denoted by q, then for binary system (q = 2) the required simplex is a straight line; for q = 3, the required simplex is an equilateral triangle; and for q = 4, the simplex is a regular tetrahedron. The response surface for such a multicomponent system is normally described by means of a high degree polynomial, of the type of Eq. (1), having number of coefficients given by n where n is the degree of the polynomial [1].

$$\hat{y} = b_0 + \sum_{1\le i\le q} b_i x_i + \sum_{1\le i<j\le q} b_{ij} x_i x_j + \sum_{1\le i<j<k\le q} b_{ij} x_i x_j x_k + \sum b i_1 i_2 \ldots i_n x i_1 x i_2 \ldots x i_n \qquad (1)$$

2. Methodology

2.1. Mathematical modelling and formulation

2.1.1. Background

Modeling means setting up mathematical models/formulations of physical or other systems. Many factors of different effects occur in nature in the world simultaneously dependently or independently. When they interplay they could inter-affect one another differently at equal, direct, combined or partially combined rates

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Notations

q	number of components
k	degree of dimensional space
Xi	proportion of ith components of mixtures
M	degree of the Scheffe polynomial
X1	fraction of water
X2	fraction of quarry dust
X3	fraction of test soil
n	degree of polynomial regression
Z	actual components
X	pseudo components
Y_1, Y_2, Y_3, Y_{12}, Y_{13}, Y_{23}	responses from treatment mixture proportions
C_1, C_2, C_3, C_{12}, C_{13}, C_{23}	responses from control mixture proportions
NMC	natural moisture content
LL	liquid limit
PL	plastic limit
PI	plasticity index
SL	shrinkage limit
FSI	free swell index
Gs	specific gravity
UCS	unconfined compressive strength
MDD	maximum dry density
OMC	optimum moisture content
CBR	California bearing ratio
GP	poorly graded
CH	high clay content
AASHTO	American Administration for State Highway Officials
β_1, β_2, β_3, β_{12}, β_{13}, β_{23}	model coefficients
Y_{swl}	optimized swelling potential of the treated soft clay soil
Y_{cbr}	optimized California bearing ratio of the treated soft clay soil
Y_{ucs}	optimized unconfined compressive strength of the treated soft clay soil
Y_{dbl}	optimized durability of the treated soft clay soil

variationally, to generate varied natural constants in the form of coefficients and/or exponents. The challenging problem is to understand and asses these distinctive constants by which the interplaying factors underscore some unique natural phenomenon towards which their natures tend, in a single, double or multiphase system [4]. For such assessment a model could be constructed for a proper observation of response from the interaction of the factors through controlled experimentation followed by schematic design where such simplex lattice approach of the type of Henry Scheffe optimization theory could be employed. Also entirely different physical systems may correspond to the same mathematical model so that they can be solved by the same methods. This is an impressive demonstration of the unifying power of mathematics [4].

2.1.2. Factor space in simplex design

Simplex is the structural representational shape of a line or planes joining assumed positions of constituent materials (atoms) of a mixture [4]. Scheffe [2,3] considered experiments with mixtures of which the property studied depends on the proportions of the components but not their quantities in the mixture. A simplex is defined as a convex polyhedron with (k + 1) vertices produced by k intersecting hyper planes in k-dimensional space [1]. Any co-ordinate system above 3-dimensions are referred to as hyper planes, such planes are not orthogonal. A 2-dimensional regular simplex is, therefore, an equilateral triangle, while a 3-dimensional regular simplex is a regular tetrahedron. Scheffe [2,3] used a regular (q – 1) – simplex to represent a factor space needed to describe a response surface for mixtures consisting of several components. If the number of components is denoted by q, then for binary system (q = 2) the required simplex is a straight line; for q = 3, the required simplex is an equilateral triangle; and for q = 4, the simplex is a regular tetrahedron.

2.1.3. Scheffe's factor space

Strength and behavior of treated soft clay soils depend on the adequate proportioning of its ingredients or test materials. Scheffe [2,3] developed an optimization theory that is used to optimize the behavior of treated soft soils. This considered experiments with mixtures or blends of which the property studied depends on the proportions or percentages by weight of the components but not their quantities in the mixture. He introduced polynomial regression to model the response, called "q, n-polynomials". These polynomials have to be of low degree (n), otherwise the polynomial contains a large number of coefficients, making interpretation difficult and requiring a large number of design points.

$$if\ n = 1 : f(x) \sum_{i=1}^{q} \beta_i x_i \tag{2}$$

$$if\ n = 2 : f(x) \sum_{i=1}^{q} \beta_i x_i + \sum_{1 \le i \le j \le q}^{q} \beta_i x_i x_j \tag{3}$$

$$if\ n = 3 : f(x) \sum_{i=1}^{q} \beta_i x_i + \sum_{1 \le i \le j \le q}^{q} \beta_i x_i x_j + \sum_{1 \le i \le j \le k \le q}^{q} (\beta_{iij} x_i^2 x_j + \beta_{iij} x_i x_j x_k) \tag{4}$$

2.1.4. Interaction of compounds in Scheffe's factor space

Mixture designs are unique response surface designs where each of the components is bounded in values 0 and 1, [2–10]. Yet the sum of the components must be equal to one. If there are q components in a mixture, only q – l components are needed to determine the value of the last component. These constraints give rise to a (q – l) dimensional design space called a simplex. That is, the dimension of the design space is always one less than the number of components [2–14]. Thus the factor space is a regular (q – 1) dimensional simplex. In (q – 1) dimensional simplex if q = 2, we have 2 points of connectivity. This gives a straight line simplex lattice. If q = 3, we have a triangular simplex lattice and for q = 4, it is however, a tetrahedron simplex lattice, etc. Taking a whole factor space in the design we have a (q, m) simplex lattice whose properties are defined as follows:

i. The factor space has uniformly distributed points,
ii. Simplex lattice designs are saturated [1]

That is, the proportions used for each factor have m + 1 equally spaced levels from 0 to 1 (x_i = 0, 1/m, 2/m… 1), and all possible combinations are derived from such values of the component concentrations. This implies that all possible mixtures with these proportions are utilized. Hence, for the quadratic lattice (q, 2), approximating the response surface with the second degree polynomials (m = 2), the following levels of every factor must be used 0, ½ and 1.

2.1.4.1. Number of coefficients. P = 3, M = 2, $N = \frac{(p+m-1)!}{m!(p-1)!}$ and $N = \frac{(3+2-1)!}{2!(3-1)!} = N = \frac{4!}{2!2!} = 6$.

2.1.5. Three-component factor space

The first three pseudo components are located at the vertices of the tetrahedron simplex; A_1 [1:0:0], A_2 [0:1:0], A_3 [0:0:1]. Three other pseudo mix ratios located at mid points of the lines joining the vertices of the simplex are A_{12} [0.5:0.5:0], A_{13} [0.5:0:0.5], A_{23} [0:0.5:0.5] as presented in Fig. 1.

2.1.6. Responses

Responses are the selected properties of soil-additive blend or treatment matrix. A simplex lattice is described as a structural representation of lines joining the atoms of a mixture. The atoms are constituent components of the mixture. For the soil-additive blend mixture, the constituent elements are water, quarry dust and soil. And so it gives a simplex of a mixture of three components. Hence the simplex lattice of this three-component mixture is a three-dimensional solid equilateral triangle. Mixture components are subject to the constraint that the sum of all the components must be equal to one [2–10].

As a rule the response surfaces in multi-component systems are very intricate. To describe such surfaces adequately, high degree polynomials are required, and hence a great many experimental trials. A polynomial of degree n in q variable has C^n_{q+n} coefficients [4–16]. If a mixture has a total of q components and X_i be the proportion of the ith component in the mixture such that,

$$X_i \geq 0\,(i = 1,\,2,\,\ldots q) \tag{5}$$

Then the sum of the component proportion is a whole unity i.e

$$X_1 + X_2 + X_3 = 1 \quad or \sum X_i - 1 = 0 \tag{6}$$

$$\begin{aligned} n = {} & b_0 + \sum b_i X_i + \sum b_{ij} X_i X_j + \sum b_{ijk} X_i X_j X_k + \ldots. \\ & + \sum b_{i1,i2} \ldots. in\, X_j,\, X_{i2}, X_{jn} \end{aligned} \tag{7}$$

where,
$1 \leq i \leq q,\ 1 \leq j \leq q,\ 1 \leq i \leq j \leq q \leq k\, and\, 1 \leq i \leq j \leq k \leq l \leq \ldots. \leq in \leq q$ respectively [2 and 3].

$$\begin{aligned} Y = {} & b_0 + b_1 X_1 + b_2 X_2 + b_3 X_3 + b_{11} X_1^2 + b_{12} X_1 X_2 + b_{13} X_1 X_3 \\ & + b_{22} X_2^2 + b_{23} X_2 X_3 + b_{33} X_3^2 \end{aligned} \tag{8}$$

where, b is a constant coefficient.

The relationship obtainable from Eq. (8) is subjected to the normalization condition of Eq. (6) for a sum of independent variables [16–24]. For a ternary mixture, which was dealt with in this work, the reduced second degree polynomial can be obtained as follows:

From Eq. (3)

$$X_1 + X_2 + X_3 = 1 \tag{9}$$

$$i.e.\ \ b_0 X_1 + b_0 X_2 + b_0 X_3 = b_0 \tag{10}$$

$$b_0 = b_0(X_1 + X_2 + X_3)$$

Multiplying Eq. (9) by X_1, X_2, and X_3 in succession gives

$$\begin{aligned} X_1^2 &= X_1 - X_1X_2 - X_1X_3 \\ X_2^2 &= X_2 - X_1X_2 - X_2X_3 \\ X_3^2 &= X_3 - X_1X_3 - X_2X_3 \end{aligned} \tag{11}$$

Substituting Eq. (10) into Eq. (11), we obtain after necessary transformation that

$$\begin{aligned} \hat{Y} \ = {} & (b_0 + b_1 + b_{11})X_1 + (b_0 + b_2 + b_{22})X_2 + (b_0 + b_3 + b_{33})X_3 \\ & + (b_{12} + b_{11} + b_{22})X_1X_2 \\ & + (b_{13} + b_{11} + b_{33})X_1X_3 + (b_{23} + b_{22} + b_{33})X_2X_3 \end{aligned} \tag{12}$$

If we denote

$$\begin{aligned} & \beta_i = b_0 + b_i + b_{ii} \\ & And\ \beta_{ij} = b_{ij} - b_{ii} - b_{jj} \end{aligned} \tag{13}$$

Then we arrive at the reduced second degree polynomial:

$$\hat{Y} = \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_{12} X_1 X_2 + \beta_{13} X_1 X_3 + \beta_{23} X_2 X_3 \tag{14}$$

$$\begin{aligned} Y_1 = \beta_1,\ Y_2 = \beta_2,\ Y_3 = \beta_3,\ & \beta_{12} = 4Y_{12} - 2Y_1 - 2Y_2, \\ & \beta_{13} = 4Y_{13} - 2Y_1 - 2Y_3, \\ & \beta_{23} = 4Y_{23} - 2Y_2 - 2Y_3 \end{aligned} \tag{15}$$

2.1.7. Actual components and pseudo components

$$AZ = AX \tag{16}$$

Z represents the actual components while X represents the pseudo components, where A is the constant; a three by three matrix for the present work under study.

The value of matrix A will be obtained from the first three mix ratios. The mix ratios are;

$$Z_1[0.10 : 0.2 : 1.0],\ Z_2[0.15 : 0.5 : 1.0],\ Z_3[0.25 : 0.95 : 1.0] \tag{17}$$

The corresponding pseudo mix ratios are of an identity matrix form thus;

$$X_1[1 : 0 : 0],\ X_2[0 : 1 : 0],\ X_3[0 : 0 : 1] \tag{18}$$

Substitution of Xi and Zi into Eq. (15), then use the corresponding pseudo components to determine the corresponding actual mixture components.

X_1 = fraction of water ratio
X_2 = fraction of quarry dust
X_3 = fraction of soil

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} a_{11}a_{12}a_{13} \\ a_{21}a_{22}a_{23} \\ a_{31}a_{32}a_{33} \end{pmatrix} \begin{pmatrix} X_1 \\ X_2 \\ X_3 \end{pmatrix} \tag{19}$$

For the first run;

$$\begin{pmatrix} 0.1 \\ 0.2 \\ 1.0 \end{pmatrix} = \begin{pmatrix} a_{11}a_{12}a_{13} \\ a_{21}a_{22}a_{23} \\ a_{31}a_{32}a_{33} \end{pmatrix} \begin{pmatrix} 1 \\ 0 \\ 0 \end{pmatrix} \tag{20}$$

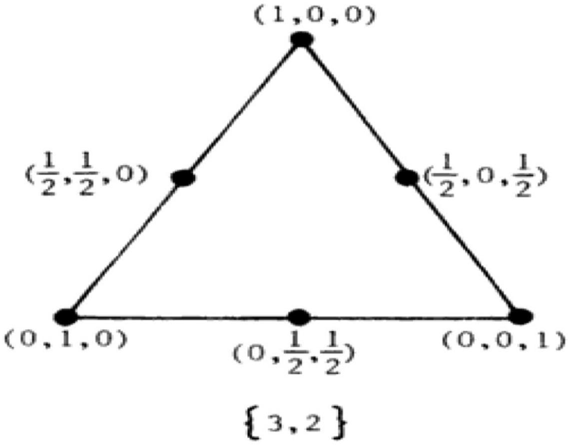

Fig. 1. Triangular simplex components.

$a_{11} = 0.1$, $a_{21} = 0.2$, $a_{31} = 1.0$,
For the second run;

$$\begin{pmatrix} 0.15 \\ 0.5 \\ 1.0 \end{pmatrix} = \begin{pmatrix} a_{11}a_{12}a_{13} \\ a_{21}a_{22}a_{23} \\ a_{31}a_{32}a_{33} \end{pmatrix} \begin{pmatrix} 0 \\ 1 \\ 0 \end{pmatrix} \tag{21}$$

$a_{12} = 0.15$, $a_{22} = 0.5$, $a_{32} = 1.0$,
And for the third run;

$$\begin{pmatrix} 0.25 \\ 0.95 \\ 1.0 \end{pmatrix} = \begin{pmatrix} a_{11}a_{12}a_{13} \\ a_{21}a_{22}a_{23} \\ a_{31}a_{32}a_{33} \end{pmatrix} \begin{pmatrix} 0 \\ 0 \\ 1 \end{pmatrix} \tag{22}$$

$a_{13} = 0.25$, $a_{23} = 0.95$, $a_{33} = 1.0$,
Substituting the values of the constants, we have;

$$\begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \tag{23}$$

Therefore, for A_{12};

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.5 \\ 0.5 \\ 0 \end{pmatrix} \tag{24}$$

$$\begin{gathered} Z_1 = (0.1 * 0.5) + (0.15 * 0.5) = 0.125; \\ Z_2 = (0.2 * 0.5) + (0.5 * 0.5) = 0.4; \\ Z_3 = (1.0 * 0.5) + (1.0 * 0.5) = 1.0 \end{gathered} \tag{25}$$

For A_{13};

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.5 \\ 0 \\ 0.5 \end{pmatrix} \tag{26}$$

$$\begin{gathered} Z_1 = (0.1 * 0.5) + (0.25 * 0.5) = 0.5;\ Z_2 = (0.2 * 0.5) \\ +(0.5 * 0.95) = 0.575; \\ Z_3 = (1.0 * 0.5) + (1.0 * 0.5) = 1.0 \end{gathered} \tag{27}$$

For A_{23};

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0 \\ 0.5 \\ 0.5 \end{pmatrix} \tag{28}$$

$$\begin{gathered} Z_1 = (0.15 * 0.5) + (0.25 * 0.5) = 0.5;\ Z_2 = (0.5 * 0.5) \\ +(0.5 * 0.95) = 0.775; \\ Z_3 = (1.0 * 0.5) + (1.0 * 0.5) = 1.0 \end{gathered} \tag{29}$$

Furthermore, the mixture proportion of control points showing actual and pseudo components are as follows;

At control points for A_1, it is observed that,

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.25 \\ 0.25 \\ 0.5 \end{pmatrix} \tag{30}$$

$$\begin{gathered} Z_1 = (0.1 * 0.25) + (0.15 * 0.25) + (0.25 * 0.5) = 0.1875; \\ Z_2 = (0.2 * 0.25) + (0.5 * 0.25) + (0.95 * 0.5) = 0.65; \\ Z_3 = (1.0 * 0.25) + (1.0 * 0.25) + (1.0 * 0.5) = 1.0 \end{gathered} \tag{31}$$

At control points for A_2, it is observed that,

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.25 \\ 0.5 \\ 0.25 \end{pmatrix} \tag{32}$$

$$\begin{gathered} Z_1 = (0.1 * 0.25) + (0.15 * 0.5) + (0.25 * 0.25) = 0.1625; \\ Z_2 = (0.2 * 0.25) + (0.5 * 0.5) + (0.95 * 0.25) = 0.5375; \\ Z_3 = (1.0 * 0.25) + (1.0 * 0.5) + (1.0 * 0.25) = 1.0 \end{gathered} \tag{33}$$

At control points for A_3, it is observed that;

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.5 \\ 0.25 \\ 0.25 \end{pmatrix} \tag{34}$$

$$\begin{gathered} Z_1 = (0.1 * 0.5) + (0.15 * 0.25) + (0.25 * 0.25) = 0.15; \\ Z_2 = (0.2 * 0.5) + (0.5 * 0.25) + (0.95 * 0.25) = 0.4625; \\ Z_3 = (1.0 * 0.5) + (1.0 * 0.25) + (1.0 * 0.25) = 1.0 \end{gathered} \tag{35}$$

At control points for A_{12}, it is observed that;

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.6 \\ 0.2 \\ 0.2 \end{pmatrix} \tag{36}$$

$$\begin{gathered} Z_1 = (0.1 * 0.6) + (0.15 * 0.2) + (0.25 * 0.2) = 0.14; \\ Z_2 = (0.2 * 0.6) + (0.5 * 0.2) + (0.95 * 0.2) = 0.41; \\ Z_3 = (1.0 * 0.6) + (1.0 * 0.2) + (1.0 * 0.2) = 1.0 \end{gathered} \tag{37}$$

At control points for A_{13}, it is observed that;

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.2 \\ 0.6 \\ 0.2 \end{pmatrix} \tag{38}$$

$$\begin{gathered} Z_1 = (0.1 * 0.2) + (0.15 * 0.6) + (0.25 * 0.2) = 0.16; \\ Z_2 = (0.2 * 0.2) + (0.5 * 0.6) + (0.95 * 0.2) = 0.53; \\ Z_3 = (1.0 * 0.2) + (1.0 * 0.6) + (1.0 * 0.2) = 1.0 \end{gathered} \tag{39}$$

At control points for A_{23}, it is observed that;

$$\begin{pmatrix} Z_1 \\ Z_2 \\ Z_3 \end{pmatrix} = \begin{pmatrix} 0.1 & 0.15 & 0.25 \\ 0.2 & 0.5 & 0.95 \\ 1.0 & 1.0 & 1.0 \end{pmatrix} \begin{pmatrix} 0.2 \\ 0.2 \\ 0.6 \end{pmatrix} \tag{40}$$

$$\begin{gathered} Z_1 = (0.1 * 0.2) + (0.15 * 0.2) + (0.25 * 0.6) = 0.2; \\ Z_2 = (0.2 * 0.2) + (0.5 * 0.2) + (0.95 * 0.6) = 0.71; \\ Z_3 = (1.0 * 0.2) + (1.0 * 0.2) + (1.0 * 0.6) = 1.0 \end{gathered} \tag{41}$$

2.2. Experimental procedure

2.2.1. Materials preparation

The soil sample was collected from Ohiya mechanic village borrow pit from a depth of 1 m. About 500 g of the disturbed sample was collected, sundried for a few days and stored in bags for use [17–42]. Quarry dust which was utilized as an admixture in the experimental program was collected from Crushed Rock quarry site, Lokpanta, Abia State, Nigeria. About 200 g of the quarry dust was collected, sundried and stored for experimentation [42–51]. The three test materials used in this programme were test soil, quarry dust (QD) and water Table 1.

2.2.2. Laboratory procedures

In the experimental program, first, the preliminary tests were conducted in accordance with the British standard requirement

Table 1
Matrix for Scheffe's (3, 2) – Lattice Polynomial.

Actual Components			Response	Pseudo Components		
Z_1	Z_2	Z_3		X_1	X_2	X_3
0.1	0.2	1	Y_1	1	0	0
0.15	0.6	1	Y_2	0	1	0
0.25	0.95	1	Y_3	0	0	1
0.125	0.4	1	Y_{12}	0.5	0.5	0
0.175	0.575	1	Y_{13}	0.5	0	0.5
0.2	0.775	1	Y_{23}	0	0.5	0.5

[52–56], to characterize the test soil and the quarry dust as well as determine the aluminosilicates content of the test materials for proper classification. This served as a control to monitor the behaviour of the treated soil under laboratory conditions to achieve results which were used to validate the modelling and optimization programs. The four parameters for which the utilization of quarry dust was optimized were; swelling potential, California bearing ratio, unconfined compressive strength and durability. In pavement constructions it is important to understudy the behaviour of soft clay soils which are exposed to hydraulically bound conditions [57–67]. The road pavements are prone to failures through the effect of moisture on the foundation materials (soft clay soil). Most cases, pavement encounter lateral deformation and create cracks, which eventually permit the migration of moisture into the foundation layer. In the event of this, there has been records of volume changes in the subgrade layer creating inconsistency in traffic load distribution from the layers above. Swelling potential test is conducted in this case to validate the swelling potential optimization exercise. The same is relevant in the strength properties of the treated soil and the durability index. Durability index experimental exercise was conducted by loss of strength on immersion into water method. That is, to determine the effect of moisture accumulation of the subgrade soil on the performance of the structure over a period of time. These test were conducted in accordance with British standard requirements [67]. Moreover, the sampling and proportioning of the test materials have been determined by a mix ratio model using the scheffe modelling mathematical method. The proportions of the test materials; quarry dust and water were gotten from iterations of the 3, 2 scheffe polynomial presented in Table 2. The values served as the percentage by weight of the dry solid added to the stabilization or treatment protocol. These exercises were conducted to validate the accuracy of the mathematical model. The specimens were prepared in that order and the swelling, California bearing ratio, unconfined compressive strength and durability tests also were experimented on under laboratory conditions.

3. Results and discussions

3.1. General behaviour and classification of test materials

Table 3
Basic properties of the Ohiya test soils.

Property Description of test Soils and Units	Values
% Passing Sieve No 200	4.6
NMC (%)	14
LL (%)	64
PL (%)	36
PI (%)	28
SL (%)	7
FSI (%)	275
G_s	2.12
AASHTO Classification	A-7
USCS	GP, CH
MDD (g/cm^3)	1.80
OMC (%)	13.13
CBR (%)	8
Color	Reddish Ash

The test soil preliminary results presented in Table 3 have shown that the soils was highly plastic soil, high swelling potential, an A-7 soil (according to AASHTO classification method), poorly graded and contains high level of clay content, and exhibits potentials of a soft and expansive soil. The strength properties also showed values below acceptable standards for materials to be used as foundation materials or geomaterials. By these properties, it is a

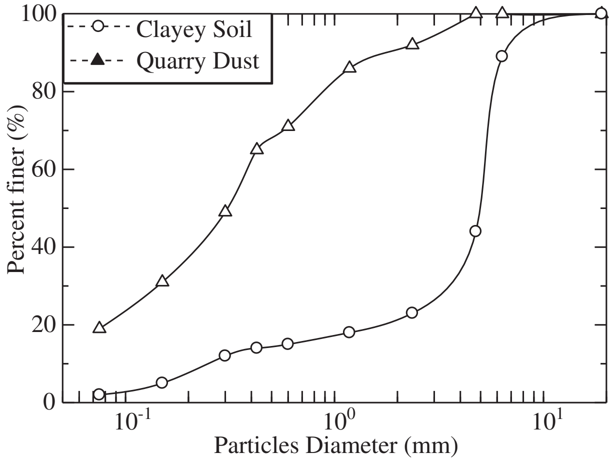

Fig. 2. Grain Size Distribution of Studied Materials.

Table 2
Mixture proportion of control points showing actual and pseudo components.

Actual Components			Response	Pseudo Components		
Z_1	Z_2	Z_3		X_1	X_2	X_3
0.1875	0.65	1	C_1	0.25	0.25	0.5
0.1625	0.5375	1	C_2	0.25	0.5	0.25
0.15	0.4625	1	C_3	0.5	0.25	0.25
0.14	0.41	1	C_{12}	0.6	0.2	0.2
0.16	0.53	1	C_{13}	0.2	0.6	0.2
0.2	0.71	1	C_{23}	0.2	0.2	0.6

problem soil and falls below the requirements to be used as a foundation material [52–67]. The quarry dust also showed a poorly graded behaviour according to Table 4 and Fig. 2. The chemical composition test conducted on the soil and quarry dust has shown the aluminosilicates contained in the test materials. This is responsible for the pozzolanic activity between the soil and the additive materials. The aluminosilicate content showed that the quarry dust has high pozzolanic potential as presented in Table 5, hence it is cementitious and served as a binder in the soil-additive matrix.

3.2. Experimental responses, model responses and model coefficients

3.2.1. Swelling potential experimental responses, model responses and model coefficients

Table 6 presents the responses of the experimental, modelling and coefficients on the swelling potential exercise.

3.2.1.1. Regression equation for swelling potential. From Eqs. (13) and (14), the coefficients of the Scheffe's second degree polynomial were determined as follows:

β_1	β_2	β_3	β_{12}	β_{13}	β_{23}
6.2	4.5	2.4	−2.2	1.2	0.2

Substituting the values of these coefficients into Eq. (13) yields

$$Y_{swl} = 6.2X_1 + 4.5X_2 + 2.4X_3 - 2.2X_1X_2 + 1.2X_1X_3 + 0.2X_2X_3 \tag{42}$$

3.2.2. California bearing ratio experimental responses, model responses and model coefficients

Table 7 presents the responses of the experimental, modelling and coefficients on the CBR exercise of the quarry dust treated soft clay soil.

3.2.2.1. Regression equation for California bearing ratio. From Eqs. (13) and (14), the coefficients of the Scheffe's second degree polynomial were determined as follows:

β_1	β_2	β_3	β_{12}	β_{13}	β_{23}
15	18	32	−2	−26	−28

Substituting the values of these coefficients into Eq. (13) yields

$$Y_{cbr} = 15X_1 + 18X_2 + 32X_3 - 2X_1X_2 - 26X_1X_3 - 28X_2X_3 \tag{43}$$

3.2.3. Unconfined compressive strength experimental responses, model responses and model coefficients

Table 8 presents the responses of the experimental, modelling and coefficients on the UCS exercise of the quarry dust treated soft clay soil.

3.2.3.1. Regression equation for unconfined compressive strength. From Eqs. (13) and (14), the coefficients of the Scheffe's second degree polynomial were determined as follows:

Table 4
Particle Size Distribution (PSD) of Test Materials.

Materials	% Passing Sieve (mm)										
	19	6.35	4.75	2.36	1.18	0.6	0.425	0.3	0.15	0.075	Pan
Clay Soil	–	–	100	92	86	71	65	49	31	19	0
Quarry dust	100	89	44	23	18	15	14	12	5	2	0

Table 5
Oxides Composition of the materials used in this paper.

Materials	Oxides Composition (content wt%)											
	SiO_2	Al_2O_3	CaO	Fe_2O_3	MgO	K_2O	Na_2O	TiO_2	LOI*	P_2O_5	SO_3	IR*	Free CaO
Clay Soil	77.75	16.65	1.42	3.22	0.07	0.89	0.02	–	–	–	–	–	–
QD*	63.48	17.72	5.56	1.77	4.65	2.76	0.01	3.17	0.88	–	–	–	–

* IR is Insoluble Residue, LOI is Loss on Ignition, QD: Quarry Dust.

Table 6
Swelling potential responses from 7 days soaked specimens from the experimental exercise and the model.

Symbol of Trails	Actual Components			Exp. Response (%)	Pseudo Components			Model Responses (%)	Model Coefficients	
	Z_1	Z_2	Z_3		X_1	X_2	X_3		Symbol	Outcome
Y_1	0.1	0.2	1	6.2	1	0	0	6.2	β_1	6.2
Y_2	0.15	0.6	1	4.5	0	1	0	4.5	β_2	4.5
Y_3	0.25	0.95	1	2.4	0	0	1	2.4	β_3	2.4
Y_{12}	0.125	0.4	1	4.8	0.5	0.5	0	4.8	β_{12}	−2.2
Y_{13}	0.175	0.575	1	4.6	0.5	0	0.5	4.6	β_{13}	1.2
Y_{23}	0.2	0.775	1	3.5	0	0.5	0.5	3.5	β_{23}	0.2
C_1	0.1875	0.65	1	4.2	0.25	0.25	0.5	4.2375	β_1	4.2
C_2	0.1625	0.5375	1	4.3	0.25	0.5	0.25	4.2375	β_2	4.3
C_3	0.15	0.4625	1	4.7	0.5	0.25	0.25	4.475	β_3	4.7
C_{12}	0.14	0.41	1	4.8	0.6	0.2	0.2	4.512	β_{12}	2.2
C_{13}	0.16	0.53	1	4.6	0.2	0.6	0.2	4.216	β_{13}	0.6
C_{23}	0.2	0.71	1	3.6	0.2	0.2	0.6	4.248	β_{23}	−3.6

β_1	β_2	β_3	β_{12}	β_{13}	β_{23}
90	135	258	46	−168	−46

Substituting the values of these coefficients into Eq. (13) yields

$$Y_{ucs} = 90X_1 + 135X_2 + 258X_3 + 46X_1X_2 - 168X_1X_3 - 46X_2X_3 \qquad (44)$$

3.2.4. Durability experimental responses, model responses and model coefficients

Table 9 presents the responses of the experimental, modelling and coefficients on the durability exercise of the quarry dust treated soil.

3.2.4.1. Regression equation for loss of strength on immersion durability potential. From Eqs. (13) and (14), the coefficients of the Scheffe's second degree polynomial were determined as follows:

β_1	β_2	β_3	β_{12}	β_{13}	β_{23}
81	88	98	81	−95	−80

Substituting the values of these coefficients into Eq. (13) yields

$$Y_{dbl} = 81X_1 + 88X_2 + 98X_3 + 81X_1X_2 - 95X_1X_3 - 80X_2X_3 \qquad (45)$$

Table 7
California bearing ratio responses from the experimental exercise and the model.

Symbol of Trails	Actual Components			Exp. Response (%)	Pseudo Components			Model Responses (%)	Model Coefficients	
	z_1	Z_2	Z_3		X_1	X_2	X_3		Symbol	Outcome
Y_1	0.1	0.2	1	15	1	0	0	15	β_1	15
Y_2	0.15	0.6	1	18	0	1	0	18	β_2	18
Y_3	0.25	0.95	1	32	0	0	1	32	β_3	32
Y_{12}	0.125	0.4	1	16	0.5	0.5	0	16	β_{12}	−2
Y_{13}	0.175	0.575	1	17	0.5	0	0.5	17	β_{13}	−26
Y_{23}	0.2	0.775	1	18	0	0.5	0.5	18	β_{23}	−28
C_1	0.1875	0.65	1	18	0.25	0.25	0.5	17.125	β_1	18
C_2	0.1625	0.5375	1	17	0.25	0.5	0.25	17	β_2	17
C_3	0.15	0.4625	1	16	0.5	0.25	0.25	16.875	β_3	16
C_{12}	0.14	0.41	1	16	0.6	0.2	0.2	16.92	β_{12}	−6
C_{13}	0.16	0.53	1	17	0.2	0.6	0.2	17	β_{13}	0
C_{23}	0.2	0.71	1	18	0.2	0.2	0.6	17.08	β_{23}	6

Table 8
Unconfined compressive strength responses from the experimental exercise of 28 days curing.

Symbol of Trails	Actual Components			Exp. Response (kN/m^2)	Pseudo Components			Model Responses (kN/m^2)	Model Coefficients	
	z_1	Z_2	Z_3		X_1	X_2	X_3		Symbol	Outcome
Y_1	0.1	0.2	1	90	1	0	0	90	β_1	90
Y_2	0.15	0.6	1	135	0	1	0	135	β_2	135
Y_3	0.25	0.95	1	258	0	0	1	258	β_3	258
Y_{12}	0.125	0.4	1	124	0.5	0.5	0	124	β_{12}	46
Y_{13}	0.175	0.575	1	132	0.5	0	0.5	132	β_{13}	−168
Y_{23}	0.2	0.775	1	185	0	0.5	0.5	185	β_{23}	−46
C_1	0.1875	0.65	1	155	0.25	0.25	0.5	151.625	β_1	155
C_2	0.1625	0.5375	1	130	0.25	0.5	0.25	150.875	β_2	130
C_3	0.15	0.4625	1	126	0.5	0.25	0.25	141	β_3	126
C_{12}	0.14	0.41	1	125	0.6	0.2	0.2	139.4	β_{12}	−70
C_{13}	0.16	0.53	1	130	0.2	0.6	0.2	150.04	β_{13}	−42
C_{23}	0.2	0.71	1	182	0.2	0.2	0.6	150.68	β_{23}	216

Table 9
Loss of strength on immersion durability potential responses from the experimental exercise.

Symbol of Trails	Actual Components			Exp. Response (%)	Pseudo Components			Model Responses (%)	Model Coefficients	
	z_1	Z_2	Z_3		X_1	X_2	X_3		Symbol	Outcome
Y_1	0.1	0.2	1	81	1	0	0	81	β_1	81
Y_2	0.15	0.6	1	88	0	1	0	88	β_2	88
Y_3	0.25	0.95	1	98	0	0	1	98	β_3	98
Y_{12}	0.125	0.4	1	83	0.5	0.5	0	83	β_{12}	81
Y_{13}	0.175	0.575	1	84	0.5	0	0.5	84	β_{13}	−95
Y_{23}	0.2	0.775	1	87	0	0.5	0.5	87	β_{23}	−80
C_1	0.1875	0.65	1	89	0.25	0.25	0.5	85.255	β_1	89
C_2	0.1625	0.5375	1	84	0.25	0.5	0.25	84.655	β_2	84
C_3	0.15	0.4625	1	83	0.5	0.25	0.25	85.75	β_3	83
C_{12}	0.14	0.41	1	83	0.6	0.2	0.2	80.655	β_{12}	−80
C_{13}	0.16	0.53	1	84	0.2	0.6	0.2	86.525	β_{13}	−72
C_{23}	0.2	0.71	1	86	0.2	0.2	0.6	88.555	β_{23}	98

3.3. Scheffe's model test for adequacy and validation

The test for adequacy of the model was done using Fischer test at 95% confidence level on the compressive strength at the control points i.e., C_1, C_2, C_3, C_{12}, C_{13} and C_{23}. In this test, two hypotheses were set as follows:

Null Hypothesis: There is no significant difference between the laboratory tests and model predicted swelling potential, California bearing ratio, compressive strength and durability results.

Alternative Hypothesis: There is a significant difference between the laboratory test and model predicted swelling potential, California bearing ratio, compressive strength and durability results.

3.3.1. Swelling potential

A two-tail test (inequality) was conducted and if t Stat <−t Critical two-tail or t Stat >t Critical two-tail, we reject the null hypothesis. From the result presented in Tables 10 and 11, t stat = 0.3 and t critical two-tail = 2.57 so t critical >t stat. Therefore, we accept the null hypothesis.

From the analysis of variance conducted on the experimental and laboratory results presented in Table 12, if F > F crit, we reject the null hypothesis. From the result, F = 0.0588 and F crit = 4.965 so F crit >F Therefore, we do not reject null hypothesis. Therefore, the difference between the experiment result and the model result was not significant. Hence, the model is adequate for use in predicting the probable Swelling potential of the quarry dust treated soil when the mix ratio is known and vice-versa.

Table 10
Swelling potential responses from the experimental exercise and the model.

Items	Response	
	Exp.	Model
C_1	4.2	4.2375
C_2	4.3	4.2375
C_3	4.7	4.475
C_{12}	4.8	4.512
C_{13}	4.6	4.216
C_{23}	3.6	4.248

Table 11
T-Test: Paired Two Sample for Means.

Descriptions	Responses	
	Exp.	Model
Mean	4.366667	4.321
Variance	0.194667	0.018099
Observations	6	6
Pearson Correlation	0.623512	
Hypothesized Mean Difference	0	
Df	5	
t Stat	0.300307	
P(T <= t) one-tail	0.388014	
t Critical one-tail	2.015048	
P(T <= t) two-tail	0.776028	
t Critical two-tail	2.570582	

Table 13
California bearing ratio responses from the experimental exercise and the model.

Items	Response	
	Exp.	Model
C_1	18	17.125
C_2	17	17
C_3	16	16.875
C_{12}	16	16.92
C_{13}	17	17
C_{23}	18	17.08

3.3.2. California bearing ratio

A two-tail test (inequality) is conducted and if t Stat < −t Critical two-tail or t Stat > t Critical two-tail, we reject the null hypothesis. From the result presented in Tables 13 and 14, t stat = 0 and t critical two-tail = 2.57 so t critical > t stat. Therefore, we accept the null hypothesis.

From the analysis of variance conducted on the experimental and laboratory results presented in Table 15, if F > F crit, we reject the null hypothesis. From the result, F = 0 and F crit = 4.965 so F crit > F Therefore, we do not reject null hypothesis. Therefore, the difference between the experiment result and the model result was not significant. Hence, the model is adequate for use in predicting the probable California bearing ratio of the quarry dust treated soil when the mix ratio is known and vice-versa.

Table 14
T-Test: Paired Two Sample for Means.

T-Test: Paired Two Sample for Means		
Items	Responses	
	Exp.	Model
Mean	17	17
Variance	0.8	0.00881
Observations	6	6
Pearson Correlation	0.976744	
Hypothesized Mean Difference	0	
df	5	
t Stat	0	
P(T <= t) one-tail	0.5	
t Critical one-tail	2.015048	
P(T <= t) two-tail	1	
t Critical two-tail	2.570582	

Table 12
Anova: Single Factor.

Anova: Single Factor						
Summary						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Exp. Response	6	26.2	4.366667	0.194667		
Model Response	6	25.926	4.321	0.018099		
ANOVA						
Source of Variation	*SS*	*Df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0.006256	1	0.006256	0.05881	0.813289	4.964603
Within Groups	1.063829	10	0.106383			
Total	1.070085	11				

Table 15
Anova: Single Factor.

Anova: Single Factor						
Summary						
Groups	*Count*	*Sum*	*Average*	*Variance*		
lab response	6	102	17	0.8		
model response	6	102	17	0.00881		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	0	1	0	0	1	4.964603
Within Groups	4.04405	10	0.404405			
Total	4.04405	11				

Table 16
Unconfined compressive strength responses from the experimental exercise.

Items	Response	
	Exp.	Model
C_1	155	151.625
C_2	130	150.875
C_3	126	141
C_{12}	125	139.4
C_{13}	130	150.04
C_{23}	182	150.68

3.3.3. *Unconfined compressive strength*

We do a two-tail test (inequality) and if t Stat < −t Critical two-tail or t Stat > t Critical two-tail, we reject the null hypothesis. From the result presented in Tables 16 and 17, t stat = −0.718 and t critical two-tail = 2.57 so t critical > t stat. Therefore, we accept the null hypothesis.

If F > F crit, we reject the null hypothesis of the analysis of variance. From the result presented in Table 18, F = 0.384 and F crit = 4.965 so F crit > F Therefore, we do not reject null hypothesis. Therefore, the difference between the experiment result and the model result was not significant. Hence, the model is adequate for use in predicting the unconfined compressive strength of the quarry dust treated soil when the mix ratio is known and vice-versa (Tables 19 and 20).

Table 17
T-Test: Paired Two Sample for Means.

T-Test: Paired Two Sample for Means		
Items	Responses	
	Exp.	Model
Mean	141.333	147.27
Variance	519.8667	30.50257
Observations	6	6
Pearson Correlation	0.556876	
Hypothesized Mean Difference	0	
df	5	
t Stat	−0.71806	
P(T <= t) one-tail	0.25243	
t Critical one-tail	2.015048	
P(T <= t) two-tail	0.50486	
t Critical two-tail	2.570582	

Table 19
Loss of strength on immersion durability potential responses.

Loss of strength on immersion durability potential responses		
	Exp. response	model response
C_1	89	84.125
C_2	84	84
C_3	83	84.125
C_{12}	83	84.56
C_{13}	84	84
C_{23}	86	84.08

Table 20
T-Test: Paired Two Sample for Means.

T-Test: Paired Two Sample for Means		
	Exp. response	model response
Mean	84.83333	84.14833
Variance	5.366667	0.043847
Observations	6	6
Pearson Correlation	−0.26456	
Hypothesized Mean Difference	0	
df	5	
t Stat	0.704827	
P(T <= t) one-tail	0.256193	
t Critical one-tail	2.015048	
P(T <= t) two-tail	0.512385	
t Critical two-tail	2.570582	

3.3.4. *Loss of strength on immersion durability potential*

We do a two-tail test (inequality) and if t Stat < −t Critical two-tail or t Stat > t Critical two-tail, we reject the null hypothesis. From the result presented in Tabs. 19 and 20, t stat = 0.7048 and t critical

Table 18
Anova: Single Factor.

Anova: Single Factor						
Summary						
Groups	*Count*	*Sum*	*Average*	*Variance*		
lab response	6	848	141.3333	519.8667		
model response	6	883.62	147.27	30.50257		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	105.732	1	105.732	0.384222	0.549215	4.964603
Within Groups	2751.846	10	275.1846			
Total	2857.578	11				

two-tail = 2.571 so t critical > t stat. Therefore, we accept the null hypothesis. Therefore, the difference between the experiment result and the model result was not significant. Hence, the model is adequate for use in predicting the probable durability of the quarry dust treated soil when the mix ratio is known and vice versa.

3.4. Discussion of results

Through the application of Scheffe's simplex model, the values of swelling potential, California bearing ratio, unconfined compressive strength and durability by loss of strength on immersion method of quarry dust treated soil were obtained. The model gave highest values of swelling potential of 6.2% corresponding to mix ratio of 0.1:0.2:1.0 for water, quarry dust, and soil respectively. The lowest response of swelling potential was found to be 2.4% corresponding to mix ratio of 0.25:0.95:1.0. This further showed that the improved value of swelling potential was achieved at a higher proportion of quarry dust, which was 95% by weight of solid The model gave highest values of California bearing ratio 32% corresponding to mix ratio of 0.25:0.95:1.0 for water, quarry dust, and soil respectively. The lowest response for California bearing ratio was found to be 15% corresponding to mix ratio of 0.1:0.2:1.0. This indicates that at higher proportion of quarry dust of 95%, the CBR of the treated soil recorded optimal value which meets the requirement of a materials utilization as a base material [68–74]. The model gave highest values of unconfined compressive strength of 258kN/m^2. This value corresponded to mix ratio of 0.25:0.95:1.0 for water, quarry dust, and soil respectively. The lowest response for unconfined compressive strength was found to be 90 kN/m^2 corresponding to mix ratio of 0.1:0.2:1.0. The model gave highest values of loss of strength on immersion durability of 98%, which corresponded to mix ratio of 0.25:0.95:1.0 for water, quarry dust, and soil respectively. The lowest response for loss of strength on immersion durability was found to be 90% corresponding to mix ratio of 0.1:0.2:1.0. The maximum strength value was greater than the minimum value specified by the American association of state highway and transport officials (AASHTO) for the mechanical properties of soil. Using the model, swelling potential, California bearing ratio, unconfined compressive strength and loss of strength on immersion durability of all points in the simplex for a quarry dust treated soil can be derived.

4. Concluding notes

From the foregoing optimization exercise by Scheffe method, selected properties of a treated soft clay soil were optimized and the following remarks are made;

(i) The swelling potential, California bearing ratio, unconfined compressive strength, and durability were optimized at Y_{swl}, Y_{cbr}, Y_{ucs} and Y_{dbl} presented in Eqs. (42)–(45) for which the independent variables viz X_1, X_2 and X_3 adapted by scheffe optimization method of experimental mixtures were utilized. With the optimized equation, the properties' design, behaviour, and performance of treated soft clay soil as a pavement subgrade material will be appropriated and monitored. This will be for any possible volume changes, shear failures, strength failures and durability failures when the material used as a hydraulically bound material is in contact with moisture beyond its optimum and subjected to dynamic load beyond its design value.

(ii) Generally, Scheffe's second degree polynomial was used to formulate models for predicting the swelling potential, California bearing ratio, unconfined compressive strength and loss of strength on immersion durability of quarry dust treated soil. These models could predict the swelling potential, California bearing ratio, unconfined compressive strength and loss of strength on immersion durability of treated soil if the mix ratios are known and vice versa. The response predicted by the models are in good agreement with the corresponding experimentally observed results. The result of these tests shows the feasibility of using quarry dust in soil stabilization. The student *t*-test and the analysis of variance (ANOVA) test were used to check the adequacy of the models, and the models were found to be adequate at 95% confidence level.

Conflict of interests

There are no recorded conflicts of interests in this research work.

Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.mset.2018.10.005.

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