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RESEARCH ARTICLE

MODELLING PORE WATER PRESSURE ON VERTICAL CONSOLIDATION INFLUENCED BY RADIAL FLOW IN HOMOGENOUS SILTY AND FINE SAND FORMATION IN COASTAL AREA OF PORT HARCOURT.

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Abstract

Pore water pressures on vertical consolidation in silty and fine sand formation were found to be influenced by radial flow. These conditions developed different settlement on some soil formation; this can be observed on the accomplishment installation of a system in vertical drain sand. This condition on sand drain essentially consist of a vertical borehole put down through the saturate fine grain soil by extension relatively to bottom, such condition provide the presences of excess pore water pressure dissipated by both vertical consolidation and radial flow as this will definitely be in vertical direction. The development of such behaviour in the study location were express on physical process, mathematical model were developed to monitor the rate of pore water pressure on vertical drain under the influences of radial flow, the derived model generated theoretical values through its simulations, validation were applied to express the behaviuor of the system, both parameter developed a favorable fit, the study is imperative because soil engineers will definitely apply this tools in monitoring the deposition of pore water pressure in the study location.

Introduction:-

Over the years engineers have tried diverse methods to stabilize soils that are subject to fluctuations in strength and rigidity properties as a function of vacillation in moisture content. Stabilization can be derived from thermal, electrical, mechanical or chemical means. The first two options are rarely used. Mechanical stabilization, or compaction, is the densification of soil by application of mechanical energy. Densification occurs as air is expelled from soil voids without much change in water content. This method is particularly effective for cohesion less soils where compaction energy can cause particle reorganization and particle interlocking. But, the method may not be effective if these soils are subjected to important moisture fluctuations. The efficacy of compaction may also reduce with an increase of the fine content, fraction smaller than about 75 µm, of the soil. This is because cohesion and inter particle bonding interferes with particle rearrangement during compaction. Altering the physico-chemical properties of fine-grained soils by means of chemical stabilizers/modifiers is a more effective form of durable stabilization than densification in these fine-grained soils. Substance stabilization of non-cohesive, coarse grained soils, soils with greater than 50 percent by weight coarser than 75 µm is also beneficial if a substantial stabilization reaction can be achieved in these soils. In this case the strength improvement can be much higher, greater than ten fold, when compared to the strength of the untreated material. This report discusses key factors associated with stabilizing soils using chemical modifiers including: The soil must first be classified as either a subgrade category or base category material. In order to be classified as a base material the following criteria must be met: (1) a maximum of 25 percent of the soil mass passes the No. 200 sieve (0.074 mm or 0.003 in.), (2) not more than 40 percent of the soil mass passes the No. 40 sieve (0.42 mm or 0.0165 in.), (3) a maximum plasticity index of 12 percent, and (4) a maximum liquid limit of 40 percent. Otherwise, it is classified as a subgrade material for stabilization purposes. The definition of modification and stabilization can be ambiguous. In this document *modification* refers soil improvement that occurs in the short term, during or shortly after mixing (within hours). This modification reduces the plasticity of the soil (improves the consistency) to the desired level and improves short-term strength to the desired level (short-term is defined as strength derived immediately within about 7-days of after compaction). Even if no significant pozzolanic or cementitious reaction occurs, the textural changes that accompany consistency improvements normally result in measurable strength improvement. *Stabilization* occurs when a significant, longer-term reaction takes place. This longer-term reaction can be due to hydration of calcium-silicates and/or calcium aluminates in Portland cement or class C fly ash or due to pozzolanic reactivity between free lime and soil pozzolans or added pozzolans. A strength increase of 50 psi (350 kPa) or greater (of the stabilized soil strength compared to the untreated soil strength under the same conditions of compaction and cure) is a reasonable criterion for stabilization. Construction steps in the stabilization process are not addressed in this document or in the Standard Practice associated with this document it permeate through soil voids [1]. Where the soil and stabilizing agent are blended and worked together, the placement process usually includes compaction. Oil stabilizing additives are used to improve the properties of less–desirable rood soils. When used these stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents [2]. A difficult problem in civil engineering works exists when the sub-grade is found to be clay soil. Soils having high clay content have the tendency to swell when their moisture content is allowed to increase [3]. Many research have been done on the subject of soil stabilization using various additives, the most common methods of soil stabilization of clay soils in pavement work are cement and lime stabilization. The high strengths obtained from cement and lime stabilization may not always be required, however, and there is justification for seeking cheaper additives which may be used to alter the soil properties. Lateritic soils are widely used as fill materials for various construction works in most tropical countries. These soils are weathered under conditions of high temperatures and humidity with well-defined alternating wet and dry seasons resulting in poor engineering properties such as high plasticity, poor workability, low strength, high permeability, tendency to retain moisture and high natural moisture content [1,2, 3]. The effective use of these soils is therefore often hindered by difficulty in handling particularly under moist and wet conditions typical of tropical regions and can only be utilized after modification/stabilization. Lateritic soils that present such problems during construction processes are termed problematic Laterite [4, 5]. The modification/stabilization of engineering properties of soils is recognized by engineers as an important process of improving the performance of problematic soils and makes marginal soils perform better as a civil engineering material. The application of chemicals such as ordinary Portland cement, lime, fly ash etc. or a combination of these often results in the transformation of the soil index properties which may involve the cementation of the particles. Previously, the most commonly used additive for soil modification or stabilization is the ordinary Portland cement. But recent studies have shown that many of the soil problems can be ameliorated by the addition of pozzolanic fly ash [6, 7]. Experience with soils in the temperate zones revealed that compositional factors namely grain size distribution and plasticity characteristics exert significant influence on the engineering properties of soils [8]. Apart from assisting in the identification and classification of soils, they are indicators of problems in the fundamental properties of the soil such as compressibility, strength, permeability, swell potential and workability. Consequently, great importance is accorded to these properties when lateritic soil is been considered for a project [9]. In this regard, they are used to screen materials for various construction purposes. For example, percentage fines ≥ 30, percentage clay ≥ 15, liquid limit ≥ 20, plasticity index ≥ 7, is specified for liner and cover materials to be used in waste landfills [8], while for road bases, materials with percentage passing BS 200 sieve > 35%, liquid limit > 35% and plasticity index > 12% are rejected without further investigation because such values give indication of poor and undesirable soil qualities for such purposes [10.11]. More so Soil stabilization refers to the procedure in which a special soil, a cementing material, or other chemical material is added to a natural soil to improve one or more of it properties'. One may achieve stabilization by mechanically mixing the natural soil and stabilizing material together so as to achieve a homogeneous mixture or by adding stabilizing material to an undisturbed soil deposit and obtaining interaction by letting it permeate through soil voids [12, 13] Where the soil and stabilizing agent are blended and worked together, the placement process usually includes compaction. Soil stabilizing additives are used to improve the properties of less–desirable rood soils. When used these stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents [14]. A difficult problem in civil engineering works exists when the sub-grade is found to be clay soil. Soils having high clay content have the tendency to swell when their moisture content is allowed to increase [10, 15]. Many research have been done on the subject of soil stabilization using various additives, the most common methods of soil stabilization of clay soils in pavement work are cement and lime stabilization. The high strengths obtained from cement and lime stabilization may not always be required

2. Governing Equation

$$C_n \frac{\partial^2 U_w}{\partial r^2} + \frac{1}{\gamma}\frac{\partial U_w}{\partial r} = \phi \frac{\partial U_w}{\partial t} \quad \text{..} \quad (1)$$

Nomenclature

U_w	=	Pore water pressure
$\frac{1}{\gamma}$	=	Radial flow
t	=	Time
C_n	=	Vertical consolidation
ϕ	=	Porosity

Let $U_w = Tr$

$$C_n Tr^{11} + \frac{1}{\gamma} r^1 T = \phi T^1 r \quad \text{..} \quad (2)$$

$$C_n \frac{r^{11}}{\gamma} + \frac{1}{\gamma}\frac{r^1}{\gamma} = \phi \frac{T^1}{T} = \varphi^2 \quad \text{..} \quad (3)$$

$$C_n \frac{r^{11}}{\gamma} = \varphi^2 \quad \text{..} \quad (4)$$

$$\frac{1}{\gamma}\frac{r^1}{\gamma} = \varphi^2 \quad \text{..} \quad (5)$$

$$\phi \frac{T^1}{T} = \varphi^2 \quad \text{..} \quad (6)$$

This implies that equation (5) and (6) can be written as:

$$\left[\frac{1}{\gamma} - \phi\right]\frac{T^1}{T} = \varphi^2 \quad \text{..} \quad (7)$$

From (4) $C_n \frac{r^{11}}{\gamma} = \varphi^2$

i.e. $C_n \frac{d\gamma}{\gamma} = \varphi^2$ .... (8)

$$\int \frac{d\gamma}{\gamma} = \varphi^2 \int d\gamma \quad \text{..} \quad (9)$$

$$Ln\gamma - \varphi^2 r + C_1 \qquad \text{………………………………….} \qquad (10)$$

$$\gamma = \ell^{\left(\varphi^2 r - C_1\right)} \qquad \text{………………………………….} \qquad (11)$$

$$\gamma = A\ell^{\varphi^2 r} \qquad \text{………………………………….} \qquad (12)$$

From (7) $\left[\frac{1}{\gamma} - \phi\right]\frac{dT}{T} = \varphi^2 dt$ …………………………………. (13)

$$\int \frac{dT}{T} = \frac{\varphi^2}{\frac{1}{\gamma} - \phi}\int dt \qquad \text{………………………………….} \qquad (14)$$

$$LnT = \frac{\varphi^2}{\frac{1}{\gamma} - \phi}T + C_2 \qquad \text{………………………………….} \qquad (15)$$

$$T = Exp\left[\frac{\varphi^2}{\frac{1}{\gamma} - \phi}T + C_2\right] \qquad \text{………………………………….} \qquad (16)$$

$$T = BExp\left[\frac{\varphi^2}{\frac{1}{\gamma} - \phi}T\right] \qquad \text{………………………………….} \qquad (17)$$

Combining (12) and (17), we have

$$U_w(\gamma T) = \gamma, T$$

$$\Rightarrow A\ell^{\varphi t} \cdot BExp\left[\frac{\varphi^2}{\frac{1}{\gamma} - \phi}t\right] \qquad \text{………………………………….} \qquad (18)$$

Hence the expressed model can be written as:

$$U_w(\gamma, T) = AB\exp\left[\frac{\gamma}{C_n} + \frac{t}{\frac{1}{\gamma}\phi}\right]\varphi^2 \qquad \text{………………………………….} \qquad (19)$$

But if Z= V.t and $T = \frac{d}{v}$

$$U_w(\gamma, T) = AB\exp\left[\frac{\gamma}{C_n} + \frac{t}{\frac{1}{\gamma} - \phi}\frac{d}{v}\right]\varphi^2 \qquad \ldots\ldots\ldots\ldots \qquad (20)$$

$$U_w(\gamma, T) = AB\exp\left[\frac{\gamma}{C_n} + \frac{t}{\frac{1}{\gamma} - \phi}V.t\right]\varphi^2 \qquad \ldots\ldots\ldots\ldots \qquad (21)$$

Materials and method:-

Standard laboratory experiment where performed to monitor the rate of flow under pore water pressure on radial flow condition at different formation, the soil deposition of the strata were collected in sequences base on the structural deposition at different locations, this samples collected at different location generate variations at different depth producing different migration of fluid flow developing radial flow at different strata, the experimental result are applied to compared with the theoretical values to determined the validation of the model.

Results and Discussion:-

Results and discussion are presented in tables including graphical representation of pore water pressure values.

Table 1: Theoretical values at pore water pressure Different Depth:-

Depth [m]	Theoretical Values of Pore Water Pressure
0.2	0.039
0.4	0.04
0.6	0.042
0.8	0.043
1	0.047
1.2	0.057
1.4	0.07
1.6	0.086
1.8	0.105
2	0.28
2.2	0.34
2.4	0.42
2.6	0.52
2.8	0.63
3	0.77
3.2	0.94
3.4	1.157
3.6	1.41
3.8	1.72
4	2.11

Table 2: Theoretical values at pore water pressure Different Time:-

Time	Theoretical Values of Pore Water Pressure
0.01	0.039
0.02	0.04
0.04	0.042
0.05	0.043
0.1	0.047
0.2	0.057
0.3	0.07
0.4	0.086
0.5	0.105
1	0.28
1.1	0.34
1.2	0.42
1.3	0.52
1.4	0.63
1.5	0.77
1.6	0.94
1.7	1.157
1.8	1.41
1.9	1.72
2	2.11

Table 3: Theoretical values at pore water pressure Different Depth:-

Depth [m]	Theoretical Values of Pore Water Pressure
4	0.047
8	0.057
12	0.07
16	0.086
20	0.105
24	0.28
28	0.43
32	0.52
36	0.63
40	0.77
44	0.94
48	1.16
52	1.42
56	2.11

Table 4: Theoretical values at pore water pressure Different Depth:-

Time	Theoretical Values of Pore Water Pressure
0.1	0.047
0.2	0.057
0.3	0.07
0.4	0.086
0.5	0.105
1	0.28
1.2	0.43
1.3	0.52
1.4	0.63
1.5	0.77
1.6	0.94
1.7	1.16
1.8	1.42
2	2.11

Table 5: Theoretical and measured Values of pore water pressure at Different Depth:-

Depth [m]	Theoretical Values of Pore Water Pressure	Measured Values
0.2	0.039	0.026
0.4	0.04	0.034
0.6	0.042	0.039
0.8	0.043	0.041
1	0.047	0.045
1.2	0.057	0.055
1.4	0.07	0.065
1.6	0.086	0.081
1.8	0.105	0.1
2	0.28	0.26
2.2	0.34	0.31
2.4	0.42	0.39
2.6	0.52	0.51
2.8	0.63	0.61
3	0.77	0.75
3.2	0.94	0.91
3.4	1.157	1.13
3.6	1.41	1.39
3.8	1.72	1.69
4	2.11	2.09

Table 6: Theoretical and measured Values of pore water pressure at Different Depth:-

Time	Theoretical Values of Pore Water Pressure	Measured Values
0.01	0.039	0.034
0.02	0.04	0.038
0.04	0.042	0.039
0.05	0.043	0.038
0.1	0.047	0.045
0.2	0.057	0.053
0.3	0.07	0.068
0.4	0.086	0.084
0.5	0.105	0.103
1	0.28	0.25
1.1	0.34	0.32
1.2	0.42	0.39
1.3	0.52	0.49
1.4	0.63	0.62
1.5	0.77	0.74
1.6	0.94	0.89
1.7	1.157	1.12
1.8	1.41	1.37
1.9	1.72	1.67
2	2.11	2.06

Table 7: Theoretical and measured Values of pore water pressure at Different Depth:-

Depth [m]	Theoretical Values of Pore Water Pressure	Measured Values
4	0.047	0.041
8	0.057	0.051
12	0.07	0.065
16	0.086	0.081
20	0.105	0.103
24	0.28	0.23
28	0.43	0.41
32	0.52	0.49
36	0.63	0.61
40	0.77	0.74
44	0.94	0.91
48	1.16	1.12
52	1.42	1.39
56	2.11	2.07

Table 8: Theoretical and measured Values of pore water pressure at Different Depth:-

Time	Theoretical Values of Pore Water Pressure	Measured Values
0.1	0.047	0.046
0.2	0.057	0.052
0.3	0.07	0.064
0.4	0.086	0.087
0.5	0.105	0.101
1	0.28	0.26
1.2	0.43	0.41
1.3	0.52	0.48
1.4	0.63	0.62
1.5	0.77	0.75
1.6	0.94	0.93
1.7	1.16	1.12
1.8	1.42	1.38
2	2.11	2.09

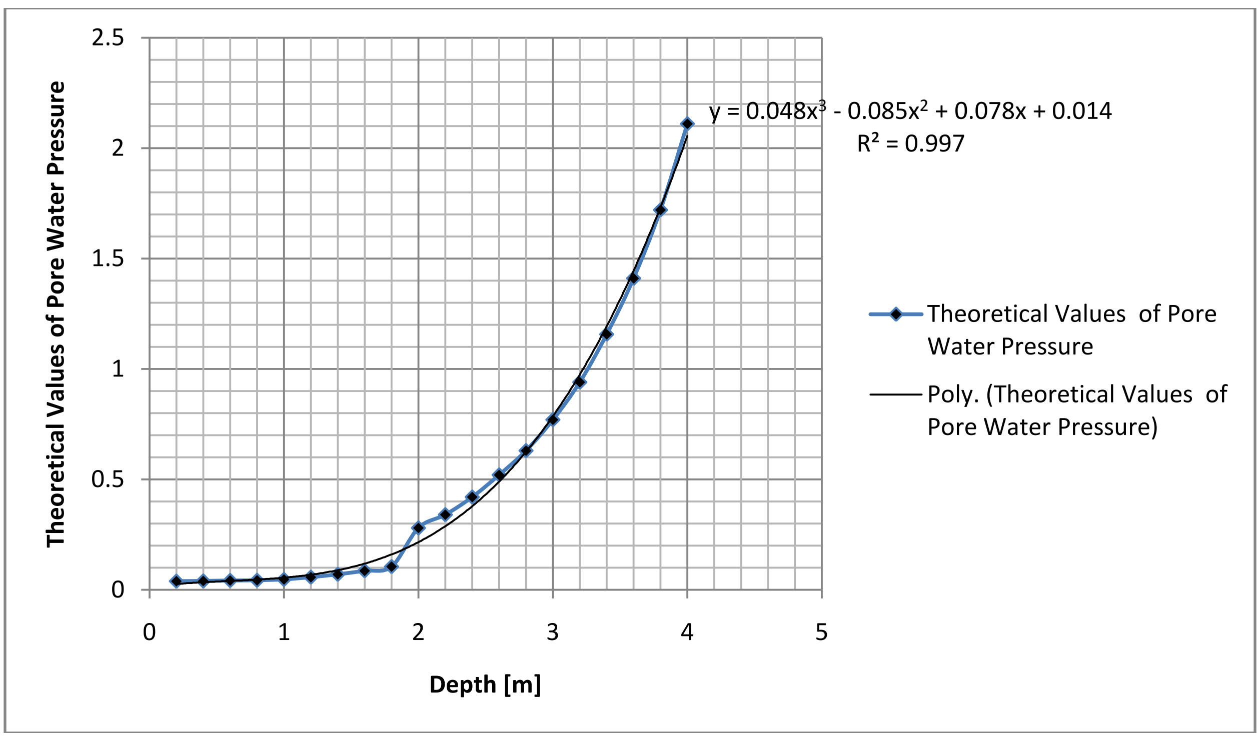

Figure: 1 Theoretical values at pore water pressure Different Depth:-

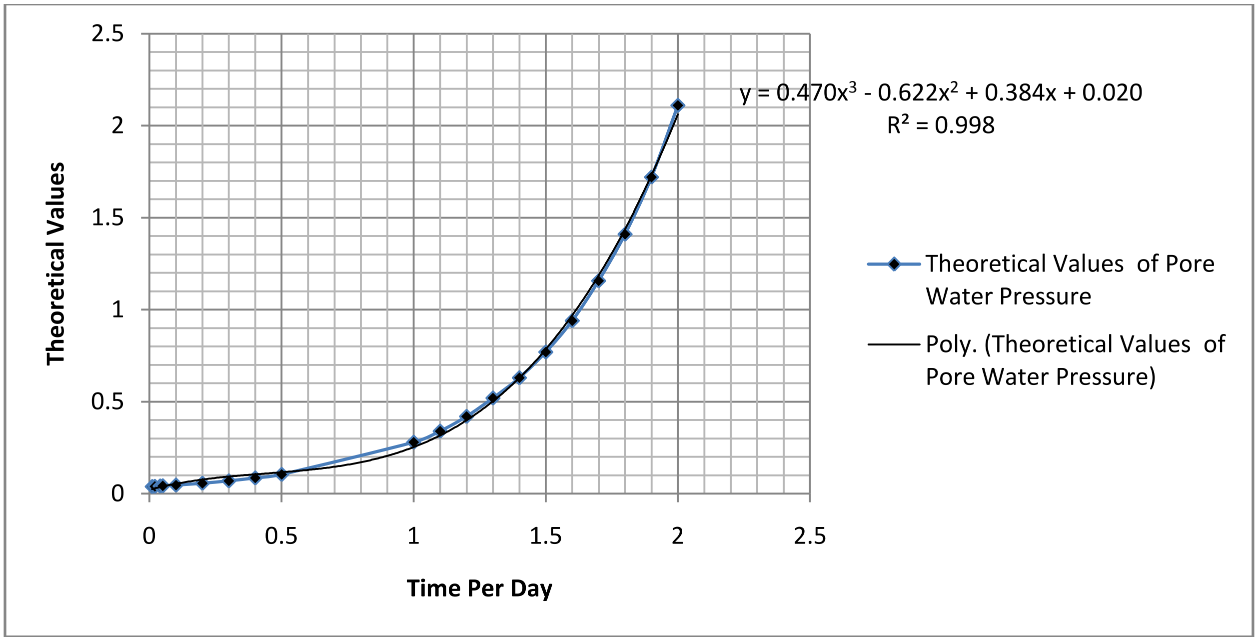

Figure: 2 Theoretical values at pore water pressure Different Time:-

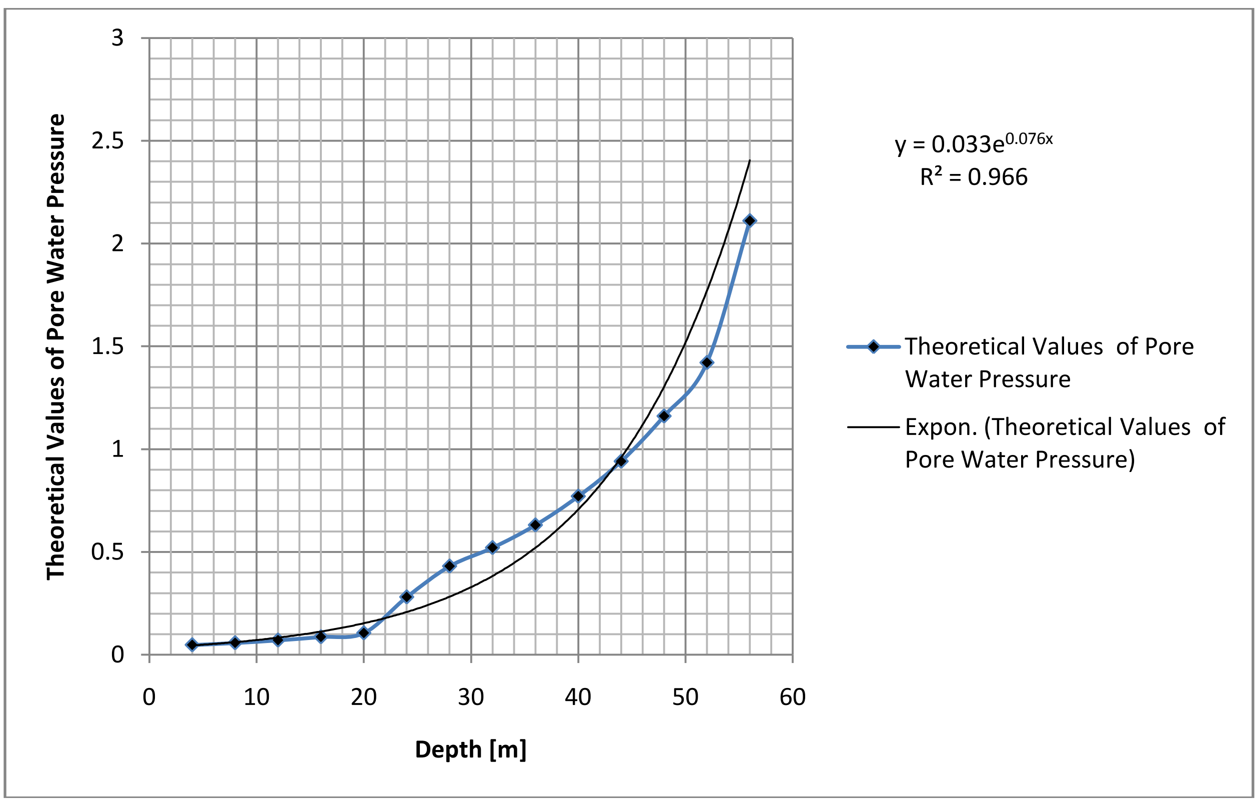

Figure: 3 Theoretical values at pore water pressure Different Depth:-

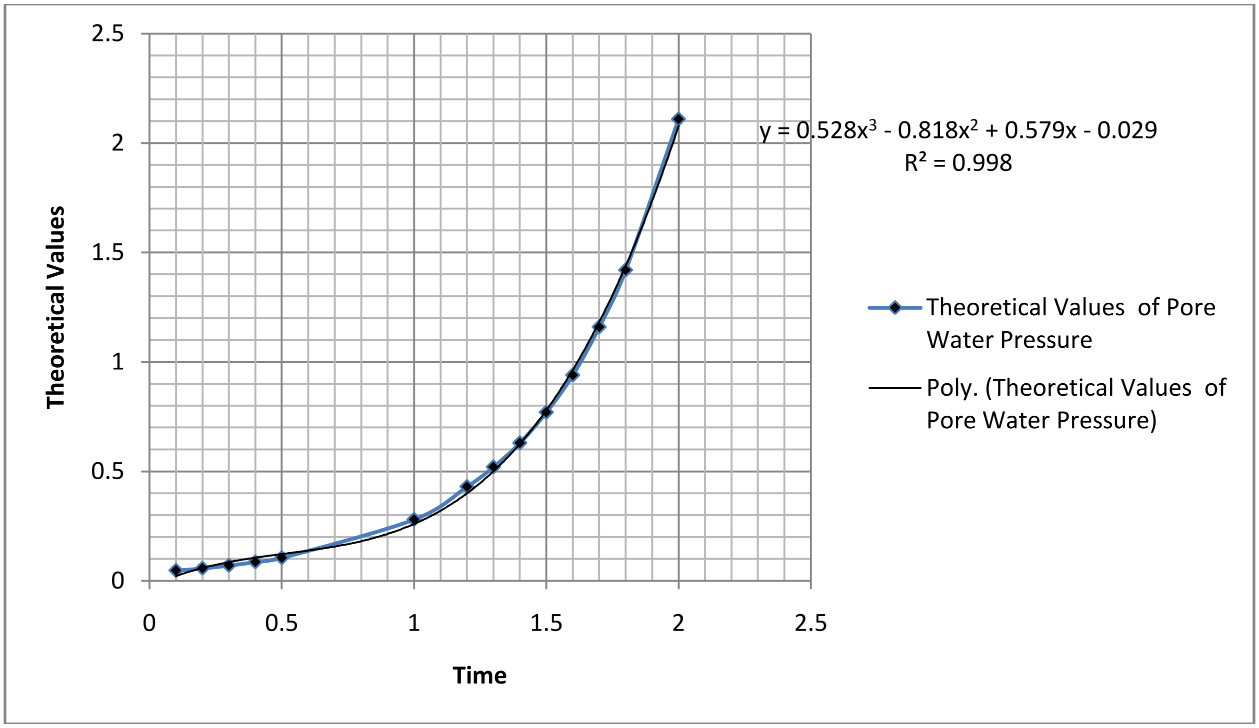

Figure: 4 Theoretical values at pore water pressure Different Time:-

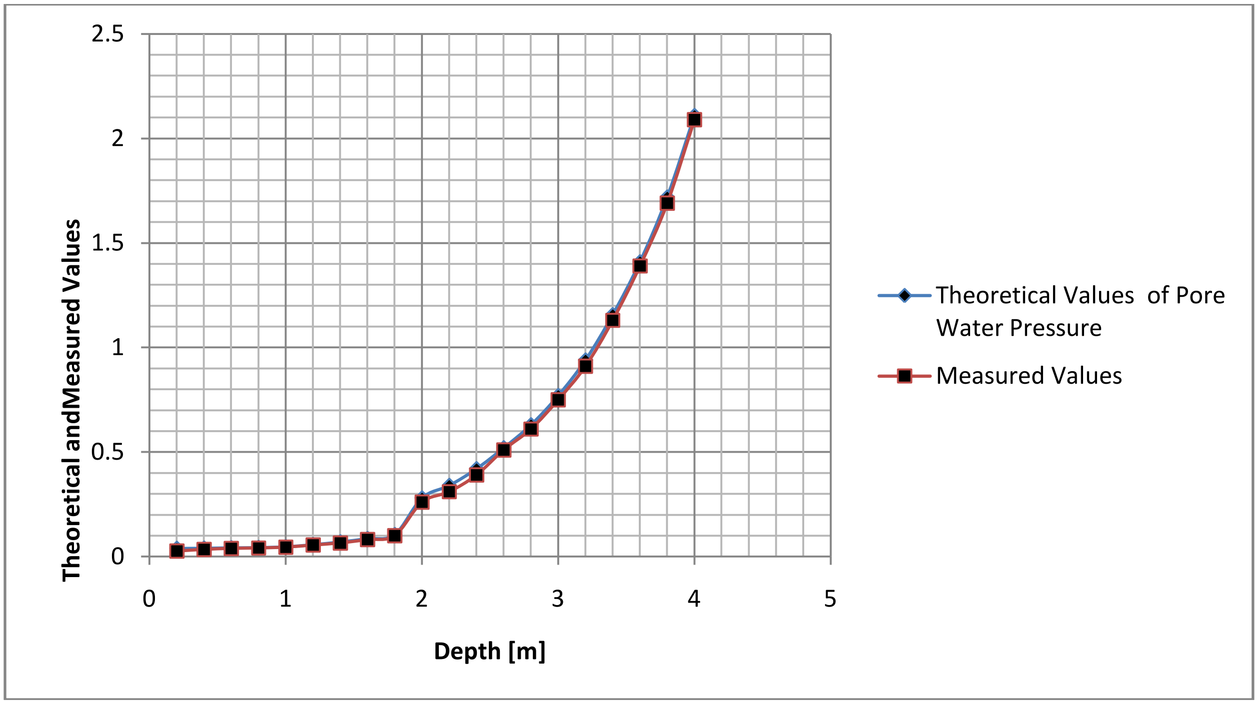

Figure: 5 Theoretical and measured Values of pore water pressure at Different Depth:-

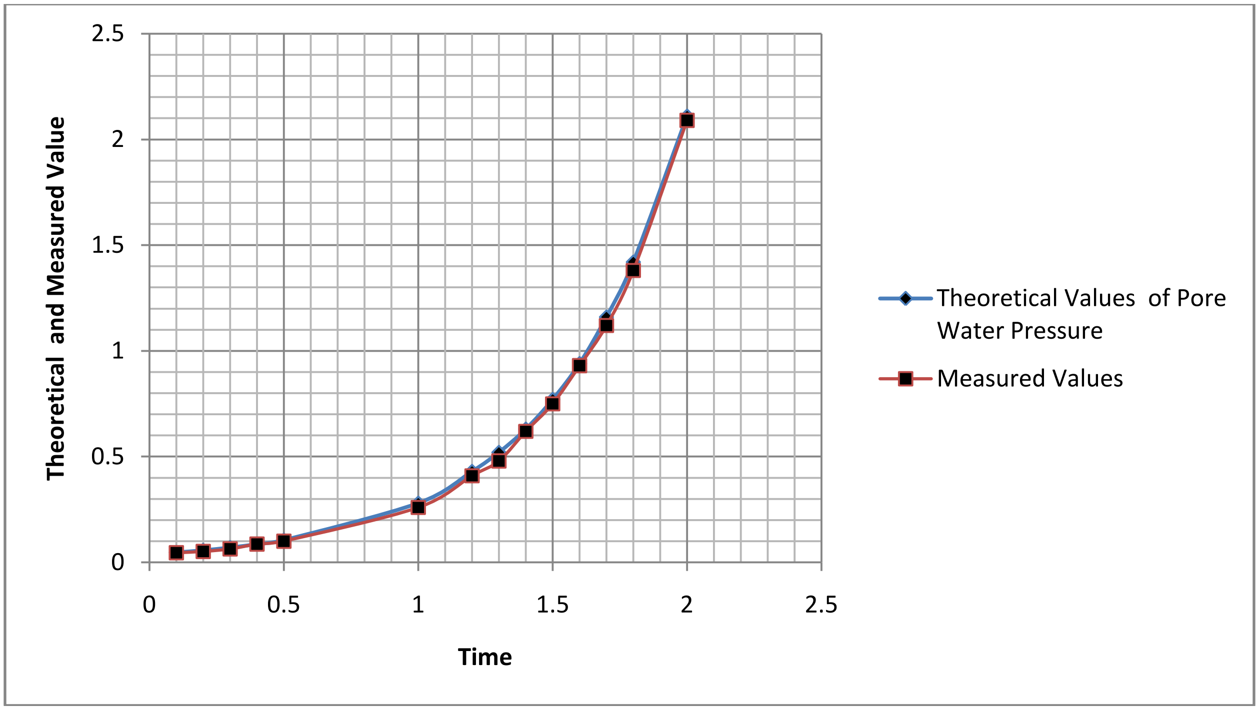

Figure: 6 Theoretical and measured Values of pore water pressure at Different Time:-

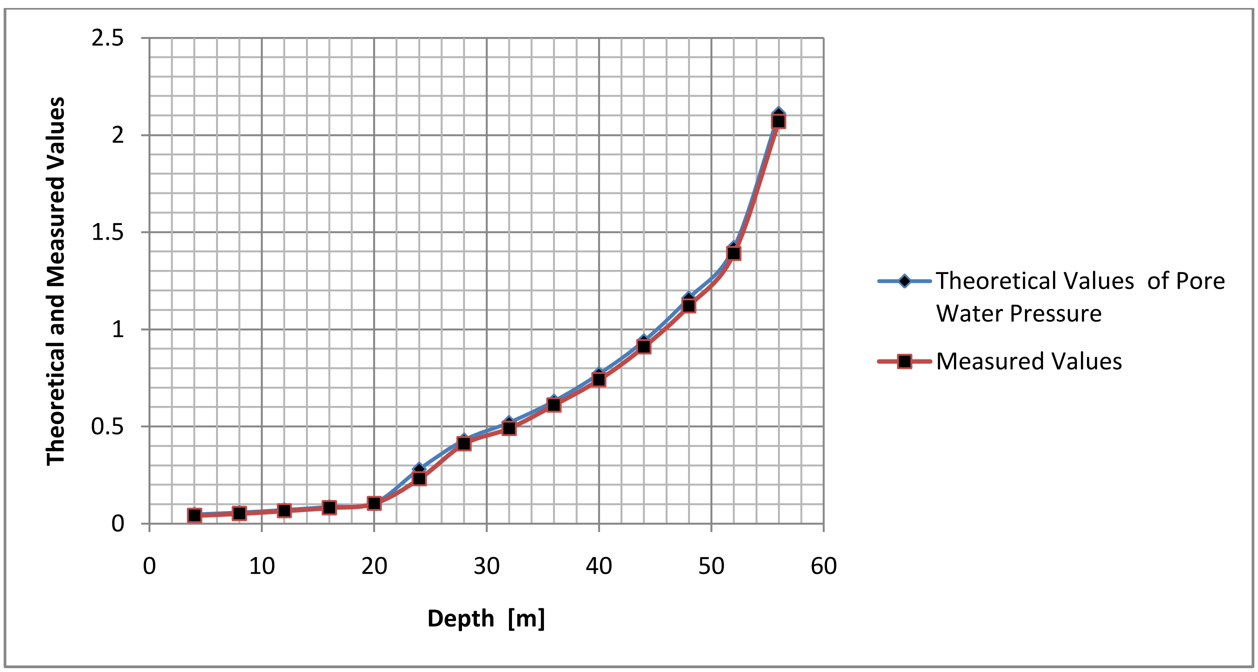

Figure: 7 Theoretical and measured Values of pore water pressure at Different Depth:-

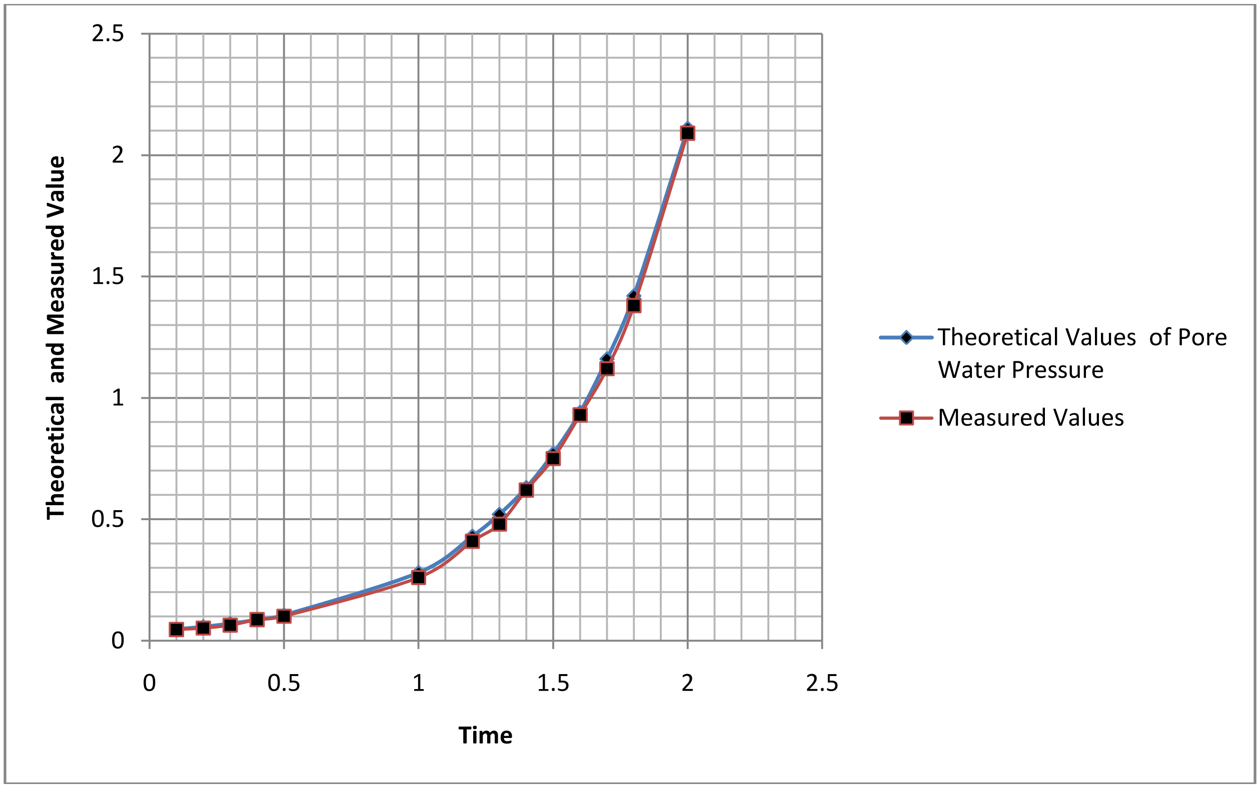

Figure: 8 Theoretical and measured Values of pore water pressure at Different Time:-

The study express the slow process of consolidation of fine grained soil through accelerated shortening the length of the drainage of the pore water escaping from the consolidating soil, these condition express the influences of pore water pressure under a vertical bore hole, expressing the modeling condition, the study generated theoretical values express through graphical representation, the figure from one to four shows exponential phase on pore pressure from the lowest at [0.2 m] and the highest at [4m] the expression were gradual migration between [0.2-1.0 m] in some certain depth, similar condition were monitored with respect to time of migration. The express theoretical values were compared with experimental values, similar condition were experienced with that theoretical values, this implies that the both parameters developed favorable fits, comparing both values developed the lowest and the highest at the same depths. These conditions established the rate of excess pore water pressure dissipate by both vertical and radial flow as it express resulting settlement that will be in vertical direction, these condition has developed free strain condition through the application of a flexible surcharge load which developed to uneven settlement at the surface .

Conclusion:-

Pore water pressure are expressed in many conditions on consolidation of the formations, in most conditions there is the tendency of compressibility of some strata, for example sandy soil deform less under static load, more than dynamic loads. Static load will not cause any reduction in the void space in the sand, but in non cohesive soil formation settlement are found to occur immediately after the application of load, this settlement is attributed to volume of change caused by lateral yielding or shear strain that occur in the soil. The layer as elastic medium. These conditions are expressed in the behaviour of pore water pressure on vertical consolidation of soil. Such condition generated other dimension that developed modeling method; it can be applied to monitor the rate of pore water pressure in radial flow through vertical consolidation. The developed model was simulated to determine the behaviour of pore water pressure under the influences of radial flow. Validation of the model were through comparison between the theoretical and experimental values, both parameter developed favorable fit.

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