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Preditive models for validation of plastic limits of soils in Portharcourt Metropolis, Rivers State, Nigeria

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ABSTRACT

This paper assesses developed models for prediction and evaluation of the effect of plastic limit for design of roads in the study area. The study was conducted to develop models that will be a specification or parameters in road design and construction in the study location. Samples from different locations were collected and were subjected to laboratory analysis. Nine samples out of numerous locations were analyzed and applied to develop predictive models equations, and the theoretical values generated from the model equations were compared with other measured values from the study location. The predictive models for the study location compared favorably well with the measured values. These values are between medium and high plasticity of soils. The predictive model can be applied as established parameter that can be integrated in the construction and design of roads. Hence sub base predictive models will be effective in maintaining the life span of the road if it is integrated in design. This paper is imperative because there exist a lot of variations in the plastic limit of soils in the study area.

INTRODUCTION

Since the early 1900's with the advent of motorized transport, roads in Africa entered a new technological era. The majority of roads were relics of the days of ox wagons. Paths followed by indigenous people became wagon tracks and then earth roads and finally these became gravelled surfaced roads (Floor, 1985).These gravelled surfaces, when affected by water, became muddy and unstable, eventually destabilized. Consequently in 1935 a National Road Board (NRB) was established to improve the condition of roads in South Africa [1]. Finally in 1938, after a study on the behaviour of gravel surface roads NRB concluded that all National Roads must be made of an all weather asphalt surface, otherwise referred to as flexible pavement. Internationally, flexible pavements were the most commonly constructed roads in the early 1800's. This is evident as the first rigid pavement, namely Portland cement concrete road was only built in 1865 in Britain, which provided access to a goods yard of Inverness railway station. By comparison, the first modern concrete pavement was constructed as late as 1968 in South Africa [1]. It is evident that South Africa was already lagging behind the rest of the world's concrete road strategies [1].Engineering properties of soil always vary from place to place due to the variation in soil formation, base on the geological deposition. When the soils within the possible corridor for the road vary in strength extensively from place to place, it is visibly advantageous to locate the pavement on the stronger soils, if this does not have other constraints [1]. Thus, since variety process of route corridor pressure the pavement structure and the construction costs, thorough examination should be done on the characteristics of subgrade. Failures of roads are being observed before their design period and are greatly distressing the economic growth of the country [6]. Such failures could be conquer by undertaking through examination on the subgrade material and the materials overlaying the subgrade and incorporating it in the design [5]. Prior to bearing in mind remedial procedures for defects or reconstruction or overlay, it is necessary that the engineer takes into account, numerous parameters that are necessary for proper assessment of the existing pavement condition. In particular, it is imperative to ascertain whether definite types of pavement distress are progressive, leading to eventual failure of the road, or whether they are non-progressive [4]. Excessive movement of flexible pavements, which eventually result in uneven riding qualities, may mostly be caused by poor qualities of the subgrade, subbase, base course or wearing course. A qualitative measure of the effect of the movement can be determined only after a thorough investigation is undertaken [5]. The investigations might take the form of trenching or bore logging in which visual inspection is made on the cross section of the Pavement structure [4]. Measurements of the thickness and/or analysis of the structural thickness of the various paving layers inside and outside the traffic lanes are certainly vital. Testing of various pavement components assists in the evaluation of the probable cause of distress [2]. Each distress must be evaluated to determine whether the distress will be progressive or whether it represents an inactive condition. Failed surfaces could be classified into different categories depending on the patterns of failures. Previous studies provides basic information on the most common types of pavement failures and their probable cause this is described by interrelated cracks forming a series of small polygons resembling an alligator's skin. This could result from the fatigue effect of repetitive heavy truck loads or ageing in combination with exponential loss of pavement thickness. It can occur with or without surface distortion and pumping [4, 1]. A rut is a longitudinal deformation at wheel tracks mainly associated with shoving along the road. This is caused by heavy loads and high tyre pressure, subgrade settlement caused by saturation, poor construction methods, or asphalt mixtures of inadequate strength Potholes are known to be irregularly shaped holes of various sizes. These is one of the most frequent result from wear or destruction of the wearing course, in some condition it is sometimes from the presence of foreign bodies in the surfacing. They can also be caused by water penetrating the surface and causing the base and/or subgrade to become wet and unstable. They are small when they first appear. In the absence of maintenance, they grow and reproduce in rows. This is one of increasing loss of pavement material. The possible cause for raveling could be disjointing of bituminous film from aggregates throughout stripping caused by insufficiency of bonding or ageing of surface due to variations in climatic and loading conditions. It can also occur due to the inconsistent deformation of the lower pavement layers [7].Other developed problem is blocking cracking, leading to chipping of pavement surfacing and/or upheaval outside the tyre cracks with associated cracking. Consequence it has result to deficiency in cohesion and internal friction in pavement base structure due to ageing and fatigue Is for certainty that pavement's wearing course is imperative component for road, the success or failure of a pavement is dependent upon the underlying subgrade material upon hich the pavement structure is built. Thus, the subgrade must be able to support the loads transmitted from the pavement structure without undergoing excessive settlement. Its performance generally depends on its load bearing capacity, moisture content and volume changes. Moreover, its load bearing capacity depends on the degree of compaction, moisture content and soil type. Hence, the relationships among the strength, density and moisture content should be studied thoroughly [3].desirable properties ensure that subgrade should possess maximum strength, drainage, ease of compaction, permanency of compaction, and permanency of strength. Since subgrade vary considerably, it is essential to make a thorough examination of the soils in place and, from this, to establish the design of the pavement. The determination of the subgrade strength in order to use for the design of the road pavement requires ascertaining the density-moisture content-strength relationship(s) specific to the subgrade soil(s) encountered along the road under study. It is also necessary to select the density which will be representative of the compacted subgrade and the moisture content during and after construction. Moisture conditions in the subgrade are controlled primarily by the local environment. Since design concepts for flexible pavements are based upon model-prototype principles, wherein samples of soil are tested in the laboratory under simulated field conditions, it becomes necessary to predict the ultimate moisture content of the subgrade so that this value can be used in the testing program. The strength of the road subgrade for flexible pavements is commonly examine in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct examination of the likely strength or CBR of the subgrade soil under the accomplished road pavement is often hard to make. Its value, however, can be inferred from an approximation of the density and moisture content of the subgrade jointly with knowledge of the relationship between strength, density and moisture content for the soil in question. This correlation must be determined in the laboratory. The density of the subgrade soil can be restricted within limits by compaction at appropriate moisture content at the time of construction. According to the ERA Pavement Design Manual [4], It is recommended that the top 25cm of all subgrade should be compacted to a relative density of at least 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction). Otherwise, at least 93% of the utmost dry density achieved by ASTM Test Method D 1557 may be specified. With current compaction equipment, a relative density of 95% of the density obtained in the heavier compaction test should be achieved without complexity, but tighter control of the moisture content will be essential [5]. As a result, it is generally suitable to base the determination of the design CBR on a density of 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction) or, alternately, on 93% of the utmost dry density achieved by ASTM Test Method D 1557 (heavy or modified compaction) [5]. The structural manual record given in the ERA Pavement Design Manual Volume I requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength Over the past 20 years, the construction industry has been forced to adapt its methods of design, specification and construction to achieve sustainability targets forced by the Government. [9] describes sustainable expansion as enabling all people all over the world to satisfy their basic needs and enjoy a better quality of life, devoid of compromising the quality of life of future generations. Therefore in the framework of this study, sustainability can be defined as the duty to conserve income and the atmosphere for future generations whilst maintaining a quality of life. A present in the UK, over 200 million tones of rock material are quarried every year for use as aggregates, cement and other building materials [8]. The Waste and Resources Action Programme (WRAP) was created to sustain the Government's plan of growing the use of recycled aggregates in England to 60 million tones per annum by 2011. Taxes on primary quarried aggregates and waste disposal to landfill have been introduced to promote the reuse of resources. The highway sector, in particular the Highways Agency, has and continues to respond to this requirement through the expansion of new standards and guidance, and of particular interest is the move toward design methods that utilize the performance related parameters of the constituent materials. An earthworks balance is performed at the design stage to maximize materials resource efficiency by the optimum use of site won material, reducing the need to bring new materials onto site from distance and unnecessary removal of material from site. In general the material from an area of cut is used in areas of fill and this process greatly minimizes aspects of environmental impact, transportation requirements and costs. In addition, much new guidance is available concerning the permissible range of recycled or marginal materials that should be assessed and potentially included into a scheme much research has been carried out that is now emerging and being applied through these new documents, and this project has formed a part of that evolution. The previous advice in the Design Manual for Roads and Bridges [10, 11] aimed to provide a 'standard foundation' prescribing a capping and/or sub-base minimum thickness, based on the condition of the subgrade (as defined by its California Bearing Ratio value). The guidance was largely based upon TRRL Report LR 1132 [12, 13] It intrinsically prescribed all foundations as both similar in performance and of adequate expected performance through the use of a method specification. Now twenty years later, reflecting the need to consider a wider range of materials in pavement design and construction, Nunn (2004) published TRL Report TRL615 which embodied a more versatile approach to flexible and flexible composite pavement design. [7,14]. Proposed to categories foundations into four classes described by their composite stiffness. This approach permitted variations in the bound upper layer design thickness depending on the foundation stiffness and forecasted traffic loading, which was a significant step forward. In 2006, the Highways Agency published Interim Advice Note 73 [14] providing detailed design guidance for the four classes of road foundations based on their performance. This included a new performance based Specification prescribing field compliance testing for assurance of 'performance' designs. Thus, the pavement designer now has the opportunity to integrate the foundation and upper pavement design and gain the potential benefits. However, it can be argued that the more sophisticated methods and guidance emerging requires greater Materials testing and understanding of their fundamental properties than was previously necessary. This project was borne out of this requirement.

MATERIALS AND METHODS

This test was conducted in accordance with Bs 1377 *1975* test 3. The plastic limit of a soil is the water content expressed as a percentage of the mass of the oven dried soil at the boundary between the plastic and semisolid states. The water content at this boundary is arbitrarily defined as the lowest water content at which the 5011 can be rolled into 3.0mm diameter threads without breaking into pieces. The plastic limit was determined by measuring the water content of the soil when threads 3.0mm diameter made from that particular soil just starts to crumble and can be taken as the smallest or minimum moisture content at which the soil can be rolled into 3.0mm diameter thread without breaking up.

Procedure.
About 50gm of laboratory air dried soil sample was ground to the consistency of powder and sieved with a sieve (300mm). 20gm of this sieved soil was then taken and mixed thoroughly with some quantity of distilled water with the aid of a spatula until it formed a ball. This soil bail was now placed on top of a flat glass plate and rolled

continuously with the palm until 3.0mm soil threads was obtained. Part of this soil was then put into the oven for its moisture content to be determined. The process was repeated with further addition of sieved soil until the *3.0mm* diameter threads just starts to umble. Part of this last soil and water mixture was removed and installed in the oven for is moisture content determination like for others. r plasticity index (P1) was calculated from the expression; P1 LL -PL, utilizing tile reading obtained after each water addition. The results generated from the experiments were subjected excel programs plotted each location result at the study area, the results plotted generated a model that that can be resolved to solve problem in other location were the experimental results are not available

RESULTS AND DISCUSSION

Results and discussion t*o* developed models prediction and evaluation to examine the effect of plastic limit of soil design mechanism of roads construction and design of roads are presented in tables and figures.

Table 1: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	16.82	20
0.4	17.99	22
0.8	21.85	23
1	22.33	25
1.5	24.14	23
2.5	24.02	24

Table 2: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	16.55	17
0.4	18.44	19
0.8	20.66	21
1	22.24	22
1.5	23.22	24
2.5	19.27	21

Table 3: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	28.37	28
0.4	26.17	26
0.8	22.63	23
1	21.19	22
1.5	18.64	21
2.5	17.97	20

Table 4: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	24.18	22
0.4	27.4	26
0.8	27.89	28
1	26	25
1.5	18.52	19
2.5	13.92	14

Table 5: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	20.25	20
0.4	21.47	21
0.8	23.33	21
1	23.98	22
1.5	24.92	23
2.5	24	22

Table 6: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	17.92	17
0.4	19.05	19
0.8	20.32	21
1	20.54	22
1.5	20.12	21
2.5	16.98	17

Table 7: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	11.06	12
0.4	17.64	15
0.8	24.78	24
1	25.97	26
1.5	24.57	24
2.5	19.93	21

Table 8: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	14.17	13
0.4	16.42	18
0.8	20.18	18
1	21.69	22
1.5	24.37	25
2.5	25.09	25

Table 9: Comparison of theoretical and measured values at different Depth

Depth	Theoretical	Measured Values
0.2	10.95	12
0.4	16.86	15
0.8	23.63	24
1	25	26
1.5	24.61	24
2.5	20.96	21

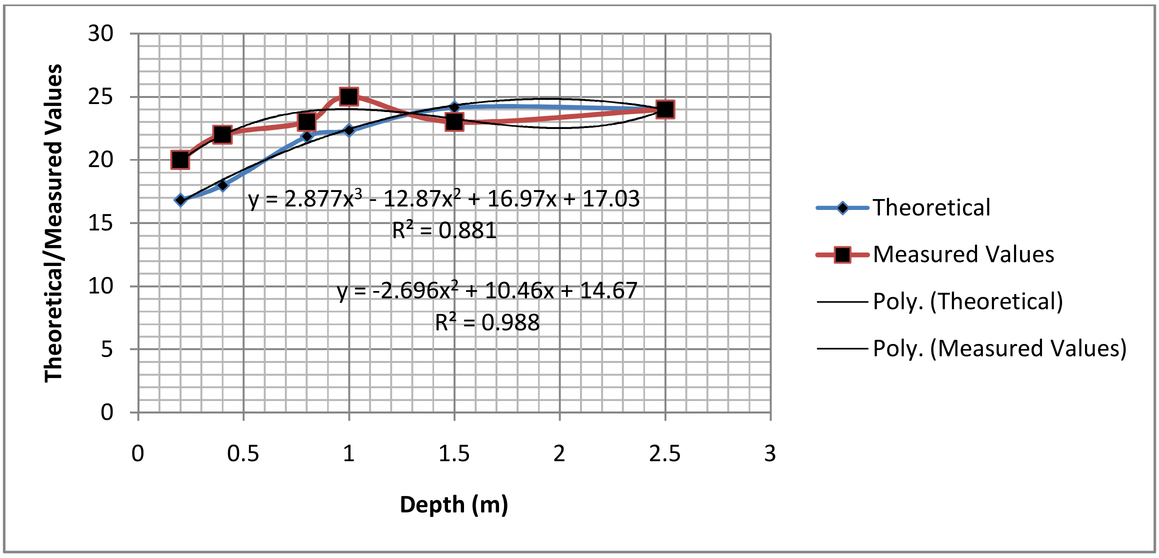

Figure 1: Comparison of theoretical and measured values at different Depth

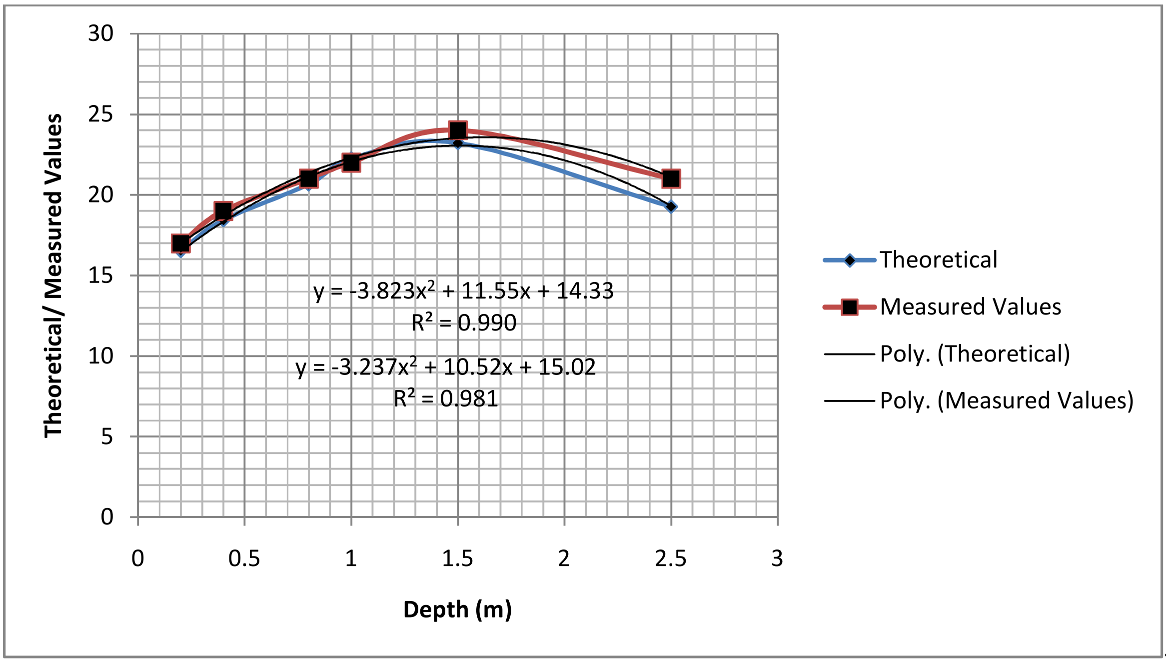

Figure 2: Comparison of theoretical and measured values at different Depth

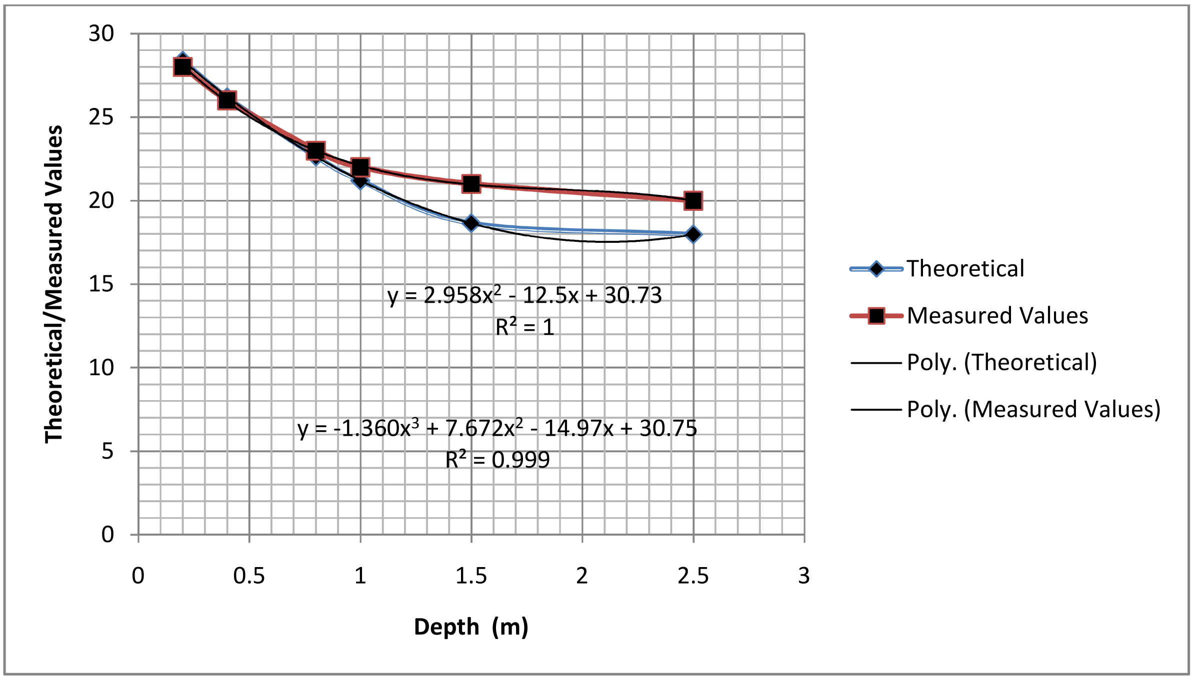

Figure 3: Comparison of theoretical and measured values at different Depth

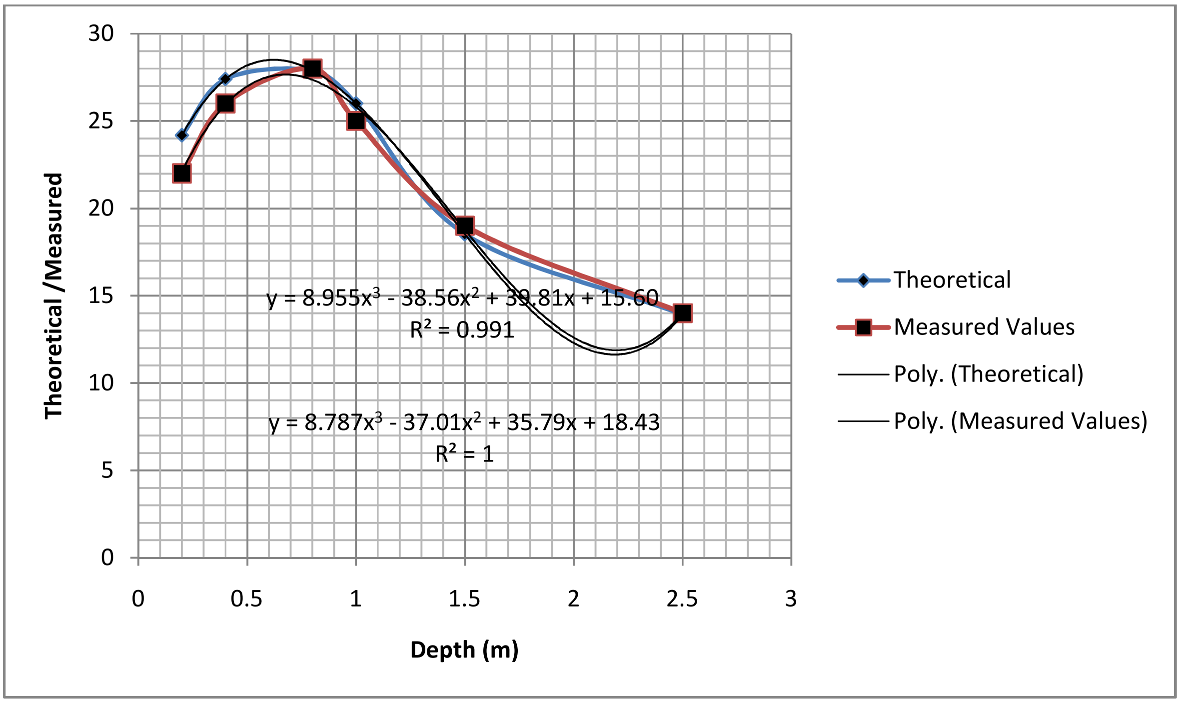

Figure 4: Comparison of theoretical and measured values at different Depth

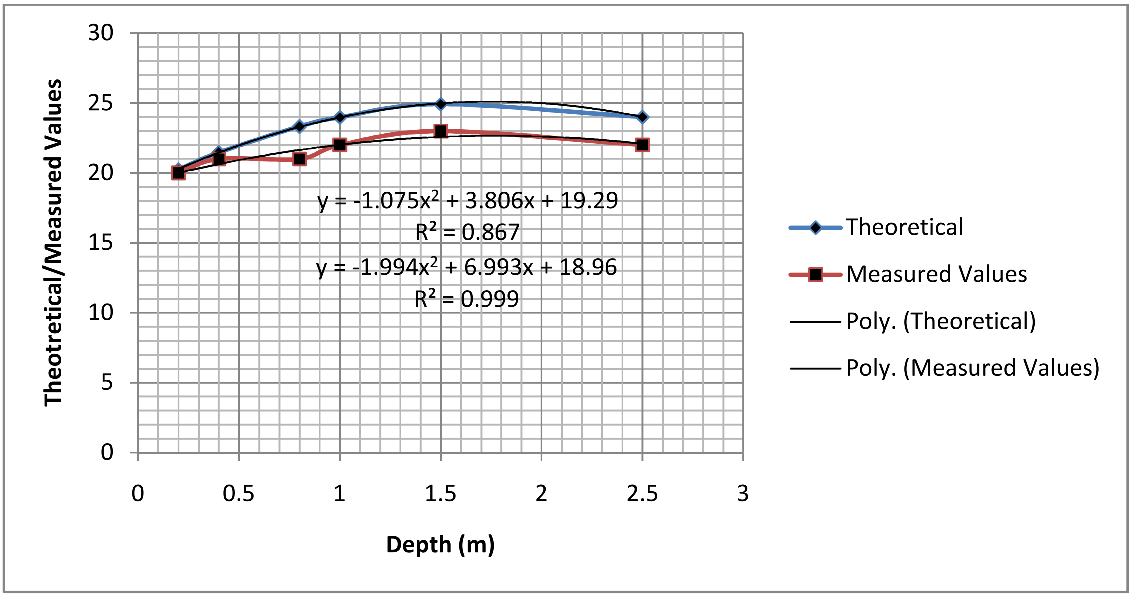

Figure 5: Comparison of theoretical and measured values at different Depth

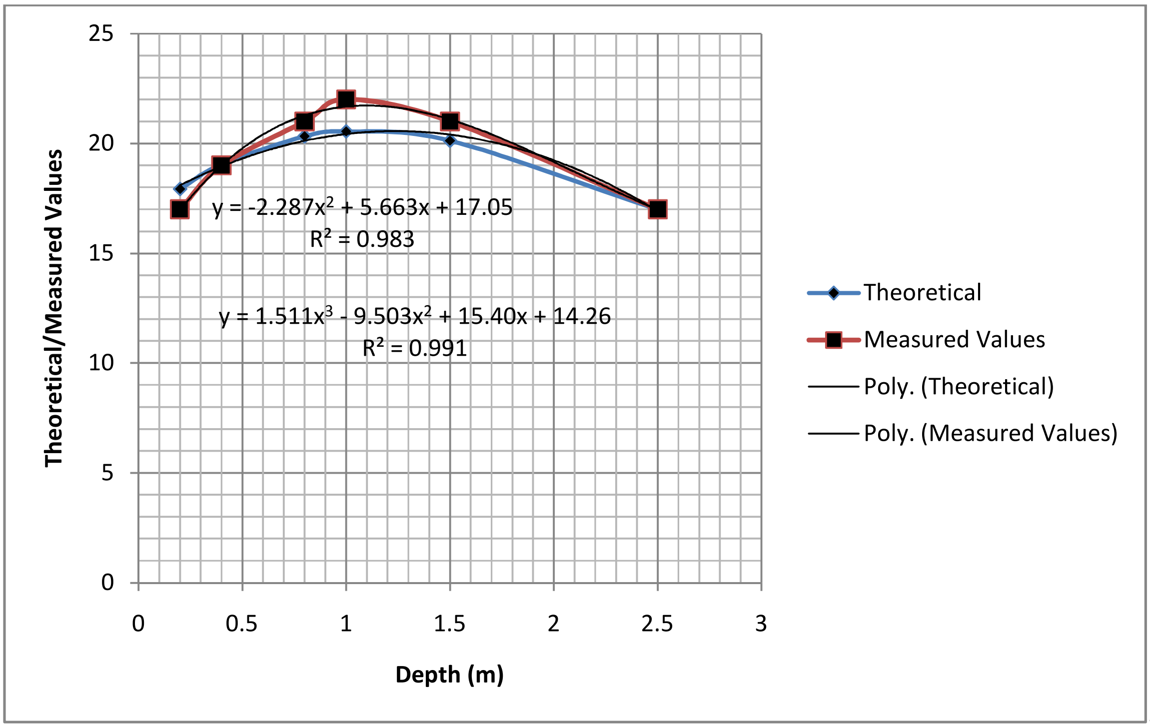

Figure 6: Comparison of theoretical and measured values at different Depth

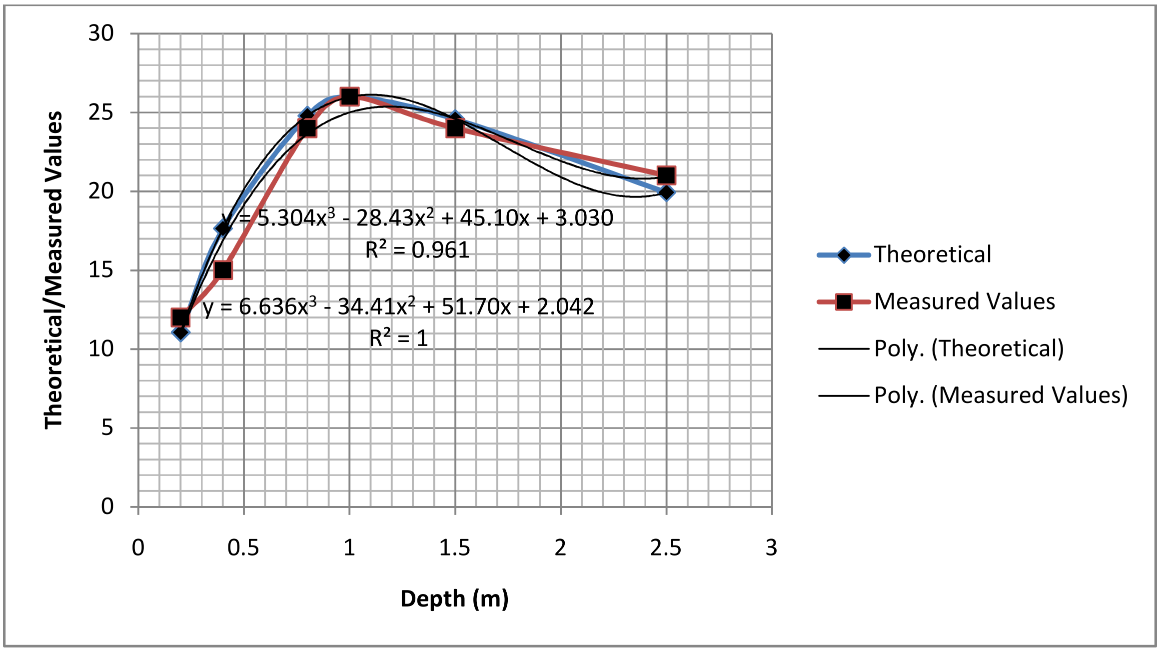

Figure 7: Comparison of theoretical and measured values at different Depth

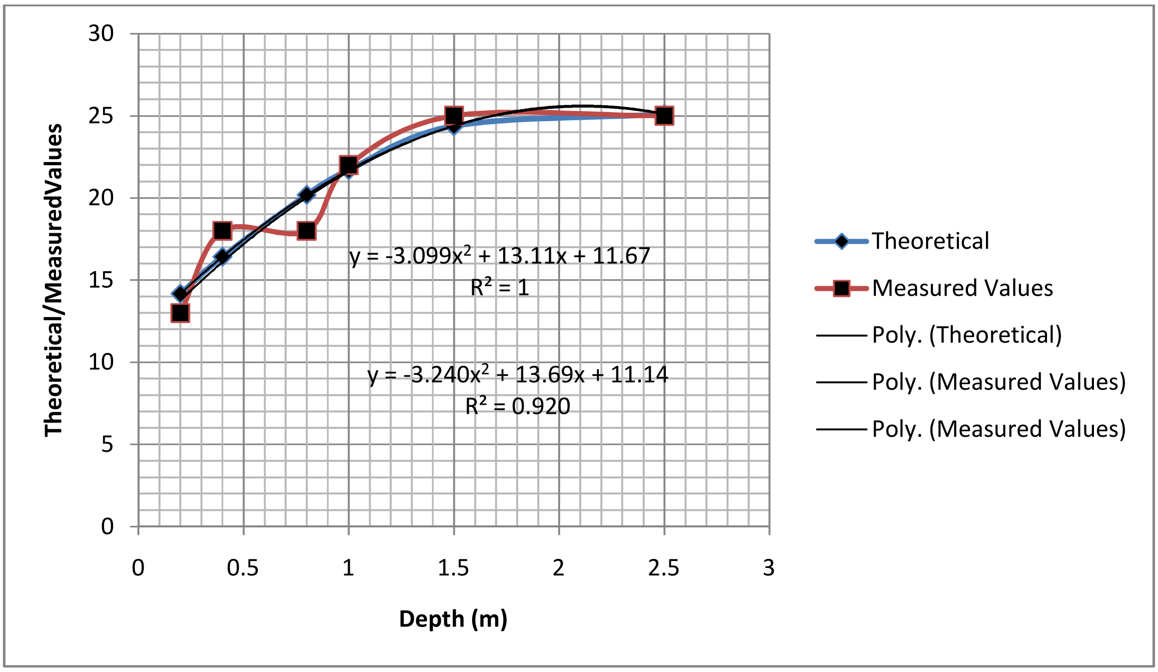

Figure 8: Comparison of theoretical and measured values at different Depth

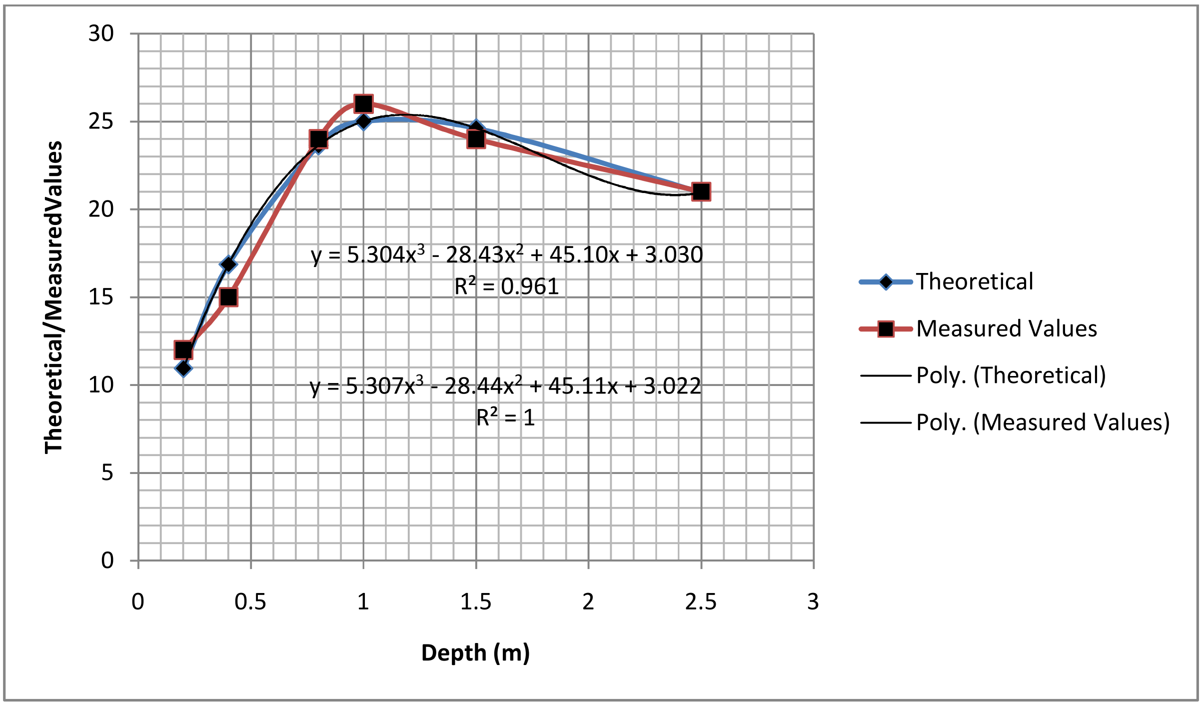

Figure 9: Comparison of theoretical and measured values at different Depth

Figure 1 shows that the theoretical model deposited its rate of plasticity from the lowest 16 at 0.4m gradually to a point where an optimum value were recorded at 24.14 1.5m deep, while the measure values were found to deposit its rate of plasticity in a fluctuation form in a similar condition like that of theoretical model, the lowest deposited at the same level like that of theoretical model, but the optimum value deposited 2.5 deep, finally maintaining a similar curve like that of theoretical model. It generated predictive model of $Y = 2.877x^3 – 12.87x^2 + 16.97x + 17.08$ and R^2

= 0.881, $Y = -2.696x^2 + 10.46x + 14.67$ and $R^2 = 0.988$. In Figure 2 both parameters gradually increase with depth to a point where an optimum value of plastic limit were recorded at 23.22 1.5m deep and finally reduced down to 19 and 21 1.5-2.5m deep respectively, theoretical and measured values were found to compare faviourably well. It generated predictive model equation as $Y = -3.823x^2 + 11.55x + 14.33$ with $R^2 = 0.990$, $Y = -3.237x^2 + 10.52x + 15.02$ with $R^2 = 0.981$. Figure 3 both parameters displayed its optimum value at 28.37 0.4m deep and gradually decrease to where it deposited the lowest plasticity of the soil, theoretical and measured values were found to deposit at 17 and 20 1.5 - 2.5m deep respectively, generating comparable fitting as well as displaying a predictive model equation as $Y = 2.958x^2 – 12.5x + 30.73$ and $R^2 = 0.999$. Figure 4 the rate of plasticity of the soil from both parameters gradually increased and deposits its optimum level at 28, 0.8m deep and suddenly decrease where the lowest level were recorded at 15 2.5m deep, the parameter developed predictive model as $Y = 8.955x^3 – 38.96x^2 + 89.81x + 15.60$ and $R^2 = 0.991$, $Y = 8.787x^3 – 37.01x^2 + 35.79x + 18.49$ and $R^2 = 1$. Figure 5 the theoretical value displayed it rate of plasticity from the lowest level at 20 0.2m deep, in a gradual form to a point where an optimum value were recorded at 25 1.5m deep and slightly decrease down at 2.5m; while measured value maintained similar condition from the same lowest point at 20 in a gradual process to the point where its optimum value were achieved at 20 1.5m deep. In the same vein, it slightly decrease down at 2.5m deep, both parameters developed the same curve fit but with little variation, it developed a predictive model as $Y = 1.075x^2 + 3.806x + 19.29$ and $R^2 = 0.867$, $Y = 1.994x^2 + 6.993x + 18.96$ and $R^2 = 0.999$. Figure 6 both parameters maintained a similar form like figure 5 and produced a predictive model equation as $Y = -2.287x^2 + 5.663x + 17.05$ and $R^2 = 0.983$, while that of measured as $Y = 1.511x^3 – 9.503x^2 + 15.40x + 14.26$ and $R^2 = 0.992$. Figure 7 deposited both parameters the theoretical and measured values from the lowest at 11 0.4m deep and rapidly increase to where an optimum value was recorded at 25 1.0m deep and slightly decrease to 2.5m deep both parameters produced $Y = 5.304x^3 – 28.43x^2 + 45.10x + 3.030$ and $R^2 = 0.961$, $Y = 6.636x^3 – 34.41x^2 + 51.70x + 2.042$ and $R^2 = 1$. Figure 8 produced its rate of plasticity from the lowest 13 0.4m deep, in a gradual process, to the point were an optimum were recorded at 25, 2.5m deep, the measured displayed some fluctuations from 0.2 – 1.0m deep producing some variations, both parameters generated a predictive model as $Y = -3.099x^3 + 13.11x^2 + 11.67x$ and $R^2 = 1$, $Y = -3.240x^2 + 13.69x + 11.14$ and $R^2 = 0.920$. Figure 9 deposited the lowest 11 and 12 at 0.4m deep and rapidly increased in depth to where an optimum value was recorded at 26, 1.0m deep, finally decreased down a little at 22 2.5m deep. It produced a predictive model equation $Y = 5.304x^3 – 28.43x^2 + 45.10x + 3.030$ and $R^2 = 0.961$, $Y = 5.307x^3 – 28.44x^2 + 44.11x + 3.022$ and $R^2 = 1$. All the locations produced a theoretical model results that compared faviourably well with the measured values from other locations in the study. This implies that the model developed for plastic limit can be applied in the design and construction of roads in the study area.

CONCLUSION

The predictive model developed for plastic limit of soil in the study location compared faviourably well with the measured values. These values are between medium and high plasticity of soil in the study location, base on classification for fine grain soil. The predictive model can be applied as established parameter that can be integrated in construction and design of roads in the study area. The plastic limit of the earth material (sub base) used to produce the predictive models will be effective in maintaining the life span of roads if it is integrated in design. This paper is imperative because there are lots of variations in the plasticity of the soil deposited in the study location. Hence, the predictive model developed will definitely solve problems of inadequate design of roads in the study area.

REFERENCES

[1] AASHTO (**1993**), Standard Specification for Transportation materials and methods of sampling and testing (Twenty first edition **2001** (Part 1)).
[2] AASHTO (**1993**), Standard Specification for Transportation materials and methods of sampling and testing (Twenty first edition **2001** (Part 2)).
[3] Awoke Sime (**2005**), Further investigation of road failures constructed on expansive soils of central Ethiopia Addis Ababa-Jimma road as a case study. A masters thesis, Addis Ababa University.
[4] ERA (**2002**), Pavement Design Manual; Flexible Pavements and Gravel Roads, Volume 1
[5] ERA (**2002**), Pavement Rehabilitation and Asphalt Overlay Manual VOL 1
[6] ERA (**2002**), Site Investigation Manual VOL 1
[7] Nunn, M.E., Brown, A., Weston, D., and Nicholls, J.C.. Design of Long Life Flexible Pavements for Heavy Traffic, TRL report 250, **1997**, TRL Limited, London, ISSN 0968-4107

[8] GLASS J., (**2005**). Civil engineering sustainable solutions using concrete. The Concrete Centre, Camberley, pp 5-7.
[9] Gilbert, P. (**2004**). Practical developments for pavement foundation specification, Proceedings of the International Seminar on Geotechnics in Pavement and Railway Design and Construction, Gomes Correia & Loizos (eds), Millpress, Rotterdam, pp. 157-165, ISBN 90 5966 038 2
[10] Design Manual for Roads and Bridges (**1991**) Earthworks – design and preparations of contract documents. Volume 4, Section 1, Part 1, **1991**, HMSO,
[11] London, UK Design Manual for Roads and Bridges (1994) Foundations. Design manual for roads and bridges. Volume 7, Section 2, Part 2, **1994**, HMSO, London, UK
[12] Design Manual for Roads and Bridges (1999). Maintenance assessment procedure. Design manual for roads and bridges. Volume 7, Section 3, Part 3, **1999**, HMSO, London, UK
[13] Design Manual for Roads and Bridges (**2004**). Conservation and the use of secondary and recycled materials. Design manual for roads and bridges. Volume 7, Section 1, Part 2, **2004**, HMSO, London, UK
[14] Design Manual for Roads and Bridges (**2006**). Pavement Design. Design manual for roads and bridges. Volume 7, Section 2, Part 3, **2006**, HMSO, London, UK