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NON-DESTRUCTIVE ASSESSMENT OF BEARING RESISTANCE OF LOCAL SUBGRADE SOILS THROUGH LIGHT WEIGHT DEFLECTOMETER MEASUREMENTS

A Thesis Submitted to the Department of Civil Engineering, University of Kerbala in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Infrastructure Engineering)

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بسم الله الرحمن الرحيم

يرفع الله الذين آمنوا منكم والذين أوتوا العلم درجات

صدق الله العلي العظيم

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ABSTRACT

In highway and airport pavement design, subgrade strength mostly affected the thickness of pavement layers, California Bearing Ratio (CBR) is one of the simple empirical methods that is commonly used to determine the strength and stiffness of subgrade soil, subbase, and base course material in airfield and highways pavement system layers. The CBR test is performed originally to determine the thicknesses of flexible pavements layers by using empirical techniques. The test is normally conducted on compacted remolded specimens and may be performed on undisturbed soils or soils in the field expressed by field, in-place or in-situ CBR test. In-situ CBR test is intended to determine static properties of unbound pavement layers represented by bearing capacity of subgrade soils without requiring the digging of test pits. However, as the testing procedure of CBR test is laborious and time consuming beside the development in testing techniques, methods are proposed for correlating CBR value with dynamic measurements such as those obtained from Light Weight Deflectometer (LWD).

In this work, two testing methods were used on three types of subgrade soils: static test method represented by the in-situ CBR to evaluate strength parameters, and dynamic test method which was implemented using LWD device to estimate the dynamic properties of subgrade soils. Three roadway projects were selected at Karbala city to evaluate different subgrade soils. The sites were Al-Meelad district, Al-Fares district, and Al-Rofaee zone at which their soils' type were classified as A-1-b, A-3, and A-7-6, respectively.

Statistical analyses were performed to predict the bearing resistance of subgrade soils based on LWD dynamic measurements such as dynamic modulus and surface deflection, in addition to the water content and dry density obtained from field density tests methods. The results of six statistical models for two subgrade groups (i.e., granular soil and clayey soil) indicate that the LWD has a significant potential for compaction quality control and for determining in-situ dynamic moduli of pavement layers during construction and surface deflections. In addition, the results indicate that the dynamic modulus from LWD is well correlated with the bearing ratio for various subgrade soils. Furthermore, basic physical properties from field density tests has significant correlation with CBR value. A comparison between dynamic measurements and physical properties indicate the dynamic modulus is most significant than field density.

For verifying the statistical models, two under construction sites were selected at Karbala city: Al-Takahe district, and Al-Emam Ali district. The subgrade soil of selected sites was classified according to AASHTO to as A-1-b and A-3 at Al-Takahe district, and Al-Emam Ali district, respectively. The initial results showed the ability to predict bearing resistance of subgrades based on their basic physical properties and dynamic measurement obtained by LWD test device.

SUPERVISOR CERTIFICATE

We certify that this thesis entitled “NON-DESTRUCTIVE ASSESSMENT OF BEARING RESISTANCE OF LOCAL SUBGRADE SOILS THROUGH LIGHT WEIGHT DEFLECTOMETER MEASUREMENTS”, which is prepared by Engineer "Rawq Mohammed Abd Ali Al-Fatteh allah ", is under our supervision at the University of Kerbala in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Infrastructure Engineering).

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LINGUISTIC CERTIFICATE

I certify that this thesis entitled "NON-DESTRUCTIVE ASSESSMENT OF BEARING RESISTANCE OF LOCAL SUBGRADE SOILS THROUGH LIGHT WEIGHT DEFLECTOMETER MEASUREMENTS" which is prepared by Rawq Mohammed Abd Ali under my linguistic supervision. It was amended to meet the English style.

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EXAMININATION COMMITTEE CERTIFICATION

We certify that this thesis entitled "NON-DESTRUCTIVE ASSESSMENT OF BEARING RESISTANCE OF LOCAL SUBGRADE SOILS THROUGH LIGHT WEIGHT DEFLECTOMETER MEASUREMENTS" and as an examining committee, we examined Engineer "Rawq Mohammed Abd Ali" in its content and in what is connected with it, and that in our opinion it is adequate as a thesis for degree of Master of Science in Civil Engineering (Infrastructure Engineering).

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DEDICATION

This thesis is dedicated to my parents and my family, my friends for their love and continuous prayers

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Praise and Glory be to Almighty ALLAH for bestowing me with health and power to complete this work and to achieve something that might serve humanity. I would like to thank my parents for their continuous support to complete my research work.

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CONTENTS

LIST OF FIGURES

AASHTO	American Association of State Highway and Transportation Officials.
AISC	American Institute of Steel Construction and European standard.
AL_2O_3	Aluminum Oxide.
ANOVA	Analysis of Variance.
ARRB	Australian Road Research Board.
ASTM	American Society for Testing and Materials.
CaO	Calcium Oxide.
$CaSO_4\ 2H_2O$	Gypsum.
CBR	California Bearing Ratio.
CCM	Core Cutter Method.
CIH	Clegg Impact Hammer.
CIV	Clegg Impact Values.
CH	High Plasticity Inorganic Clay.
CSIR	South African Council on Scientific and Industrial Research.
C_c	Coefficient of Curvature.
C_u	Coefficient of Uniformity.
D_c	Degree of Compatibility.
DCP	Dynamic Cone Penetration.
DCPI	Dynamic Cone Penetration Index.
D_{oc}	Degree of Compaction.
E	Modulus of Elasticity.
E_d	Dynamic Modulus.
E_{LWD}	Modulus of Resilience by Using The Light Weight Deflectometer.
Epfwd	Modulus of Resilience by Using The Portable Falling Weight Deflectometer.
Fe_2O_3	Iron Oxide.
FWD	Falling Weight Deflectometer.
GC	Gravel With Inorganic Clay.
GM	Gravel With Inorganic Silt.
GP	Poorly Graded Gravel.
GS	Specific Gravity.
GW	Well Graded Gravel.
k_s	Modulus of Subgrade Reaction.
LL	Liquid Limit.
LoS	Loss on Ignition.
LVDT	Linear Variable Differential Transformer.

LWD	Light Weight Deflectometer.
MDD	Maximum Dry Density.
MH	High plasticity inorganic silt.
MI	Silt Of Intermmediate Compressibility.
ML	Low Plasticity Inorganic Silt.
MLRA	Multiple Linear Regression Analysis.
MPMT	Miniaturized Pressure Meter Test.
Mr	Resilient Modulus.
NAASRA	National Association of Australian State Road Authorities.
NCDOT	North Carolina DOT .
NDT	Non Destructive Test .
NOP	Number of Passes.
NRRL	Norwegian Road Research Laboratory.
OH	High Plasticity Organic Silt or Clay.
OL	Low Plasticity Organic Silt or Clay.
OMC	Optimum Moisture Content.
PFWD	Portable Falling Weight Deflectometer.
PI	Plasticity Index.
PL	Plastic Limit.
PLT	Plate Loading Test.
PPMT	Pencel Pressuremeter Test.
RMSE	Root Mean Square Error .
SC	Sandy Soil with Inorganic Clay.
SDG	Soil Density Gauge.
S_d	Surface Deflection.
SiO_2	Silicon Dioxide.
SLRA	Simple Linear Regression Analysis.
SM	Sandy Soil with Inorganic Silt.
SO_3	Sulfur Trioxide.
SP	Poorly Graded Sand.
SPSS	Statistical Package For The Social Science.
SRM	Sand Replacement Method.
SSE	Residual Sum of Square.
SSG	Soil Stiffness Gauge.
SSR	Regression Sum of Square.

SW	Well Graded Sand.
TACH-MD	Recently Proposed Mechanistic-Empirical Methods of Design
TRRL	Transportation and Road Research Laboratory.
UoK	University of Kerbala.
UPE	U Channels with Parallel Flange and Universal I Beam IPE.
USACE	US Army Corps of Engineers.
W_c	Water Content.

Chapter One

INTRODUCTION

1.1 Background

An essential design parameter that needs to be considered in the design of highway and airport pavements is the characteristic of the subgrade where the pavement is placed on. Subgrade soils are typically characterized by their ability to resist a deformation under load, which can be either a measure of their strength or stiffness. In general, more subgrade resistant to deformation could reflect more load carrying capacity before reaching a critical deformation value. Weakness in the subgrade, wetting, and poor drainage are main important parameters causes failures in the subgrade layer, hence in the pavement structure, see Figure (1.1).

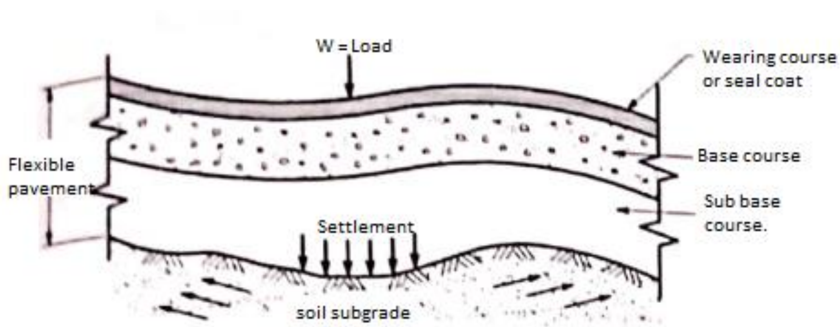


Figure 1.1: Subgrade failure of flexible pavement.

A pavement system is conventionally composed of several well compacted hard courses or layers which are constructed over a natural subgrade soil. Structural performance of this system depends essentially on strength characteristics of the granular materials, which can be indexed in terms of stiffness modulus, density, California Bearing Ratio (CBR), etc. The standard CBR test can be carried out in the laboratory or on site under standard test methods, namely ASTM D1883, and ASTM D4429, respectively, and the correlation can be used for the determination of soil subgrade bearing resistance.

However, many researchers were developed correlative models to associate CBR with other tests and devices to estimate the strength and stiffness of subgrades. Abu-Farsakh et al.(2004) developed a correlation between the stiffness modulus values determined using the Geo-gauge and the CBR, George et al.(2009) developed correlations between the CBR values, dynamic cone penetration (DCP) observations, dynamic cone penetration index (DCPI) and the modulus of resilience using the portable falling weight deflectometer (PFWD), George and Kumar (2018) developed correlation between CBR value and resilient modulus determined by PFWD and cyclic tri-axial devices, Pattison et al.(2010) developed a new correlative model that relates the CBR to 20 kg Clegg impact values (CIV) which was developed entirely from field data.

Although the resilient modulus (M_r) of the subgrade is a very important factor in a modern pavement design methods and evaluation process. Considering the costly devices measurement requirements and the complexity of the test, it is desirable to develop approximate methods for the evaluation of M_r commonly, this parameter is estimated using simple empirical relationships with CBR values (Heukelom and Klomp, 1962), (Green and Hall, 1975).

Light Weight Deflectometer (LWD) is a portable and nondestructive device which is used for compaction quality control of soil layers in highway construction by determining the deflection (δ) in flexible pavement layers, LWD can also be used to determine the stiffness of surface soils. The method of calculation LWD modulus (E_{LWD}) was based on the Boussineq elastic half space theory (Fleming et al., 2009). The LWD is gaining acceptance and popularity over the years, and the pavement and transportation geotechnics community are currently moving toward more mechanistic based design and quality control evaluation of pavement layers and fill embankment. The developments in testing methods such as (LWD) display more precise, ease of use and portability, fast, non distructive and modern method, in addition to the surface deflection and surface stiffness of compacted soil can be also known (Chen, 2014).

1.2 **Problem Statement**

The strength and stiffness of pavement and subgrade soil layers are main parameters for the most pavement layers design and analyses methods. A pavement structure system is essentially consisting of a flexible or rigid surface layer which is constructed on several unbound granular layers, and subgrade soils. The characteristics of subgrade affect mainly on the structural performance of the pavement system. In general, subgrade layer presents primarily as a platform to support others pavement layers. For evaluating stiffness and strength properties of subgrade soils, several laboratories and in-situ testing methods were developed.

Among these test methods was CBR test method which can evaluate the strength of subgrade layers indirectly. However, there are many of essential limitations in using CBR test because this testing method is complicated, laborious and time consuming. Since in-situ testing methods provide the ability to determine the strength and modulus of subgrade layer by adopting non-destructive tests such as LWD with less time and effort, so these test methods were adopted in the assessment of soil strengthen and compered with traditional testing methods.

1.3 **Research Aim and Objectives**

The main aim of this research is to predict the bearing resistance of local subgrade soils properties and identify their strength and stiffness using the modern, simple and reliable testing procedure. This will be achieved through the following objectives:

- Evaluating selected local subgrade soils, and identifying their engineering and physical properties.
- Using the lightweight deflectometer test to predict the bearing capacity of subgrade soil by obtaining dynamic measurements such as dynamic modulus.
- Correlating the dynamic measurements from LWD with conventional subgrade soils properties such as California bearing ratio (CBR).

1.4 Scope of the Research Work

Within the wide range of conditions, materials, testing methods, and design methods, this research work was achieved under the following scope:

1- All subgrade soils used in this research were collected from local project sites at Karbala city. However, for the limiting time and resources, only three soil types were used.

2- Selected subgrade soils were evaluated in the lab in terms of basic physical and chemical properties, and laboratory testing methods.

3- All test methods were conducted according to standard specifications.

4- All results obtained from this research work were analyzed by the statistical program.

5- Sites evaluation was conducted during research work. Only two sites were nominated to validate the developed models due to lack of time and resources.

6- All tests were performed at the University of Kerbala (UoK) laboratories.

Laboratory testing setup locally manufactured, including loading frame system, steel box and data acquisition system. The testing setup apparatus was considered the first device that was designed by the University of Kerbala to provide a similar environment for sites and conducting field tests. The steel box was designed with dimensions (240*120*125) cm, so that the soil sample can be tested in this research work by weight of (3) tons approximately, also the loading system can apply up to (20) ton. Moreover, the data acquisition system was programmed and computerized locally with the help of an experienced programmer.

1.5 Thesis Structure

The thesis consists of six chapters to demonstrate the study work outcomes as listed below:

Chapter 1 Introduces the background of the research, problem statement, aim and objectives, scope of the research work, and finally the thesis layout.

Chapter 2 Reviews previous studies for correlating the CBR test and LWD device to other test methods and devices, mechanical properties for subgrade soil.

Chapter 3 Describes soils types and their locations that are used in this research, physical and chemical properties of selected types of soil, the testing device, laboratory tests to examine the selected soil, and finally research methodology,.

Chapter 4 Illustrates the results of the laboratory tests including field density test by using two test methods, CBR test, and LWD test for a selected type of soils.

Chapter 5 Discloses statistical relationships between CBR value with the parameters LWD test and main basic physical properties for subgrade soil.

Chapter 6 Describes the field sites and characterizing their soils types, adopted physical and laboratory tests to examine the selected soil. finally, employment the statistical models to obtained CBR values.

Chapter 7 Demonstrates the main conclusions, recommendations included the suggestion future development studies to enhance strength and stiffness of the pavement system.

Chapter Two

LITERATURE REVIEW

2.1 Introduction

Subgrades provide structural stability to pavements layers by transmitting traffic loads safely to the soil layers below. The subgrade strength evaluation plays a major effect in the design of pavement layers since superimposed traffic loads need to be transmitted in a manner in which the subgrade deformation is within elastic limits and the shear forces developed are within safe limits under adverse climatic and loading conditions. In this connection, subgrades need to be evaluated for the soil stiffness and strength.

In addition, to this, an investigation into the influence of the soil properties of the subgrade on pavement performance is also essential. In a number of circumstances, road engineers come across situations where subgrades need to be strengthened in order to improve their load carrying characteristics. Traditionally, flexible pavements are designed based on the California bearing ratio (CBR) approach or be considering elastic deformations as in the case of (Burmister, 1958) layer theory.

The CBR method of analysis gives more importance to the estimation of the strength of the subgrades and the pavement layers, while the analysis based on quality control of pavements relies more on the determination of in-situ density and moisture content. (Varghese et al., 2009). The objective of this chapter is to provide a background about conventional bearing resistance tests and tests devices that respond to provide strength and stiffness of soil measurements and summarizes their theoretical and statistical models were developed by numerous researchers.

2.2 Mechanical Properties of Unbounded Pavement Materials

Quality of design pavement foundation layers is critical for achieving excellent pavement performance. Strength and stiffness of the soil are considered as essential and relevant engineering and mechanical properties in both design and construction of earthworks, while soil density and water content are necessary physical measurements during the construction process. However, soils prepared at the same density and water content may have different stiffness and strength, which are dependent on several factors, including the state of stress, strain level, boundary condition, and fabric of the soil (Hikouei at el., 2016).

Several research centres in Europe and North America have suggested that flexible pavement design could be undertaken using a structural design procedure (Kirwan & Snaith, 1976). For such a procedure, it is necessary to obtain a value for the Poisson's ratio and resilient modulus for each layer of the pavement structure. The resilient modulus (M_r) is defined as the quotient of repeated axial (dynamic) deviator stress (σ_d) in triaxial compression to the recoverable (resilient) axial strain (ε_a) measured between successive applications of stress:

$$M_r = \frac{\sigma_d}{\varepsilon_a} \quad \text{--} \quad (2.1)$$

where $\sigma_d = \sigma_1 - \sigma_3$ (where σ_1 is the total axial stress or defined as the major principal stress, and σ_3 is the total radial stress or defined as the minor principal stress) while ε_a represent the recoverable or elastic strain.

M_r values were determined either directly from laboratory testing, or indirectly through correlation with laboratory, field tests, or back calculated from deflection measurements. The procedure testing requirement to determine of M_r consists of applied repeated deviator stress (σ_d), with a constant cell pressure to measure resilient axial strain. Due to the applying repeated load tests, can be observed the number of load cycles will be increased, and will be caused to increase the secant modulus. After a number of load cycles, the modulus becomes nearly constant, and the response can be presumed to be elastic. This steady value of modulus is defined as the resilient modulus (Rahim, 2005).

Since 1960, numerous researchers devoted their efforts to characterize the resilient behavior of granular materials. It is well known that granular pavement layers show a nonlinear and time-dependent elastoplastic response under traffic loading (Lekarp at el., 2000). To deal with this nonlinearity and to differentiate from the traditional elasticity theories, the resilient response of granular materials is usually defined by resilient modulus and Poisson's ratio. Alternatively, the use of shear and bulk moduli was suggested. For design purposes, it is important to consider how resilient behavior varies with changes in different influencing factors. From literature, it appears that the resilient behavior of unbound granular materials may be affected with varying degrees of importance by several factors as listed below:

1- Impact of density and moisture content.
2- Impact of grading, fines content, and maximum grain size.
3- Impact of stress history and number of load cycles studies.
4- Impact of load duration, frequency, and load sequence.

2.3 California Bearing Ratio (CBR)

The CBR test was originally developed by the California State Highway Department and was thereafter incorporated by the Army Corps of Engineers for the design of flexible pavements. It has become so globally popular that it is incorporated in many international standards. The significance of the CBR test originated from the following two facts:

- For almost all pavement design charts, unbound materials are basically characterized in terms of their CBR values when they are compacted in pavement layers.
- The CBR value has been was correlated with some fundamental properties of soils, such as plasticity indices, grain-size distribution, bearing capacity, modulus of subgrade reaction, modulus of resilience, density, and moisture content.

The CBR of a soil is the ratio obtained by dividing the stress required to cause a standard piston to penetrate 2.54, 5.08, 7.62, 10.16, and 12.70 mm into the soil by standard penetration stress at each depth of penetration (ASTM D1883, 2014). Some general classifications of soils as subgrade, subbase and base courses corresponding to various ranges of CBR values are given in Table (2.1) (Asphalt Institute,1962, Bowles, 1978, Hazirbaba, 2018).

Table 2.1: General ratings of soils for roads and runways corresponding to various ranges of CBR values (Asphalt Institute, 1962, Bowels, 1978).

CBR%	**General rating**	**Uses**	**Classification system**	
			USCS	**AASHTO**
0-3	Very poor	Subgrade	OH, CH, MH, OL	A-5, A-6, A-7
3-7	Poor to fair	Subgrade	OH, CH, MH, OL	A-4, A-5, A-6, A-7
7-20	Fair	Subbase	OL, CL, ML, SC, SM, SP	A-2, A-4, A-6, A-7
20-50	Good	Base, subbase	GM, GC, SW, SM, SP, GP	A-1-b, A-2-5, A3, A-2-6
>50	excellent	Base	GW, GM	A-1-a, A-2-4, A-3

The CBR test is an economical and simple tool for measuring strength gain and improvement in soils. CBR tests in this study were performed in accordance with ASTM D1883 and ASTM D4429, for both laboratory and in-situ CBR tests. In order to assess the structural properties of the pavement subgrade, there are different correlations were suggested between the CBR test and properties and test devices of the pavement subgrade, as shall be presented in the next subsections:

2.3.1 Correlation of CBR and Physical Properties of Soil

The CBR value of soil depends on many factors such as soil physical characteristics like maximum dry density (MDD), optimum moisture content (OMC), liquid limit (LL), plastic limit (PL), plasticity index (PI), grain size distribution, permeability of soil etc., and testing condition like soaked or unsoaked soil condition (Shirur and Hiremath, 2014) , (Talukdar, 2014), (Samson and Ibrahim, 2017). Correlation was tried extensively by many research works for estimating bearing resistance of soils from its physical properties.

Based on experimental results and simple linear regression analysis (SLRA) of subgrade soils with an average liquid limit (20% to 70%), there is no significant relation exists to predict CBR value from liquid limit and plastic limit, but there is a good relation obtained by SLRA to predict CBR value from MDD and OMC (Shirur and Hiremath, 2014, Chandra et al., 2017):

$$CBR=4.99\ MDD-5.711 \qquad (R^2=0.78) \qquad \text{------------------------} \qquad (2.2)$$

$$CBR=-0.2443\ OMC+7.5264 \qquad (R^2=0.71) \qquad \text{------------------------} \qquad (2.3)$$

While CBR value of fine grained silty soil of low compressibility (ML) and silts of intermediate compressibility (MI) bears significant correlation with PI, MDD and OMC, and observed CBR value decreases with the increase in the plasticity index and optimum moisture content of soil but increases with the increase in the maximum dry density (Shirur and Hiremath, 2014), and (Chandra et al., 2017) and there is a slight difference between the CBR value determined in the laboratory and computed by using multiple linear regression model involving LL, PL, PI, MDD and OMC (Talukdar, 2014) :

$$CBR_{soaked} = 0.127(LL) + 0.00\ (PL) - 0.1598(PI) + 1.405\ (MDD) - 0.259\ (OMC) + 4.618 \qquad (2.4)$$

There is a suggested other empirical relation obtained from multiple linear regression analysis (MLRA) of fine grained soils shows good relation to predicting CBR value from a combination of MDD and OMC (Talukdar, 2014):

CBR= -4.8353–1.56856(OMC) +4.6351(MDD) (R^2=0.82) ------------------ (2.5)

Based on experimental results and simple linear regression analysis (SLRA), there is no significant relation exists to predict CBR value from liquid limit and plastic limit (Shirur and Hiremath, 2014), and (Chandra et al., 2017) , but there is a slight difference between the CBR value determined in the laboratory and computed by using multiple linear regression model involving LL, PL, PI, MDD and OMC (Talukdar, 2014).

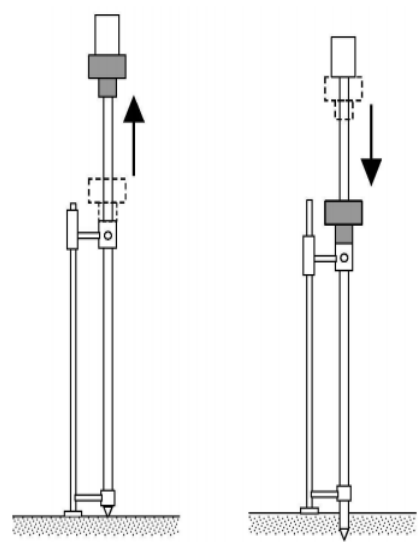
2.3.2 **Correlation of CBR and Dynamic Cone Penetration (DCP)**

In recent years, in-situ techniques have become widely used by geotechnical and pavement engineers due to their simplicity and low cost of operation. This new device and technique shows in Figure (2.1a,b) and provides the density, moisture assessment of pavement materials and in-situ strength and stiffness which are direct structural parameters for determining load support capacity and deformation characteristic in engineering design (Mahamid, 2013), (Amadi et al., 2018). Moreover, adopting the in-situ technique as part of the quality control program will provide more data points which may help to enhance the construction quality and compliance. Examples of these techniques include Light Weight Deflectometer (LWD), Portable Falling Weight Deflectometer (PFWD), Soil Stiffness Gauge (SSG), Dynamic Cone Penetrometer (DCP), Soil Density Gauge (SDG) amongst others (A A Amadi et al., 2018).

The DCP device was developed in South Africa for evaluation of in-situ pavement layers strength in the 1960s. Different variations of the DCP was developed and adopted for nearly half a century to estimate soil strength parameters. Vuuren (1969) developed original design had a 30° cone. Kleyn et al. (1982) developed another DCP design that conducted a 60° cone tip, 8-kg hammer, and 575-mm free fall. This design was then gradually adopted by countries around the world and became the design specified in the standard test method ASTM D6951, 2009 (MejiasSantiago et al., 2015). Kleyn et al. (1982) established that DCP testing was highly repeatable and sensitive enough for use in practice for the design of pavements, evaluation of pavement, and sublayers assessing earthwork construction quality.

Assessment of structural properties of the pavement layers by using of DCP test that required the development of reliable correlations with conventional methods such as the CBR test. A number of researchers were performed the development of empirical relationship between dynamic cone penetration resistance (DCPI) and CBR measurements (E.G. Kleyn, 1975), (Smith and Pratt, 1983), (Harrison, 1986), (Livneh and Ishai, 1987), and (M. Livneh and N.A. Livneh, 1994). According to the results of past studies, Numerous correlations were developed between the DCP test results and CBR values. Table (2.2) summarizes the theoretical models that were developed for various soil types.

a) *Automated DCP apparatus*



b) Basic DCP operational mechanism.

Figure 2.1: Dynamic cone penetration test device.

Table 2.2 : Summary of correlations between CBR and DCPI adapted by (Shaban, 2016) & (Thach Nguyen and Mohajerani, 2017).

Proposed relationship	**References**
Log CBR = 2.620 - 1.270 log (DCPI*)	(Kleyn, 1975)
Log CBR = 2.555 - 1.450 log (DCPI)	(Smith and Pratt, 1983)
Log CBR = 2.640 − 1.080 log (PR**)	(North Carolina DOT, 1987)
Log CBR = 2.20 − 0.7 $[\log (PR)]^{1.5}$	(Livneh et al, 1987)
Log CBR = 2.560 - 1.160 log (DCPI)	(Harison, 1989)
log CBR = 2.465 − 1.12 log (PR)	(USACE, 1992)
Log CBR = 2.450 - 1.120 log (DCPI)	(Livneh et al, 1992)
CBR = $292/PR^{1.12}$	(Webster et al., 1994)
log CBR = 2.669 − 1.065 log (PR)	(Ese at al, 1994)
Log CBR = 2.530 - 1.140 log (DCPI)	(Coonse, 1999)
Log CBR = 1.550 – 0.550 log (DCPI)	(Gabr et al., 2000)
CBR =2559.44/ [(DCPI1.84 − 7.35) − 1.41]	(Nazzal, 2003)
CBR = $1,161.1/PR^{1.52}$	(Abu-Farsakh et al., 2005)
Log CBR = 67.898 − 17.483 ln (PR)	(Sahoo & Reddy, 2009)
Log CBR = 2.51- 1.074 log (DCPI)	(A A Amadi *et al.*, 2018)
CBR = 47.32 DCPI−0.7852	(George and Kumar, 2018)

where:
**DCPI is the dynamic cone pentration index in mm/blow (Amadi et al., 2018).*
*** PR is the penetration through the layer in millimeters (Benedetto, Tosti and Domenico, 2012).*

The relationship for developed models which proposed by various researches and agencies were shown in Figure (2.2).

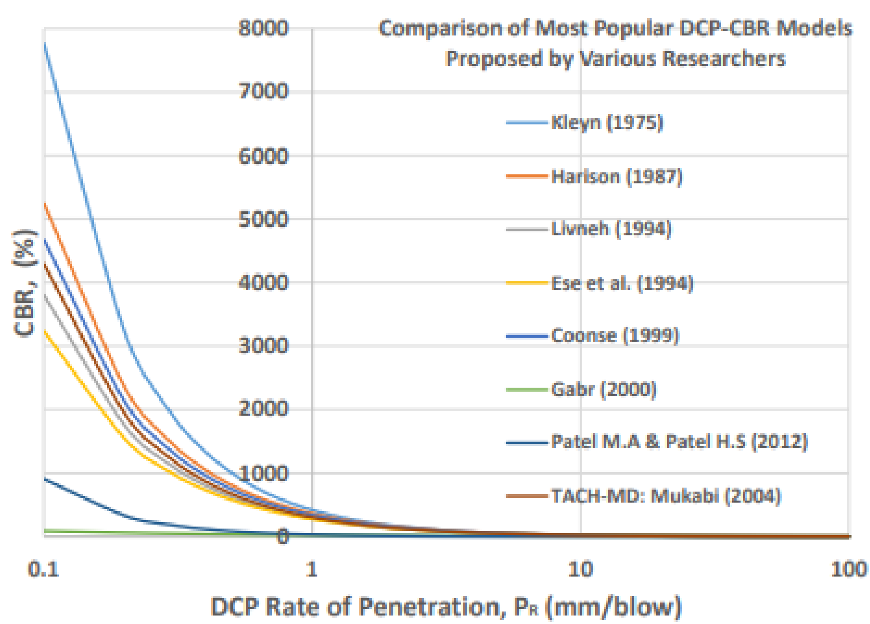


Figure (2.2a): Comparison of most popular DCP-CBR models proposed by various Researchers

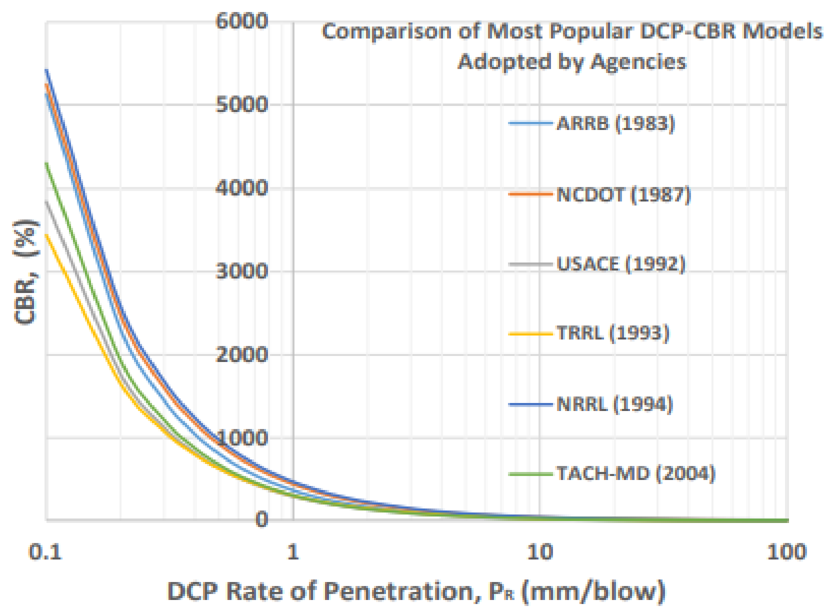


Figure (2.2b): Comparison of most popular DCP-CBR models proposed by various Agencies.

Figure 2.2: Relationship between DCPI and laboratory CBR adapted by (J. Mukabi, 2017).

2.3.3 **Correlation of CBR and Portable Falling Weight Deflectometer (PFWD)**

The stiffness and strength of subgrade have a vital role in identifying a stability of pavements structure because of the heavy vehicular traffic causes stresses on pavements which are transferred repeatedly to the subgrade layer, by others layers of the pavements structure. The pavement must be designed according to the transmitting stresses to the subgrade layers are within the elastic limits. Higher traffic loads can be resisted by using stronger subgrades layers and need to be constructed with a thicker base and subbase courses layers. Hence, a proper understanding of the subgrade soil properties consider an important factor that response on the selection of materials to be used for base and subbase courses.

Conventional method tests for the determination of the stiffness and strength of soil subgrades such as California Bearing Ratio (CBR) test, cyclic triaxial test, and plate loading test (PLT). However, these methods are tedious, time consuming labor-intensive, and not easily portable. due to the simplicity in operation, and portability for the determination of the modulus of stiffness (E) or modulus of resilience(M_r) in a more reliable manner ,the using of non destructive testing devices include the (FWD) and (PFWD) have acquired popularity in the testing of soil subgrades (Livneh, 1997), (Fleming, 2000), (Moshe and Yair , 2001), (Kim et al., 2007).

These in situ testing devices were initially developed in Germany as an alternative to the plate load test and now used extensively in Europe and Japan (Nazzal, 2003). These devices impart an impact or vibratory load to the surface with the applied force, and the induced pavement surface deflections can be simultaneously monitored (Hoffmann et al., 2004), (Lin et al., 2006). Several works were conducted in the last decade to assess PFWD measurements, to evaluate the influence of some relevant parameters such as temperature, moisture content, grading and compaction, or to correlate the PFWD modulus with other test results such as FWD and CBR.

Correlations between PFWD results with CBR values was done by several researchers, Kavussi et al. (2010), George et al. (2009), and Nazzal (2003) showed that there is good correlation between these parameters; whereas, Phillips (2005) and Seyman (2003) showed that there is a poor correlation was reported in some other research works. To cite a few of these studies, Table (2.3) summarizes the theoretical models that were recently developed, and the relation between the CBR value and E_{PFWD} was adapted by (George and Kumar, 2018) shows in Figure (2.3).

Table 2.3 : Summary of correlations between CBR and PFWD.

Proposed relationship	References
$CBR = -14.0 + 0.66\ (E_{PFWD}*)$	Nazzal (2003)
$CBR = -5.58 + 0.484\ (E_{PFWD})$	Kavussi et al., (2010)
$CBR = 0.194\ (E_{PFWD}) + 2.784$	George and Kumar (2018)

** where Epfwd is elastic modulus in Mpa.*

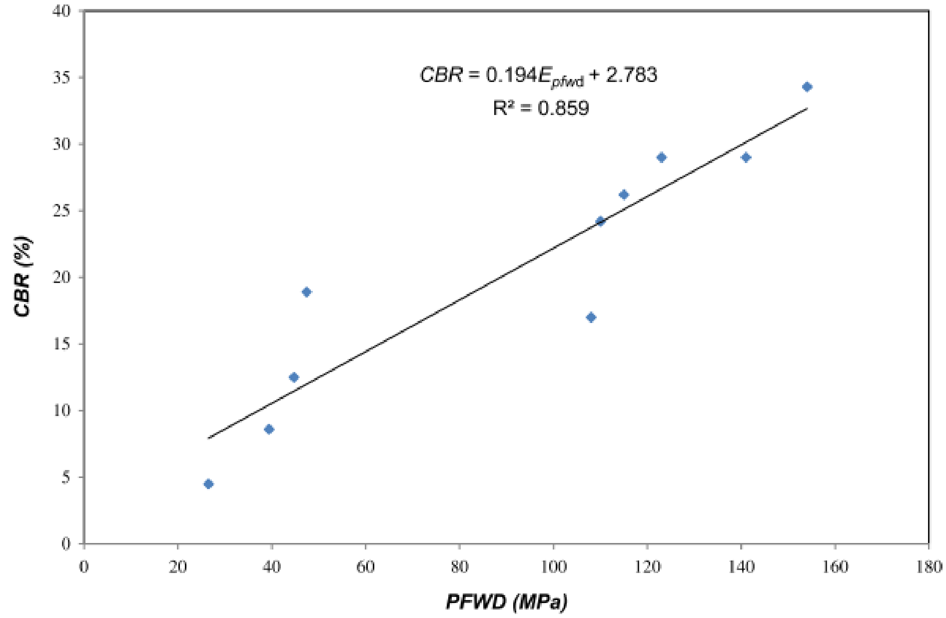


Figure 2.3: Relation between the CBR value and EPFWD was adapted by George and Kumar (2018).

2.3.4 **Correlations of CBR and Clegg Impact Hammer (CIH)**

Development of tools such as the Clegg impact hammer allows for rapid testing of the subgrade during construction, and Clegg Impact Values (CIV) were correlated with more soil test values such as the California Bearing Ratio (CBR) that are often used for road design. The Clegg impact hammer was developed in Australian the 1970s (Pattison et al, 2010). This method is claimed as a possible alternative to the CBR test, because it may practically be performed in both the field and laboratory. Further, the hammer tester provides quick, an easy to operate, and portable device as well as a cost effective means of process control by monitoring the effect of roller passes and checking the variability of field compaction easily. Clegg (1976) proposed to use 4.5-kg Clegg hammer to obtained the CBR value of soil from a CIV, Other empirical equations proposed by Al-Amoudi et al. (2002), Alkire (1987), and Mathur and Coghlan (1987), are shown in Table (2.4).

Table 2.4: Summary of correlations between CBR and CIV.

Proposed relationship	References
$CBR = 0.07 \times (CIV_{4.5kg})^{2}$	Clegg (1976)
$CBR = 0.224 \times (CIV_{4.5kg})^{1.67}$	Alkire (1987)
$CBR = 0.069 \times (CIV_{4.5kg})^{2}$	Mathur and Coghlan (1987)
$CBR = 0.19 \times (CIV_{4.5kg})^{1.535}$	Al-Amoudi et al. (2002) -Lab tests
$CBR = 1.35 \times (CIV_{4.5kg})^{1.0115}$	Al-Amoudi et al. (2002) -Field tests – GM & SM soils
$\text{Log } CBR = \log(-0.128) + 1.26 \log CIV_{20kg}$	Pattison et al. (2010)

Mathur and Coghlan (1987) and Alkire (1987) selected the CBR value versus $CIV_{4.5kg}$ as the comparative tests only by using laboratory samples, and illustrated that the correlation between CBR and $CIV_{4.5kg}$ could influence in the soil type and local conditions. While Al-Amoudi et al. (2002) showed test results for each laboratory and field conditions. New correlative model was developed by Pattison et al. (2010) that related the CBR to 20 kg CIV was developed entirely from field data according to standard specification (ASTM standard D 5874, 2007) of the subgrade and aggregate surface and presented that a strong correlation between CBR values and CIV_{20kg} results were existed, and the variance in the correlation is large enough that a lower bound of correlation is necessary for practical field application, see Figure (2.4).

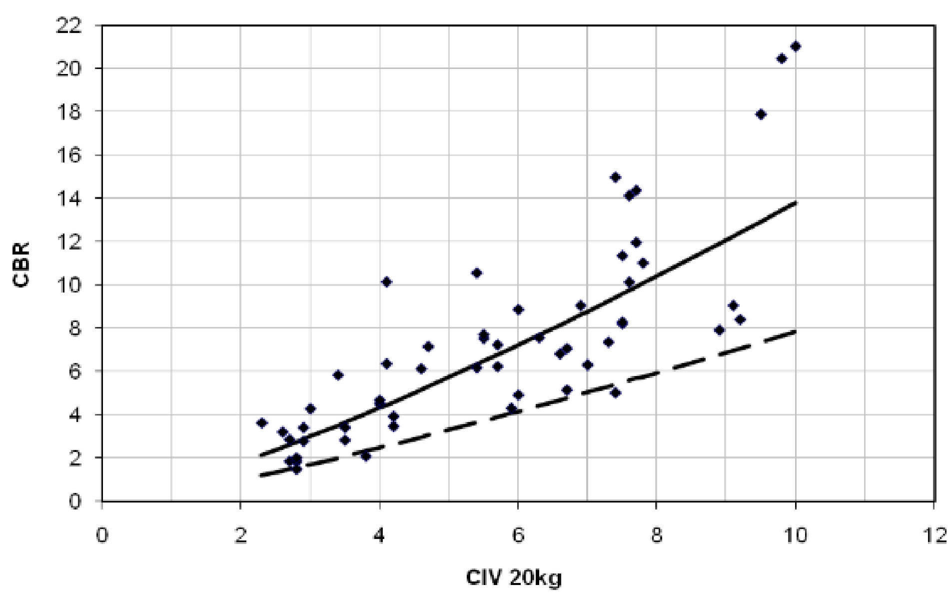


Figure 2.4: CBR - CIV relationship with the back-transformed regression equation and back-transformed lower one-tailed 90% prediction limit, adapted by Pattison et al. (2010).

2.3.5 **Correlation of CBR and Resilient Modulus (M_r).**

The AASHTO Pavement Design Guide (1993) incorporated the resilient modulus (M_r) concept to characterize pavement materials subjected to repeated traffic loads (Rahim, 2005). M_r values may be evaluated directly from laboratory test methods, due to these tests are costly, time consuming, and required a large number of samples to be collected and tested for finding a reliable results. Even then, it is difficult to provide in-situ conditions. AASHTO design guide suggests that agencies concerned in pavement structures design confirm correlations based on standard soil test methods and index properties to obtain M_r design values such as (CBR) test. In the past, there were numerous researchers were discussed the ability of depending CBR test results on pavement design such as Porter (1938,1950), Turnbull (1950), Hight and Stevens (1982) and Fleming and Rogers (1995) whose pointed out that CBR tends to be a bearing value (more of a parameter in terms of strength) rather than a support value (in terms of recoverable behavior) of materials, while Thompson and Robnett (1979) could not find a reliable correlation between CBR and M_r. Hight and Stevens (1982) opined that the CBR does not find a correlation with either stiffness or strength, and Sukumaran (2004) stated that there is an apparent wide variation in the M_r value that can be determined by using the CBR value.

However, on the other hand, Lister and Powell (1987) found that CBR can be related within a reasonable limit to subgrade stiffness. Hossain (2007) believed that CBR test is still one of the most widely used test methods for estimating the competency of unbounded pavement layers. Razouki and Kuttah (2004) opined that a strong linear relationship between the M_r and subgrade CBR developed on the basis of ultrasonic pulse velocity technique. Garg et al. (2009) stated that CBR value have a strong trend in the correlation with the resilient modulus but there is a lot of scattering and pointed that can be converted bearing ratio of subgrade soil to the resilient modulus. Putri et al.(2012) stated that the CBR value can correlate with M_r and gives an advantage to obtain the modulus of subgrade reaction (k_s) used for designing the pavement thickness with more feasibility. Alternatively, others such as Rahim (2005) suggested to estimate M_r for subgrade soils can use subgrade soil index properties.

Carmichael and Stuart (1985) proposed a comprehensive M_r prediction model depending on deviator stress and a number of soil index properties as explanatory parameters. Drumm et al. (1991) improved two regression models for the resilient modulus as a function of deviator stress and soil index properties for fine grained soils. May and Witczak (1985), Uzan (1985), Dai and Zollars (2001), Yau and Von Quintus (2001), Ni et al. (2002) developed constitutive regression equations to correlate M_r to the material stress state. Table (2.5) summarizes the theoretical models that were developed correlations between CBR and M_r.

Table 2.5: Summary of Correlations between CBR and M_r.

Proposed relationship	Reference	Notes
M_r (MPa) = 16.2 CBR $^{0.7}$	NAASRA (1950)	For CBR less than 5
M_r (MPa) = 22.4 CBR $^{0.5}$	NAASRA (1950)	For CBR more than 5
M_r (Psi) = 1500 × CBR	Heukelom and Klomp (1962)	This correlation is only for fine grained non expansive soils with a soaked CBR < 100%
M_r (MPa) = 10.34 × CBR	Heukelom and Klomp (1962)	/
M_r (MPa) = 38 CBR $^{0.711}$	Green, J.L. and Hall (1975)	/
M_r MPa) = 17.58 × CBR $^{0.64}$	Powell et al. (1984)	/
M_r (MPa) = 18 CBR $^{0.64}$	Lister and Powell (1987)	/
M_r (MPa) = 21 CBR $^{0.65}$	Ayres (1997)	/
M_r = 5.00535CBR + 2.95173	Razouki and Kuttah (2004)	For CBR ≥ 1
M_r = 810 CBR (kPa)	Putri et al.(2012)	M_R = 863.82 CBR, ν = 0 M_R = 840.53 CBR, ν = 0.3 M_R = 751 CBR, ν = 0.4
M_r = 8.795(CBR) − 0.972	George and Kumar (2018)	Resilient modulus measured using the cyclic triaxial test

The empirical correlations between modulus of elasticity E with CBR that were worked out by Heukelom and Klomp (1962), NAASRA (1950) and Powell et al. (1984), were presented in

Figure (2.5). While Figure (2.6) shows a comparison of popular Agencies adopted models correlating CBR and M_r adapted by Mukabi (2017).

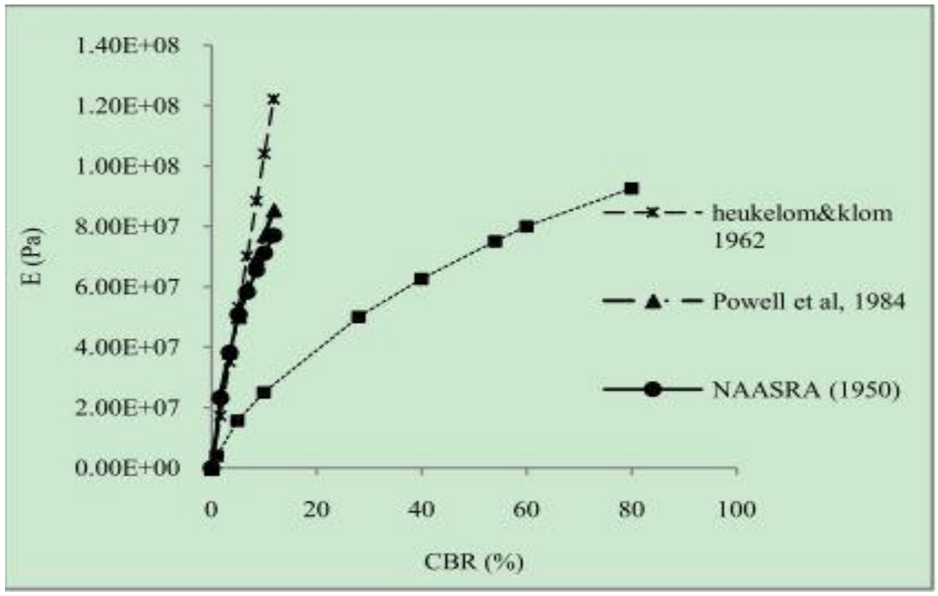


Figure 2.5: California Bearing Ratio versus Modulus of Elasticity adapted by (Putri et al, 2012).

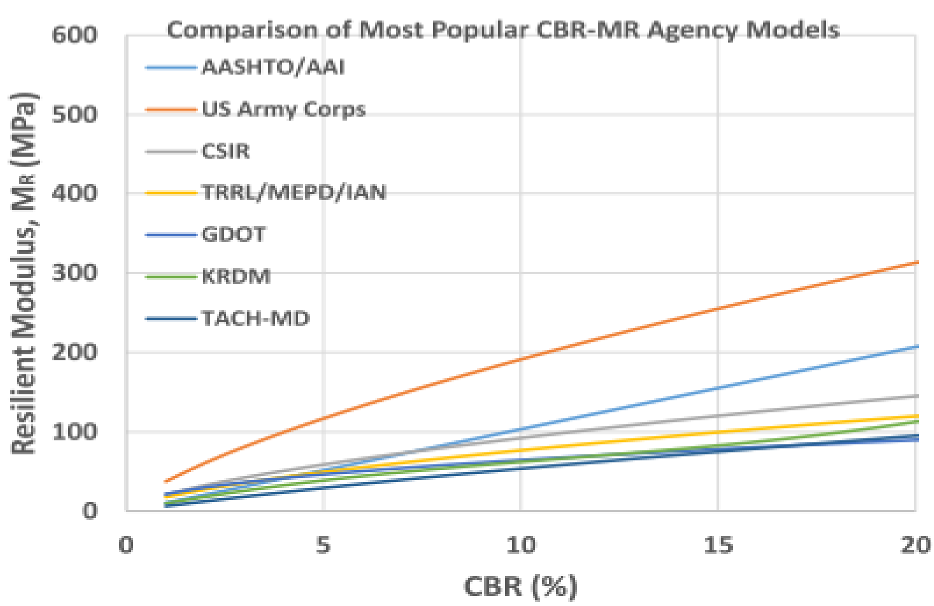


Figure 2.6: Comparison of popular Agencies adopted models correlating CBR and M_r for low stiffness Subbase and variance stiffness Subgrade levels

2.3.6 **Correlation of CBR and Modulus of Subgrade Reaction (K_s)**

CBR test is commonly used for evaluating the suitability for a subgrade or subbase soils for construction and runway and highway design. A plate load test is generally used for predicting the deformations, failure characteristics of subgrade soil, and modulus of subgrade reaction (k_s). which is an important parameter in foundation design, soil structure interaction studies and design of highway pavement for each flexible and rigid pavements (Putri et al., 2012) .

The modulus of subgrade reaction (k_s) also known as coefficient of elastic uniform compression, is a relation between deflection and soil pressure which is comparative to its vertical displacement as clarified in Winkler's soil model (Hetenyi, 1946), (Jones, 1997). While Kameswara Rao. (2000) defined the modulus of subgrade reaction as the ratio of uniform pressure imposed on the soil to the elastic part of the settlement.

Very little works were developed to find a correlation between CBR test and the modulus of subgrade reaction (k_s) (Putri et al., 2012). Terzaghi (1955) discussed various parameters PLT by using a circular plate with a diameter of 760 mm and a thickness of 16 mm. Jones (1997) opined that the correlation between of k_s and CBR can be used to predict the results of field plate load test and other elastic analyses.

In order to develop the correlation between CBR results with the PLT results, firstly that required development of a rational approach to correlate of CBR versus E. These values are subsequently used for evaluating the modulus of subgrade reaction, thus providing an easier way for analysis of soil structure interaction and pavements. Timoshenko and Goodier (1951), Harr (1966) and Kameswara Rao (2000) opined the k_s of the clayey sand soil can be expressed by using theory of elasticity solution for a rigid plate on a semi-infinite elastic soil medium subjected to a concentrated load, and CBR value described by means of the developed correlation with k_s, if the Poisson's ratio of the soil is 0.4, by the following expression :

$$ks = 1.13\ \frac{E}{(1-\nu^2)}\frac{1}{\sqrt{A}} \quad \text{--} \quad (2.5)$$

Where:

E = Modulus of Elasticity =751 CBR (kPa), if Poisson's ratio equal to 0.4

ν = Poisson's ratio.

A = area of the plate or CBR plunger.

Hajiannia et al. (2016) opined that the relation was proposed for the determination of the modulus of elasticity found from PLT through CBR test results for the soil of the site, has yielded quite acceptable results, see Figure (2.7), and the value of the Poisson's ratio (ν) affects in the modulus of elasticity (E), and (ν) the values of 0.3 - 0.4 are quite acceptable.

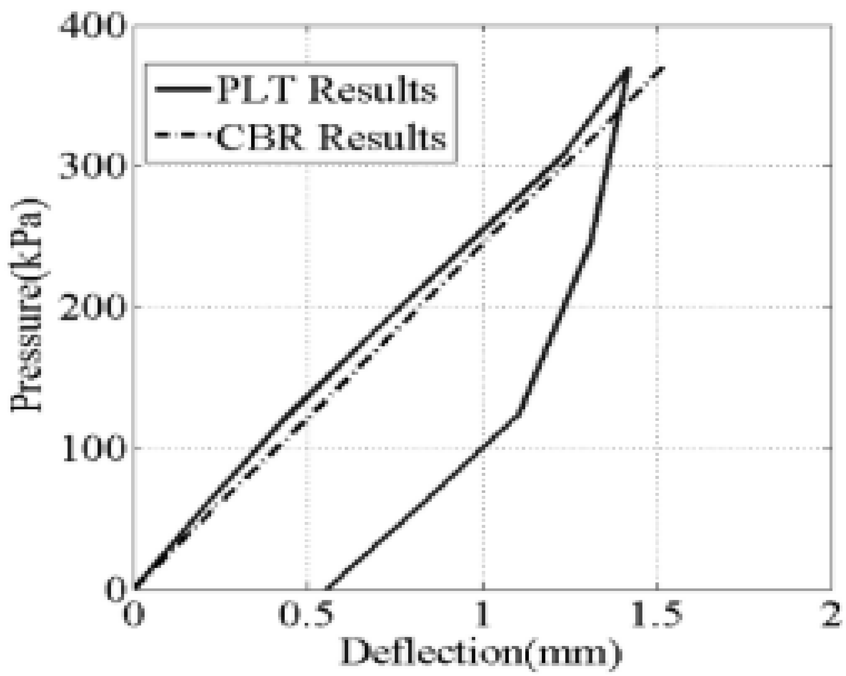


Figure 2.7 : Comparing PLT load-settlement curve and the pre- sent study's CBR results adapted by (Hajiannia et al., 2016)

2.4 Light Weight Deflectometer (LWD)

Many test methods were developed to determine the bearing resistance of unbounded materials pavement layers. Some of these testing methods require sampling, coring, laboratory testing and cause damages to the pavement layers. Because these processes are very time consuming and costly, these methods are not as popular. These tests methods are classified as “Destructive Test Methods”. In contrast, Non-Destructive Test (NDT) test methods are vastly and more popular used for pavement layers evaluation. The (NDT) devices are in-situ tests that can be conducted almost at any construction times and some of these tests after the construction of the pavement layers. One of these non-destructive test devices is the Light Weight Deflectometer (LWD), see Figure (2.8).

Figure 2.8 : Light weight deflectometer test device

LWD is a non destructive and portable device used to measure the in situ dynamic modulus of pavement layers and fill embankment.. Many different comparatives were suggested between the LWD test and other test devices, as shall be presented in the next subsections:

2.4.1 **Comparatives Among LWD and other Test Devices**

Many comparatives were conducted to evaluate the resilient modulus from in-situ test devices such as falling weight deflectometer (FWD), Miniaturized Pressuremeter Tests, laboratory triaxial test, and laboratory CBR test. Fleming et al. (2009) carried out a comparative investigation between FWD and LWD devices and opined that absence to correlate between FWD and LWD moduli, because of the different loading rates used in the two tests. Shaban (2016) developed a miniaturized pressure meter test (MPMT). The MPMT defined as a small version of the pencel pressuremeter (PPMT), which was developed evaluate the elastic properties of pavement layers and thin pavement layers without worrying about the edge effects of unbound granular layers. Shaban and Cosentino (2016) reported that the initial elastic modulus E_i which extracted from reduced the stress-strain data calculated from the MPMT with the linear elastic theory and reload elastic modulus E_r were correlated with the dynamic modulus E_d calculated from LWD data.

Louay et al. (2003) performed a comparative study between a field LWD test and a laboratory triaxial test to predict the laboratory resilient moduli of subgrades for in situ soil moduli. The results of the analysis were presented in two models. The first model relates M_r with LWD soil modulus (E_{LWD}). The statistical coefficients obtained from the model, including R-square (R^2) was 0.54 and root mean square error (RMSE) was 9.66. The resulting equation is:

$$M_r = 27.75 \times (E_{LWD})^{0.18} \qquad R^2 = 0.54 \qquad \text{--------------------} \qquad (2.6)$$

Because of the weak correlation, the second model was predicted depending on physical properties of the soils that was tested namely water content (W_c) in addition to the resilient modulus of subgrades in dynamic LWD soil modulus included in multi variable statistical analysis. The results of a regression model were recorded R^2 = 0.7 and RMSE = 7.

$$M_r = 11.23 + 12.64(E_{LWD})^{0.2} + 242.32(1/W_c) \qquad R^2 = 0.70 \qquad \text{------------------} \qquad (2.7)$$

Rao et al. (2008) opined that the CBR obtained with traditional testing was closely correlated to the LWD modulus, that was observed in the following linear model, with R^2 of 0.90.

$$CBR = -2.7543 + 0.2867\,E_{LWD} \qquad R^2 = 0.90 \qquad \text{------------------} \qquad (2.8)$$

2.5 Summary.

From the extensive literature review that was achieved, the following points can be highlighted:

- Resilient modulus (M_r) of the subgrade is a very important factor for evaluation airfield and highway pavement design, whereas, under limited lab facilities, this factor can be determined using simple empirical relationships with CBR (California bearing ratio) values.
- Because of the laboratory tests are costly, time consuming, need to sample and considered as a destructive test, the field tests are suggested to evaluate the strength and stiffness of pavement layers.
- Due to the simplicity and rapidity, recently conducting field test by LWD is widely used to evaluate the subgrade strength of subgrade soils.
- Extensive researches were conducted to develop a relationship between CBR value and several test devices in addition to correlate the bearing resistance to soil properties.

However, in this research, the correlation between the in – situ CBR and LWD was suggested to extend the current knowledge regarding the prediction of subgrade bearing strength values from advanced techniques.

Chapter Three

SOILS, TESTING, AND METHODOLOGY

3.1 Introduction

This chapter focuses on the experimental works, using materials and devices, manufacturing apparatuses of the test setup, testing procedures, and testing program. The experimental works include a series of laboratory tests to determine the bearing resistance of local subgrade soils, to achieve this purpose that required evaluating of selected local subgrade soils and identifying their engineering and physical properties. In addition, this research work adopted multi-stages to test the main hypothesizes:

- Approval light weight deflectometer test to predict the bearing capacity of subgrade soil by obtaining dynamic measurements such as dynamic modulus.
- Correlating the dynamic measurements from LWD with conventional subgrade soil properties such as California bearing ratio CBR.

3.2 Soils Characterizations

In order to ensure the economic considerations and to sustain the local practice, evaluated subgrade soils in this work were collected from three roadway projects in Karbala city. The sites project was AL-Meelad district, AL-Fares district, and AL-Rofaee zone. The materials selection was based on covering as much as possible the predominant soil types that available in Karbala. As shown in Figure (3.1), different laboratory tests including physical and chemical tests were conducted on each soil type to define the basic soil properties. These tests were conducted according to ASTM standards and AASHTO procedures.

Figure 3.1: Laboratory physical tests.

3.2.1 **AL-Meelad District.**

This project is located at southwestern of Karbala city, which has a length of 1400 m and width of 12 m. A sample of 2 m^3 soil was collected from different zones in this roadway site. The aerial view of a selected zone is shown in Figure (3.2). Physical and chemical tests were conducted, six test points for each LWD measurements and in-situ CBR were taken. Six attempts of the cone cutter method and sand replacement method were performed to achieve field density. Table (3.2) lists the basic physical characteristics and the chemical elements of the subgrade soil, respectively.

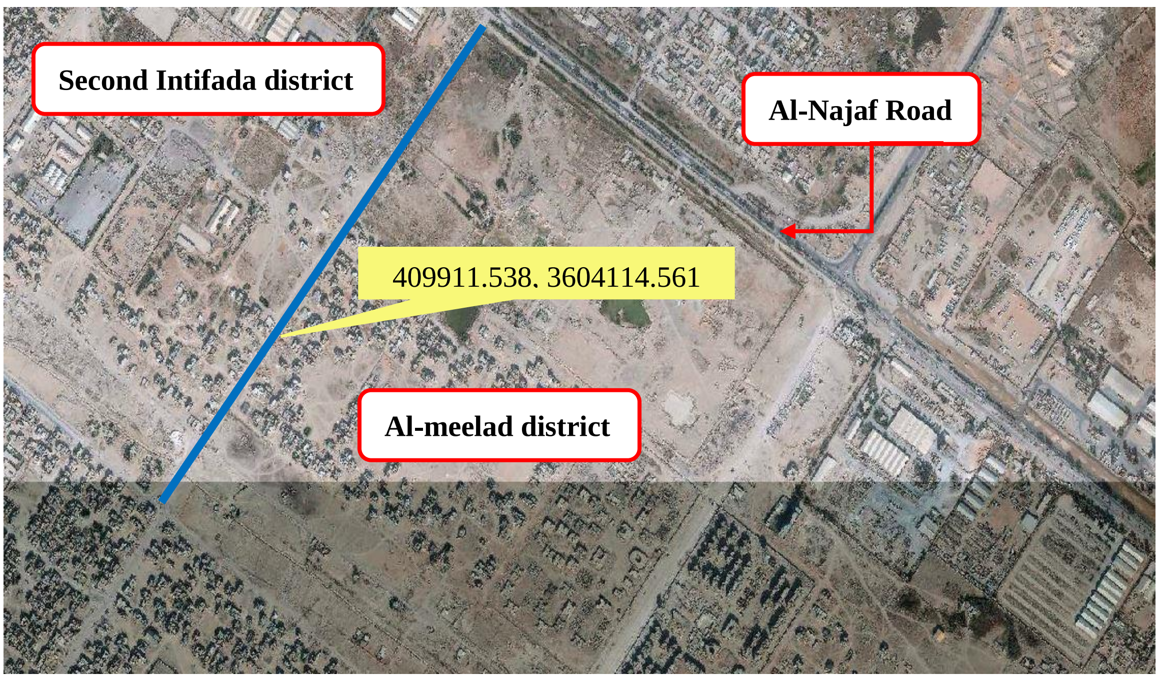


Figure 3.2: The aerial views of AL-Meelad district.

Table 3.1: Average physical and chemical characteristics of subgrade soil.

Property	AL-Meelad	AL-Fares	AL-Rofaee	Specification
Physical characteristics				
Site coordination	409911.538, 3604114.561	406139.763, 3604069.317	406073.683, 3617974.787	/
USCS classification	poorly graded sand with silt (SP-SM)	poorly graded sand (SP)	Inorganic elastic silt with high of plasticity (MH)	ASTM D2487
AASHTO classification	A-1-b	A-3	A-7-6	AASHTO M145
Dry Unit Weight	16.87 kN/m3	20.94 kN/m3	16.87 kN/m^3	ASTM D1557
O.M.C	15.5%	8.75%	18 %	ASTM D4643
G. S	2.72	2.74	2.55	ASTM D891
D_{10}, D_{30}, D_{60}	0.13, 0.30, 0.62	0.17, 0.29, 0.72	/	ASTM D2487
Uniformity Coefficient, C_u	4.77	2.47	/	
Curvature Coefficient, C_c	1.11	1.18	/	
Gravel Fraction, GF	9.20%	1.52 %	/	
Fine Content	5.50%	4.01 %	90.20 %	
Lab. CBR – unsoaked	61.5%	99 %	22.2 %	ASTM D1883
Lab. CBR – soaked	27.2%	50.5%	5.2 %	
Liquid limit	0	0	64.90 %	ASTM D4318
Plasticity Index	N. P	N. P	25.96	
Chemical characteristics				
SO_3	4.409	4.959	*1.86*	BS1377-3:1990
SiO_2	28.00	51.0	*53.40*	
AL_2O_3	4.50	6.24	*9.78*	
Fe_2O_3	1.60	4.90	*8.90*	
LoS	11.80	6.20	*17.90*	
CaO	26.29	11.17	*8.63*	
$CaSO_4$ $2H_2O$	9.48	10.66	*3.99*	
Total Soluble Salts	1.56	1.10	*3.53*	

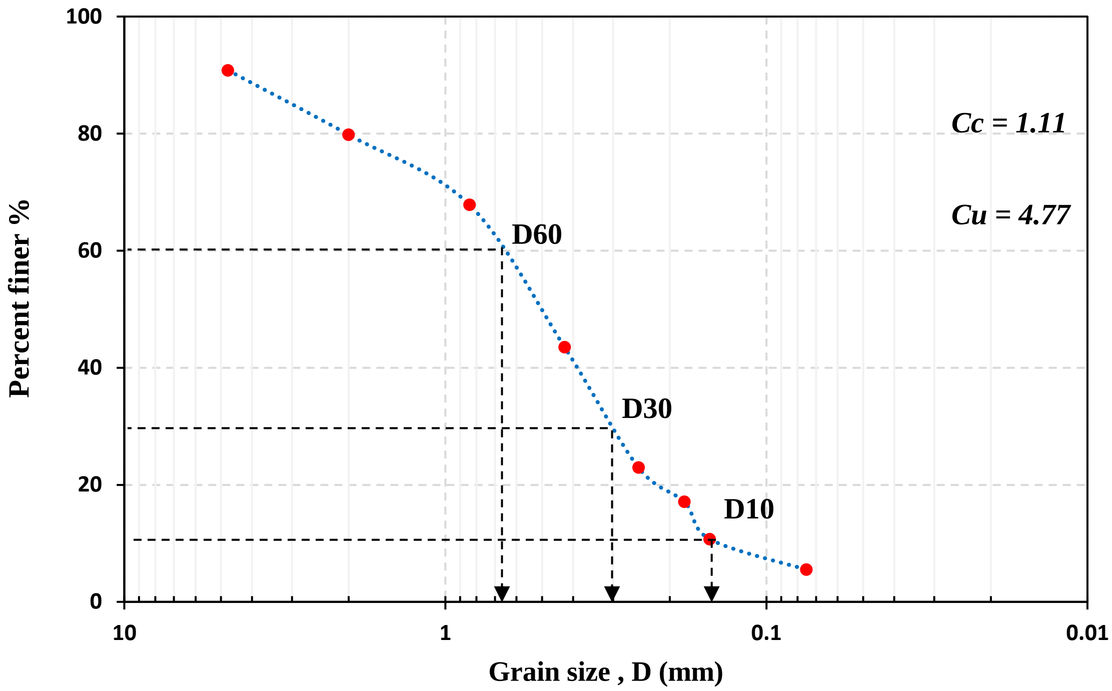


Figure 3.3: Grain size distribution of subgrade soil – AL-Meelad district.

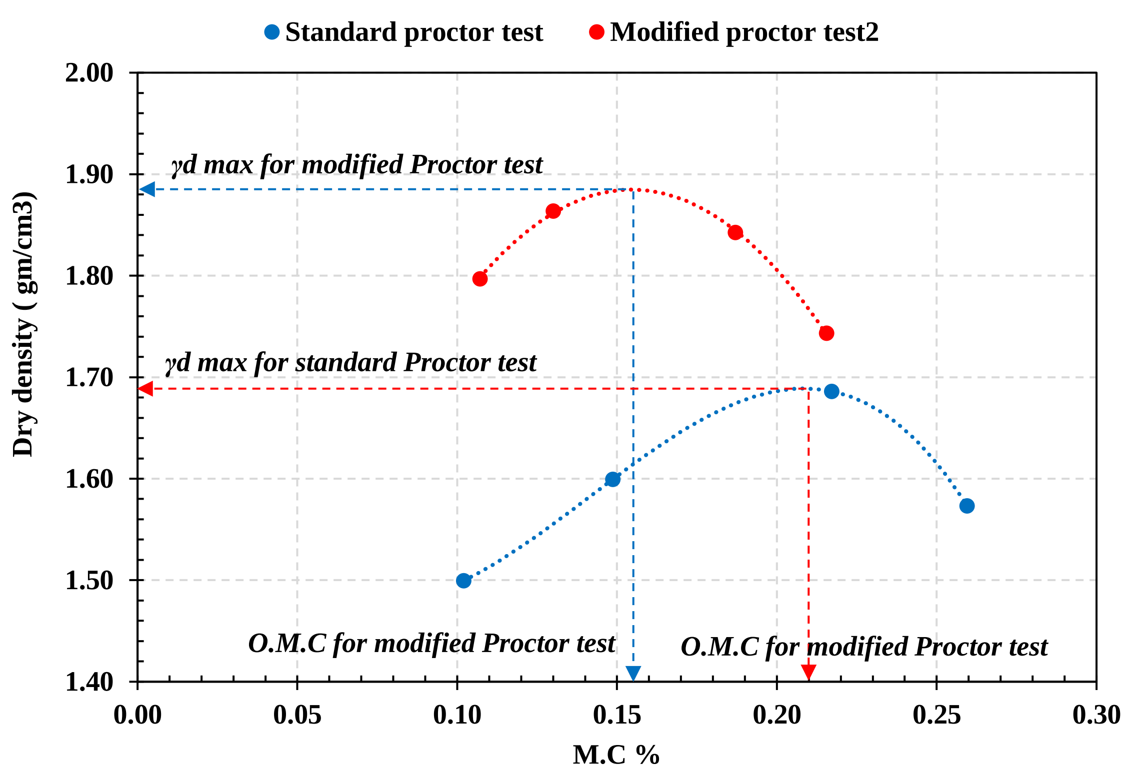


Figure 3.4: Standard and modified Proctor test of subgrade soil - AL-Meelad district.

3.2.2 **AL-Fares District.**

This project is located at south-eastern Kerbala city which consists of many local streets of length 200 m and width 10 m and a collector with length 900 m and width 10 m. This zone was submitted to a stage of rehabilitation and development of its local streets. Aerial view of the selected zone is shown in Figure (3.5). The methods of selection, providing the sample and testing are similar to methods used to select and provid a previous subgrade soil type. At the same procedure, physical and chemical tests were conducted, see Table (3.2).

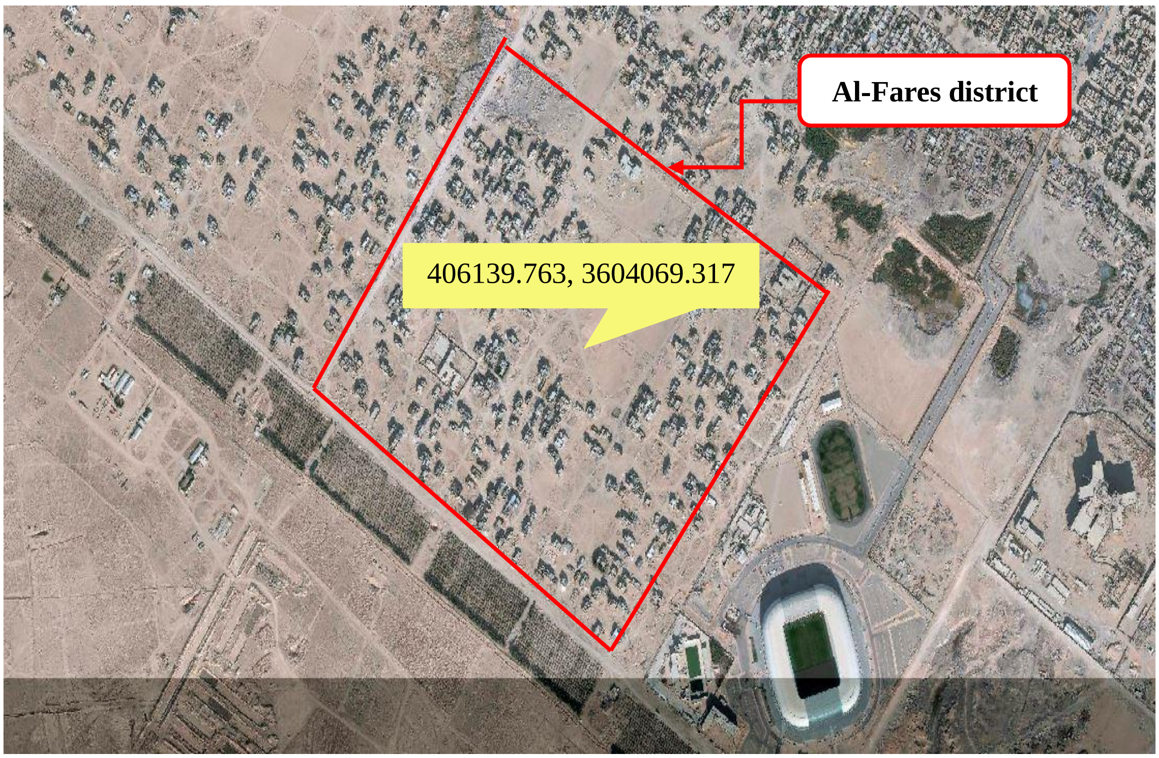


Figure 3.5: Aerial views of Al- Fares district.

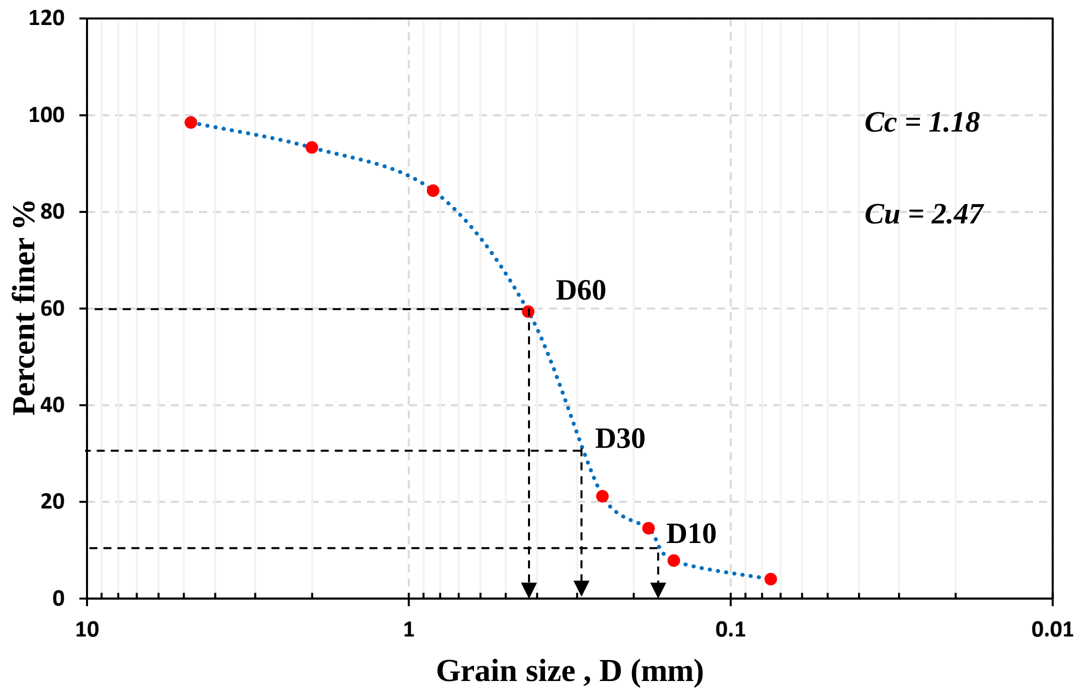


Figure 3.6: Grain size distribution of subgrade soil - Al-Fares district.

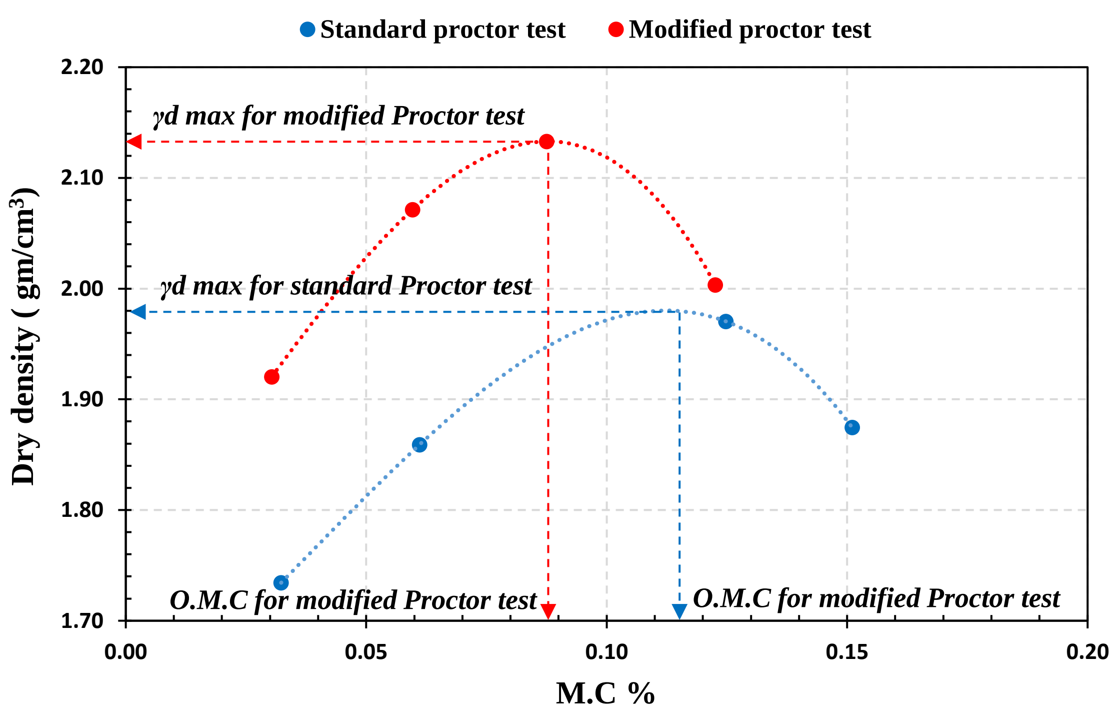


Figure 3.7: Standard and modified Proctor test of subgrade soil - Al-Fares district.

3.2.3 **Al- Rofaee Zone.**

This project is located west of Karbala city. This zone is considered one of the quarries in the city for the preparation of clayey soils to projects that require either for replacement of subgrade soil for non-conformity with a specification or in cases of increasing a highway level. The aerial view for Al-Rofaee zone is shown in Figure (3.8). Physical and chemical tests were conducted as presented in Table (3.2).

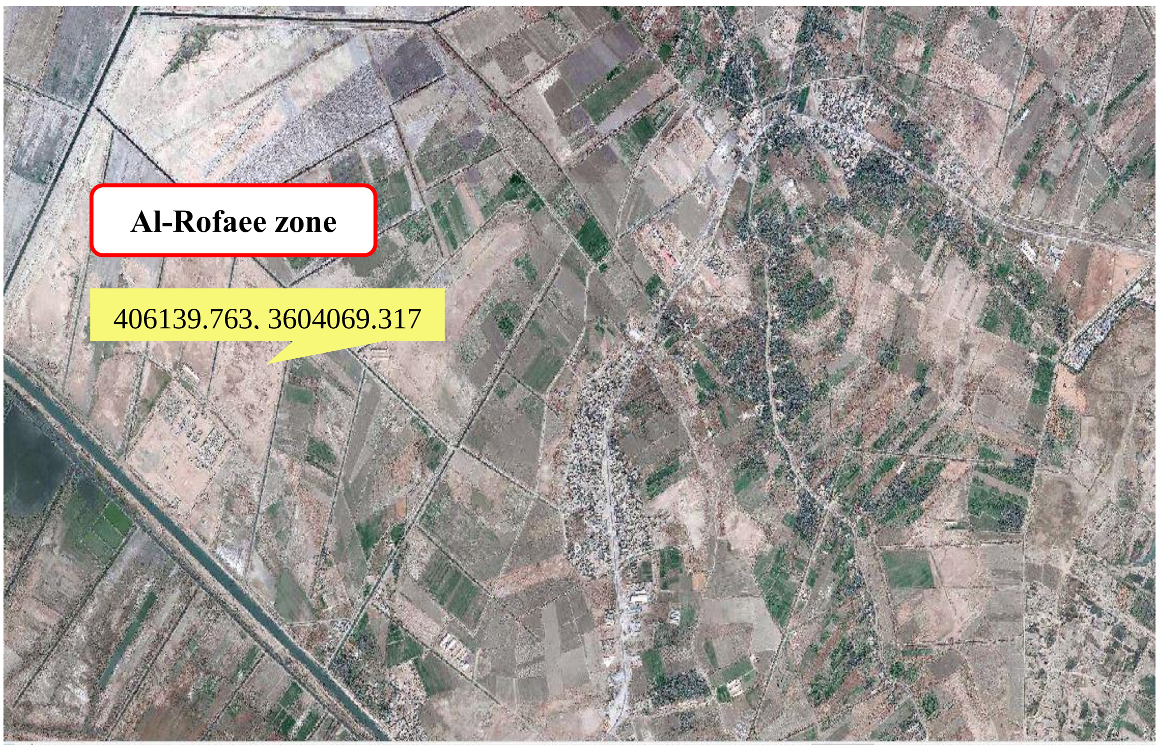


Figure 3.8: Aerial views of Al- Rofaee zone.

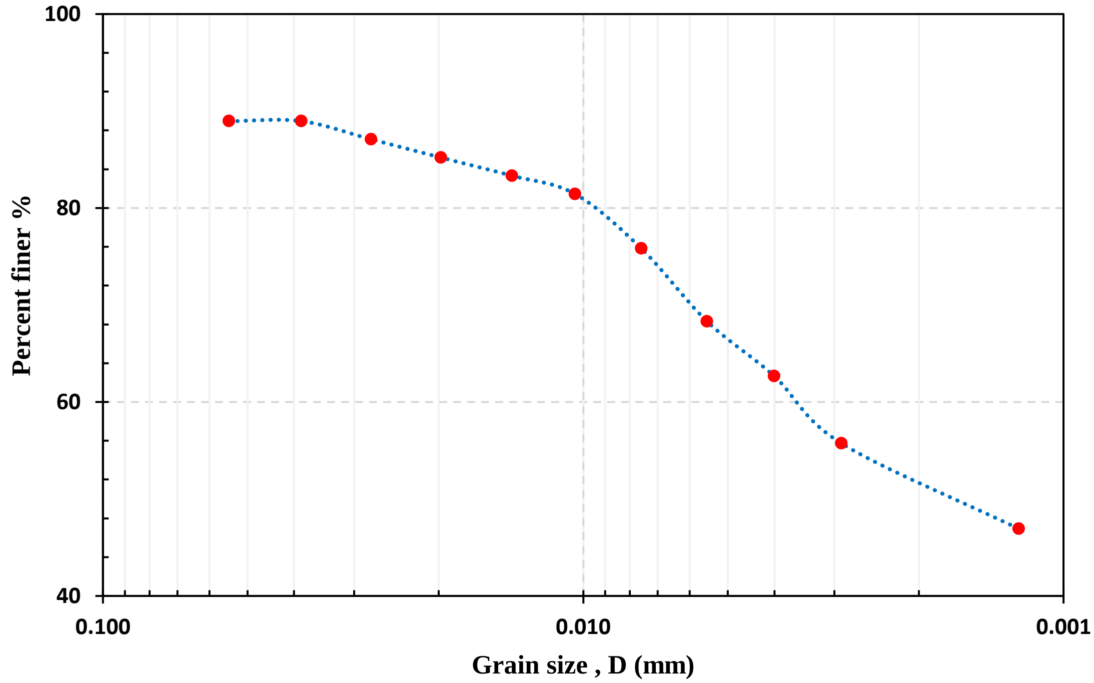


Figure 3.9: Grain size distribution of subgrade soil - Al-Rofaee zone.

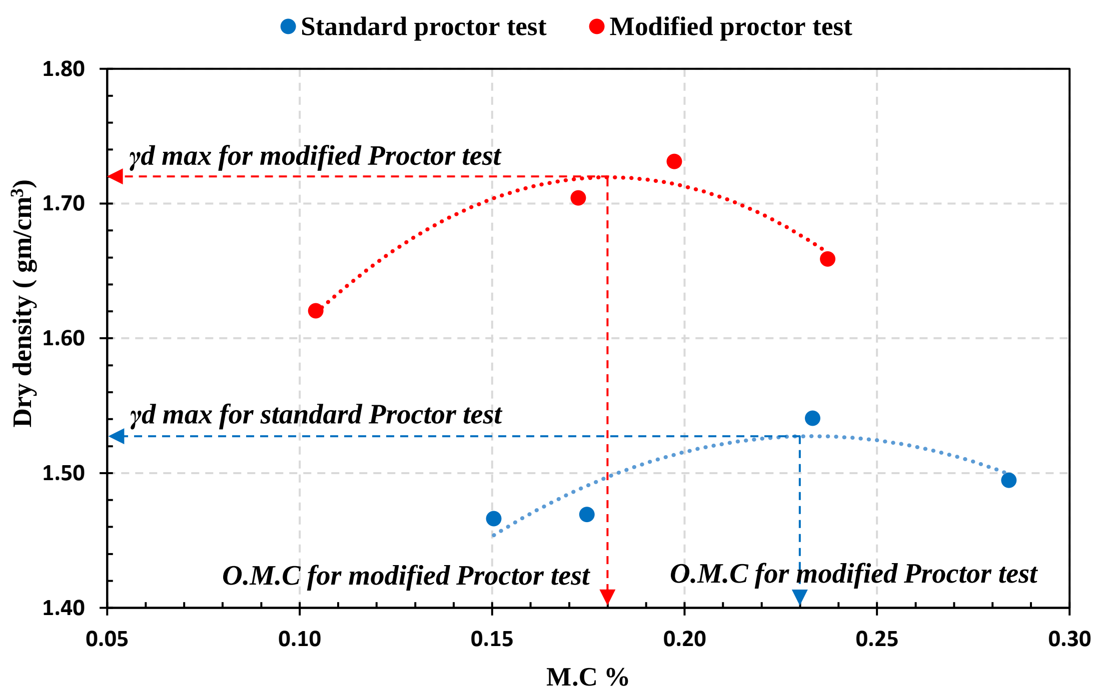


Figure 3.10: Standard and modified Proctor test of subgrade soil - Al-Rofaee zone.

3.3 Testing System.

To evaluate and predict bearing resistance of local subgrade soils, and identify their engineering and physical properties, it is necessary to simulate the conditions in the field as close as possible. To achieve this purpose, a special testing apparatus and different accessories were designed and manufactured. The apparatus has a steel box with capability of containing soil about 2 m^3 and applying different static and dynamic loads. The general view is shown in Figure (3.11). The testing device consists of the following parts:

1- Loading steel frame.

2- Axial loading device.

3- Steel box.

4- Data acquisition.

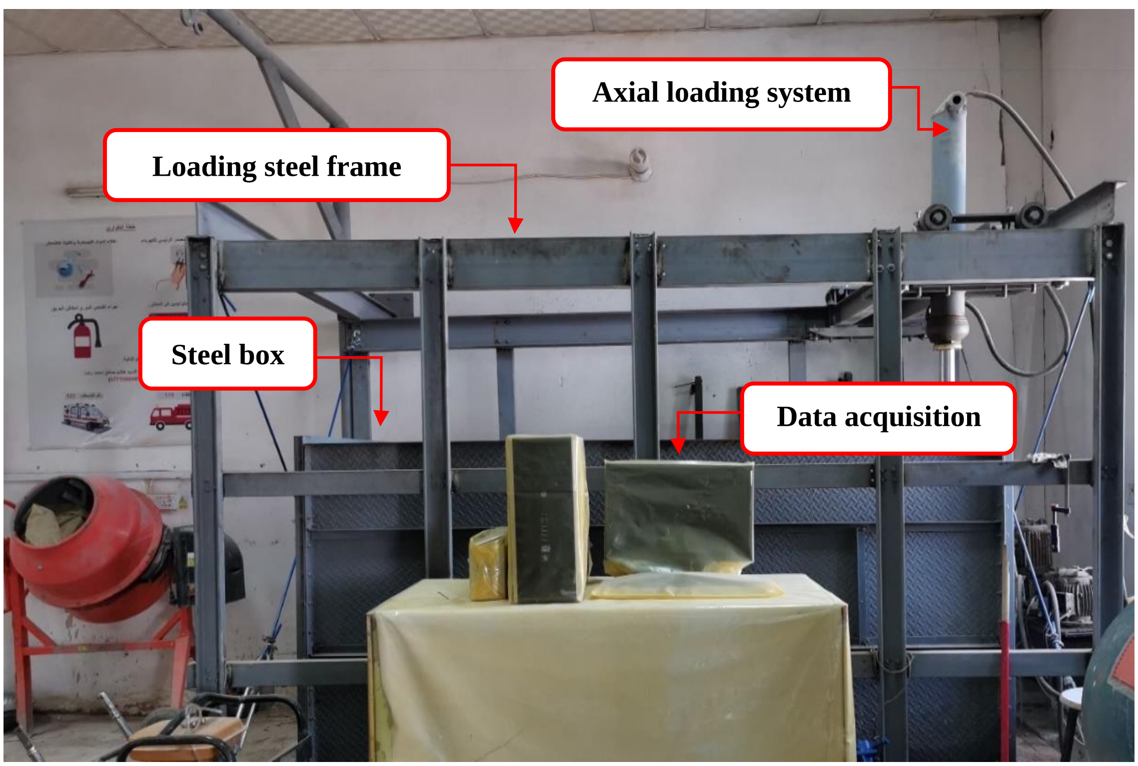


Figure 3.11: General view of the apparatus.

3.3.1 Loading Steel Frame

The loading steel frame was designed with dimensions (length× width× height) of (295cm×175cm×250cm). It consists mainly of longitudinal and horizontal steel sections in each direction with different shapes, dimension and purpose, these sections were connected with each other by the number of bolts and welding to carry hydraulic jack system. Table (3.2) shows the details of the steel section classified according to the American Institute of Steel Construction AISC and European standard U channels UPE with parallel flange and universal I beam IPE. (three-dimension details (3-D) and two-dimension details (2-D) were shown in Figures (3.12) and (3.13), respectively.

Table 3.2: Details of steel section used in the loading steel frame.

Sec. description	Symbol	Number	Designation According AISC	Designation According European S.	Area in^2	Depth in	Web thickness t_w	Flange Width b_f	Flange thickness t_f
Longitudinal C-sec.	C1	10	C3×6	UPE 80	1.76	3.00	0.356	1.596	0.273
Horizontal C-sec.	C2	4	C4×7.25	UPE 100	2.13	4.00	0.321	1.721	0.296
Horizontal S-sec.	S1	4	S5×10	IPE 140	2.94	5.00	0.214	3.004	0.326

Figure 3.12: 3-D details of loading steel frame .

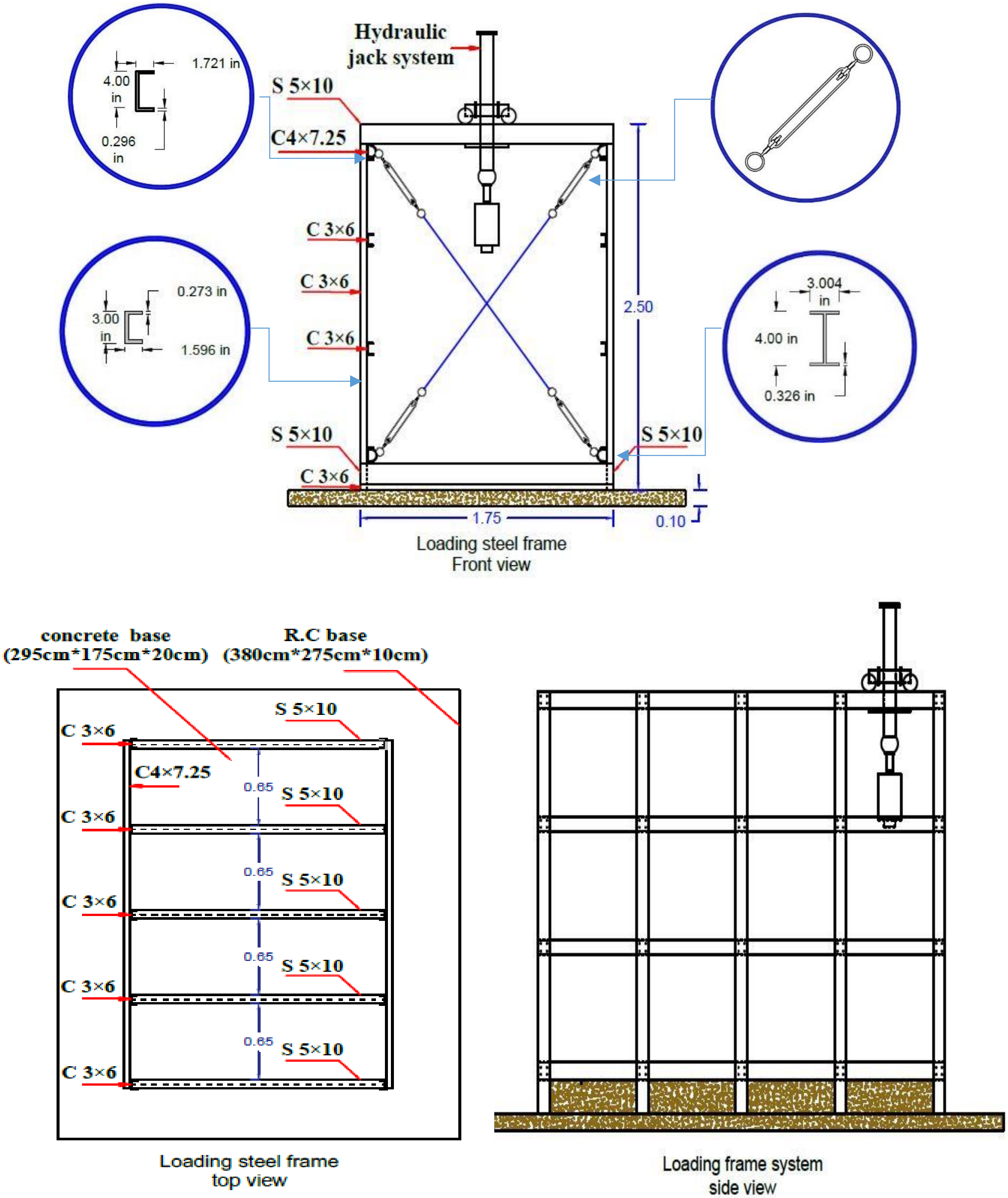


Figure 3.13: 2-D details of loading steel frame .

3.3.2 **Axial Loading Device.**

The axial loading device consists of:

a) Hydraulic jack system: It consists of a hydraulic steel jack with a capacity of up to 20 tons and a height of 65 cm. The hydraulic jack system carries by rail subjected to loading steel frame that enables the hydraulic jack to have flexible horizontal movement in four directions (right, left, forward and backward). This system consists of a piston with a diameter of (50.8 mm) and has the capability to flexible vertical movement. The lower end of the piston has a load cell that is responsible for measuring the amount of applied load on the soil sample.

b) Hydraulic control system: The control device consists of a system that is responsible for applying the load, and movement of the piston. The control system consists of two motors with speed of (1500 r.p.m) which are responsible for controlling piston movement speed. the movement (up and down) can be controlled either manually or automatically from a computer, Figure (3.14) shows general views, 2-D and 3-D details of axial loading system.

Figure (3.14a): Ganeral view

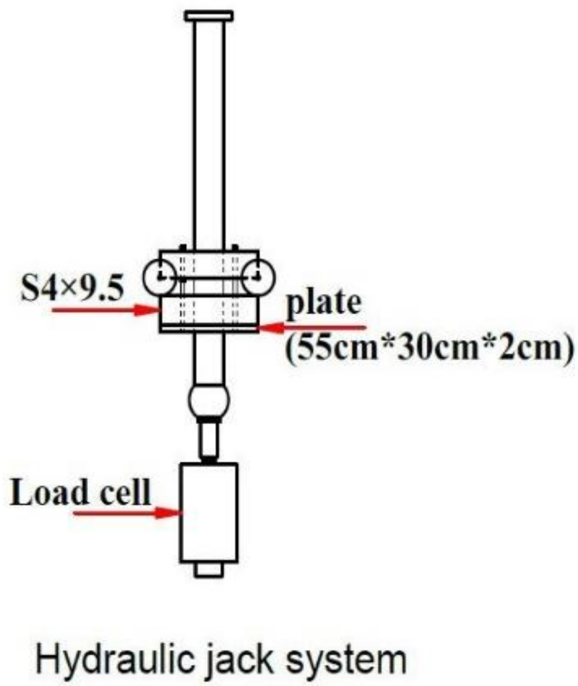


Figure (3.14b): 2-D schematic view

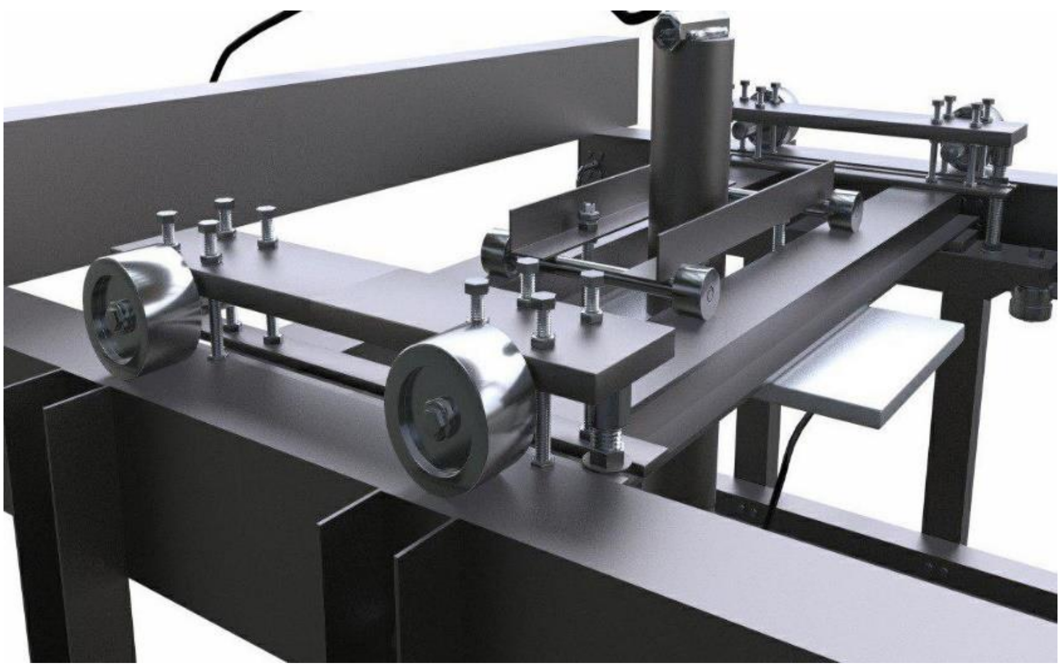


Figure (3.14c): 3-D schematic view

Figure 3.14: Details of axial loading system: a) original view, b) 2-D, c) 3-D sckematic view

3.3.3 **Steel Box**

The experiments were conducted in a steel box of 240 cm length,120 cm width and 125 cm height to contain soil sample placed in three layers, the thickness of each layer is 20 cm and the total height is 60 cm. All internal faces of the box were divided into intervals every 10 cm to ensure the level of soil sample inside the box to the desired level. Steel box details are shown in Figure (3.15).

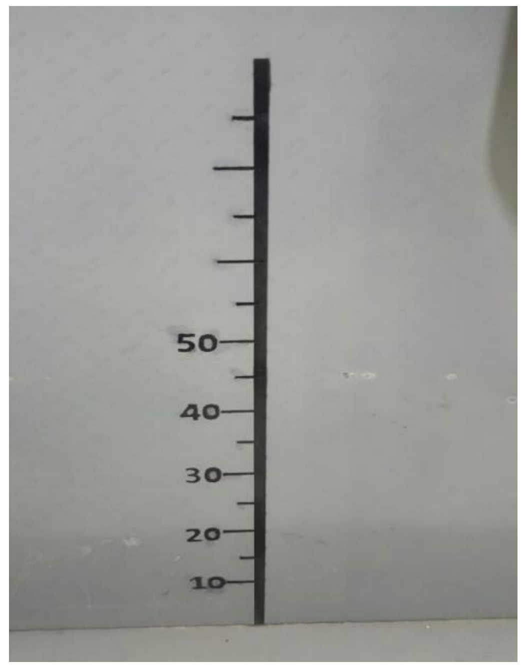


Figure 3.15: Steel box details.

3.3.4 **Data Acquisition System.**

To investigate a real soil behavior during the application load, it is necessary to develop a procedure to measure and sense the displacement that occurred due to applied load during the test. This procedure, which enables to record the total accurate data consists of huge readings in a very short time. For this reason, the data acquisition system was used. This system is controlled by a control unit system as shown in Figure (3.16) which consists of:

- LAB VIEW.
- Load Cell.
- LVDT.

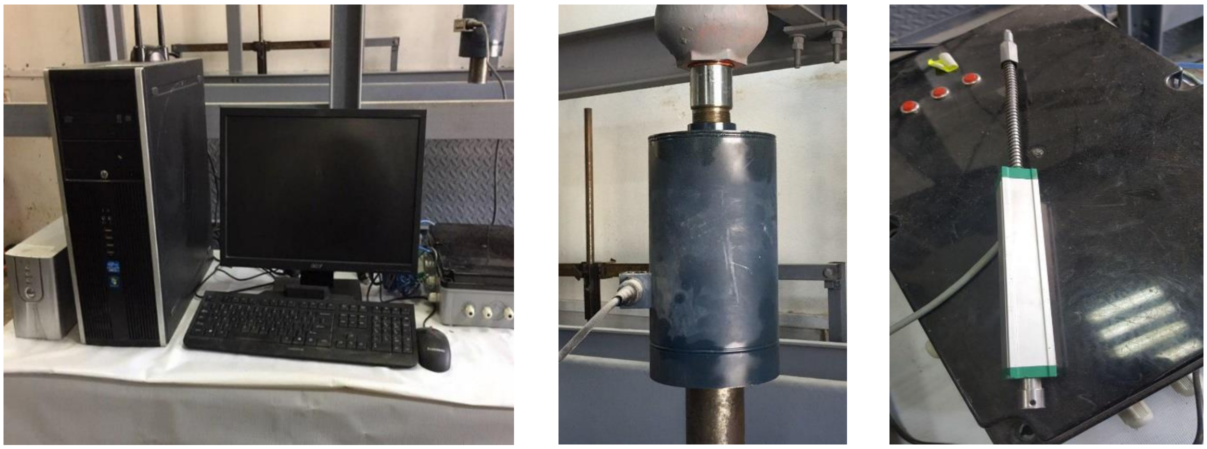


Figure 3.16: Data acquisition system.

LabVIEW is a software development environment created by National Instruments. Originally it was focused on taking measurements from various lab instruments, but it is expanded drastically from its inception. Strictly speaking, LabVIEW is not a coding language, it is a development environment. LabVIEW is used for four main purposes:

1. Automated Manufacturing test
2. Automated Product design validation
3. Control and/or monitoring of a machine
4. Condition monitoring of a machine

In experiments, LabVIEW has the ability to operate the loading system automatically and defines the applied load that causes the failure of the soil by measuring the applied load compared to the measuring deflection of the soil at a rate of readings 100 readings per second. Where the load records by the load cell and deflection records by an LVDT instrument.

3.4 Soil Preparation

Three collected subgrade soils were utilized to prepare four laboratory testing sections that simulate in-situ conditions of subgrade soils., which are represented by field density test by approval two test methods CCM and SRM, in-situ CBR test, and LWD test in the following manner:

According to the values of moisture content obtained from the Modified Procter Test for each subgrade soil type, the soil was moistened with optimum moisture content using a drum mixer of 0.25 m^3 capacity. For each mixture, 150 kg of soil was divided into containers 25 kg for easy sample transfer to the mixer and ensure the mixing process, as shown in Figure (3.17). Six mixtures in amount 0.25 m^3 are required to prepare one layer with thickness 20 m^3. To conduct these tests and to ensure the influence depth, three subgrade layers were required to prepare before testing, that means each layer was required 1.5 m^3 volume, which equal to 0.9 ton of soil and the total weight of testing sample equal 2.7 tons.

Figure 3.17: Soil sample division and mixing instrument.

Before placing the soil sample in the steel box, the inner walls of the steel box were covered with a light insulation plastic sheet to prevent soil sticking to the walls of the mold. The mixed soil sample was placed in the steel box in the form of three compacted layers as mentioned previously, see Figure (3.18a). Each soil layer was compacted to increase the density of the soil by packing the particles closer together. The compacted effort was achieved by using a compacter (model: petrol engine with power 6.0 Kw, weight 160 kg, and frequency 4000 VPM), see Figure (3.18b).

a) Compacted subgrade layers b) compactor

Figure 3.18: Soil compacted layers and compaction instrument.

Three compaction efforts were adopted for each soil type depending on the number of compactor passes (NOP) applied on each layer. The compaction effort was classified into eight numbers of passing (8NOP), twelve numbers of passing (12NOP) and sixteen numbers of passing (16NOP). The purpose of using three compaction efforts was to achieve a variety of densities for each soil type and identify the extent of compaction impact on the results of the tests.

3.5 **Laboratory Tests**

After completing the preparation of the soil sample in the steel box with a total thickness of 60 cm and ensuring the top surface is leveled to get as near as possible a flat surface. The soil surface is divided into six testing zones as shown in Figure (3.19). For each testing zone several tests were implemented including; 1) in-situ CBR test, 2) LWD test, 3) core cutter method (CCM), 4) sand replacement method (SRM), and water content. (Wc).

Figure (3.19a): Laboratory testing zones.

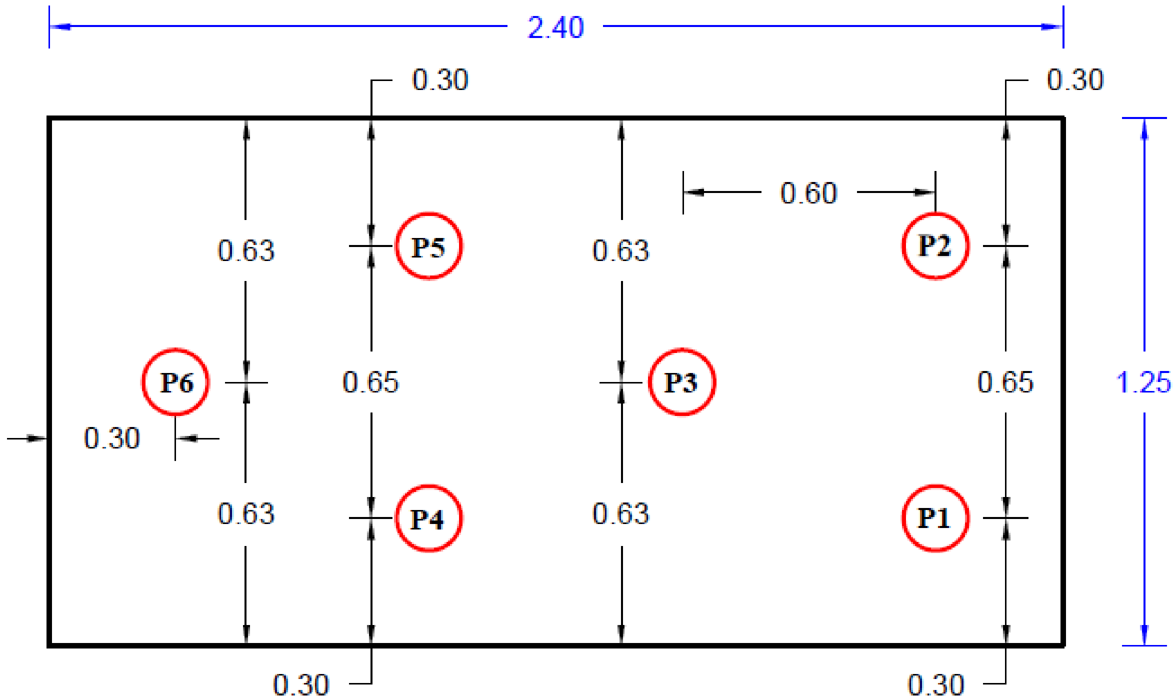


Figure (3.19b): Test points diagram

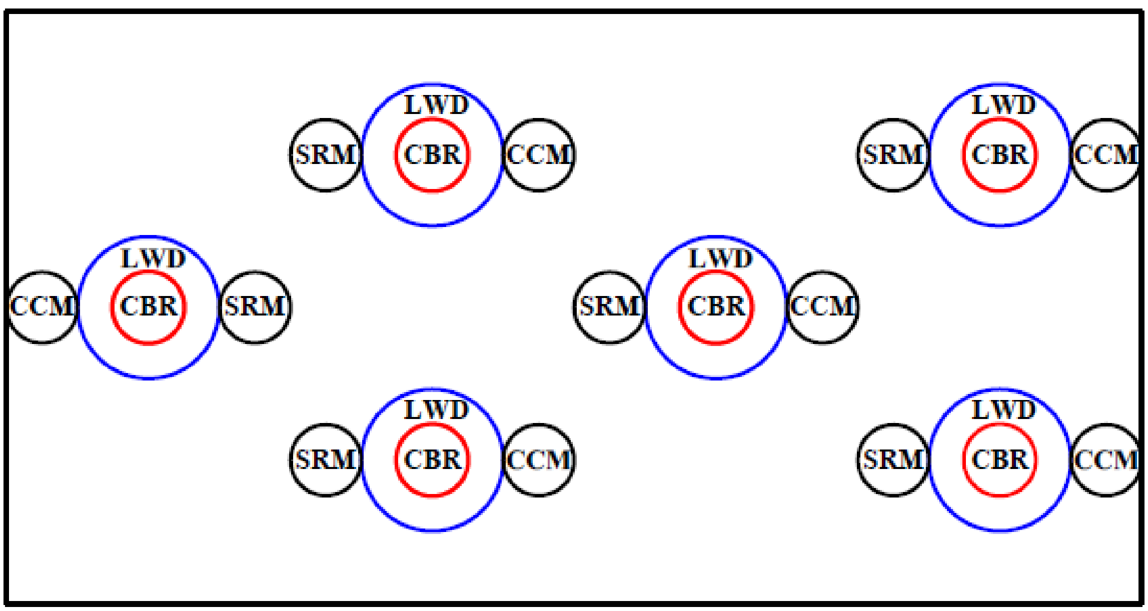


Figure (3.19c): Test conducted for each test points

Figure 3.19: Test points diagram.

3.5.1 **In-situ CBR Test.**

The CBR test is used to estimate the load bearing capacity and mechanical strength of unbound pavement layers. The CBR test is used primarily to empirically determine the required thicknesses of flexible pavements. Originally developed by the California Department of Transportation, it is now widely used in the design of roads, pavements, car parks, and similar applications. The CBR test is performed on laboratory remolded soil specimens and field (in-situ) subgrade soil to determine their bearing capacity. To achieve this purpose, static load was applied at a rate equivalent to 0.05 inch per minute (1.27 mm/min) by using a piston with diameter 2 inch (50.8 mm) and depth 0.5 inch (102mm) (ASTM D4429). At intervals of 0.1 inch and 0.2 inch penetration, readings of static applied load and vertical displacement were recorded, and the plot of stress- penetration curve was drawn. Then, the CBR value for 0.1inch and 0.2 inch penetration was measured as follows:

$$\sigma\text{applied} = \frac{\text{Penetration Force (kN)}}{\text{Piston area (mm2)}} \qquad \text{----------------------} \quad (3.1)$$

At 0.10-inch penetration:

$$\text{CBR}_{0.1} = \left(\frac{\text{stress applied @ 0.1 penetration}}{6.9}\right) * 100\% \qquad \text{----------------------} \quad (3.2)$$

At 0.20-inch penetration:

$$\text{CBR}_{0.2} = \left(\frac{\text{stress applied @ 0.1 penetration}}{10.3}\right) * 100\% \qquad \text{----------------------} \quad (3.3)$$

In-situ CBR instrument consists of many apperatuses which was selected in the line with recommendations of ASTM (D4429) as shown in Figure (3.20) such as :

1- Mechanical screw jack equipped with a special swivel head for applying the load to the penetration piston, provided a uniform load penetration rate of 0.05 inch (1.3 mm)/min, and designed with maximum capacity of (3000 kg) and minimum lift of 2 inch (50.8 mm).
2- Penetration piston has a 2 inch (50.8 mm) diameter with overall contact area of 3 in^2 (2000 mm^2).
3- LVDT was considered as dial gage in conventional in-situ CBR test for measuring penetration reading.
4- Load cell was considered as proving load cell ring in conventional in-situ CBR test for measuring loading range.
5- Surcharge plates were selected to simulate overburden pressure of upper pavement layers, it is consisting of a circular steel plate (15 mm) in diameter with a (50.8 mm) diameter hole in the center and weight (4.54 kg), slotted and circular surcharge units of weight (4.54 kg) and (216 mm) in diameter.
6- Hydraulic jack (reaction) was considered as a truck (or piece of heavy equipment) used in in-situ CBR to achieve loading sufficiently and provide a reaction of approximately (31 kN**)** (ASTM D4429, 2014) for forcing the penetration piston into the soil.

Figure 3.20: Apparatuses of in-situ CBR test.

The testing procedure of the in-situ CBR includes the following steps:

1. Locating the hydraulic jack in a sufficient height insured that the mechanical screw jack is directly over the surface to be tested, and connect the load cell.
2. Installing the mechanical screw test jack with the swivel to the underside of the load cell, and position the mechanical screw jack to the correct testing point.
3. Attaching the penetration piston in connect to the surface to be tested, and check the level mounted on the jack to be certain the assembly is vertical and adjust it if necessary.

4. Placing the surcharge plate beneath the penetration piston so that when the piston is lowered it will pass through the center hole.
5. Installing the LVDT beside the penetration piston and above the surcharge plate for recording penetration readings.
6. Add surcharge weights to the surcharge plate so that the unit load is equivalent to the load intensity of the material or pavement which will overlie the subgrade or base, or both. The installation of the in-situ CBR test apparatuses is displayed in Figure (3.21).

Figure 3.21:Installation procedure of in-situ CBR test.

3.5.2 **Light Weight Deflectometer Test**

A portable LWD device which was utilized in this research, it was manufactured by ZORN instruments (Type ZFG 3.0). The main purpose to use this device, it was to evaluate dynamic and compaction characteristics such as the dynamic modulus E_d, surface deflection S_d, and degree of compatibility D_c for selecting subgrade materials. This instrument consists of three main components, as shown in Figure (3.22):

1- Loading device which includes a (10 kg) drop weight,

2- A 30-cm loading plate equipped with an accelerometer to measure the vertical surface deflection of soil, and

3- Portable control unit used to record and display LWD testing measurements.

The LWD is defined as a nondestructive testing device used to determine in-situ stiffness properties of pavement materials under the effect of dynamic impact loads at in-situ conditions. This device provides a single dynamic stiffness back-calculated based on the actual wave velocity propagated inside a pavement layer (Rayden and Mooney, 2009). The influence depth of LWD pulse is at range (1.5 - 2.0) times of loading plate diameter, for this reason, the LWD device is considered as not suitable device to evaluate in-situ stiffness for depth is often greater than 20 inches (50.8 cm).

The general testing procedure of LWD test was conducted according to the standard test method which is described in (ASTM E2583) as follows:

- Placing a 30 cm circular loading plate on a test point under which soil properties were required. The test point was leveled to provide full contact with the circular steel plate and ensured the test surface was clean and smooth as possible. For gravel surfaces, it is recommended that a thin layer of fine sand be placed over the test point. This helps in obtaining uniform contact between the load plate and the surface. A suitable rubber pad may be used for improving load distribution (ASTM E2583, 2007).
- Drop mass of 10 kg was dropped from a falling height equal to 116 cm, caused a dynamic impulse load on the soil surface. As shown in Figure (3.23), three drops were conducted for each testing point to decrease the effect of loose soil particles that may cause unfavorable plastic deformations. The dynamic modulus E_d, vertical surface deflection S_d, and degree of compatibility D_c are measured by using LWD test device depending on the concept of back-calculation based on acceleration data of the impulse load.

Figure 3.22: Components of LWD field test equipment

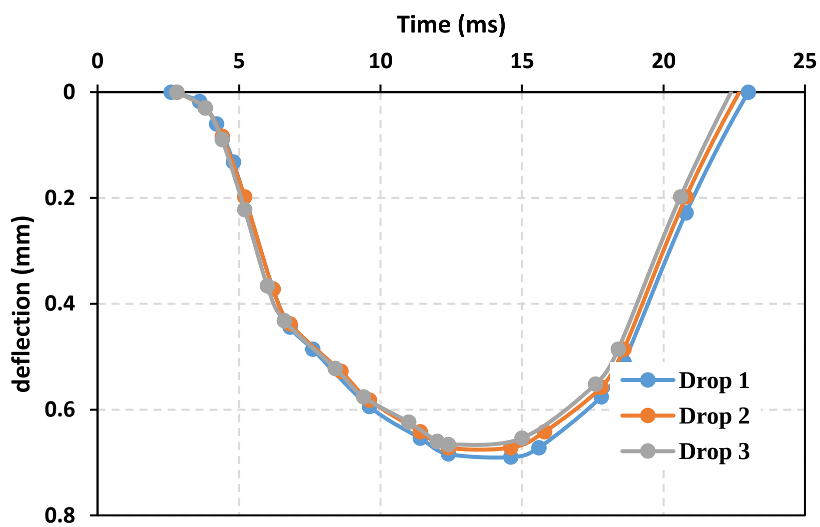


Figure 3.23: Typical Time-Deflection Curve of LWD Test

The LWD parameters measured during this test are listed below:

- Surface Deflection (S_d): is derived by doubles integrating the acceleration data with respect to time of pulse waves. The acceleration data are recorded using an accelerometer mounted inside the loading plate.

- Dynamic Modulus (E_d): is a back calculated from S_d depending on Boussineq theory that can be presented by the following expression (Shaban and Cosentino, 2016).

$$E_d = \frac{(1 - \upsilon^2)\, \sigma_o\, a}{\delta} . f \qquad (3.4)$$

Where is the E_d (MPa), σ_o is peak applied stress (MPa), a is the radius of loading plate (mm), and υ is Poisson's ratio, δ is peak vertical deflection (mm), and f = plate rigidity factor which is typically assumed (f = 2) for LWD ZFG model.

- Degree of Compatibility (D_c): is defined as a parameter which indicates a compaction characteristics of pavement layer and can be determined by dividing the mean surface deflection by the mean pulse velocity of dynamic impact load applied in surface layer. This parameter gives an indication of compaction characteristics. Generally, the compaction effort is well if the D_c is less than or equal to 3.5, while further compaction effort is recommended if the D_c is greater than 3.5 further compaction is recommended.

3.5.3 Determination of In-situ Density

3.5.3.1 Core Cutter Method (CCM).

Field density can be determined using a core cutter test. The method can be used successfully whenever soil conditions permit pushing of cutter for sampling and taking it out in the laboratory without much disturbance. The general testing procedure of CCM test was conducted according to the standard test method which is described in (ASTM D2937)

3.5.3.2 Sand Replacement Method (SRM)

This test is performed to determine the field density of soil according to ASTM D1556. S.R.M is applicable for soils without appreciable amounts of rock or coarse materials which exceeds

1.5 in. (38 mm) in, but it is also suitable for organic, saturated, or highly plastic soils that would deform or compress during the excavation of the test hole.

3.6 Methodology

To achieve the main aim of this research work, different types of subgrade soils were selected for measuring the static and dynamic strength for each type of soil, realizing physical and chemical properties and correlating these properties by using different testing methods. Figure (3.24) and Table (3.3) provides a soil designation and summary of testing method matrices that have been implemented in this work, respectively.

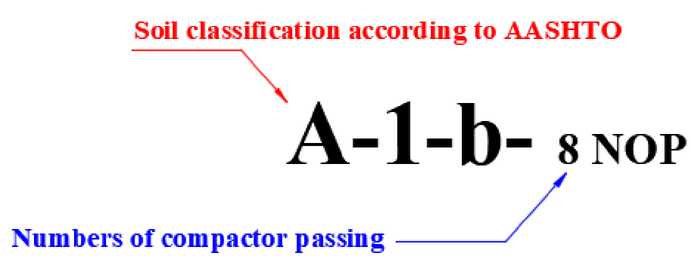


Figure 3.24: Subgrade soil mix designation.

The methodology comprised the following main stages, while more details and traditional methodology stages can be seen in Figure (3.25):

1- Selection of different sites and identifying their characterization by performing physical and chemical tests.

2- Identifying the bearing capacity of subgrade soil by using in-situ CBR test, LWD test and field densities by using different test methods.

3- Building up statistical models to predict the subgrade bearing strength from physical soil indexes and from advance LWD parameters

4- Verifying the statistical models by selecting two field sites and estimation their bearing capacity depending on their physical and mechanical properties.

Table 3.3: Tests were conducted on the soil samples.

Type of tests	Physical Tests										Chemical Tests							Lab. Tests				CCM Specimens	SRM Specimens	CBR Specimens	LWD Specimens
	Sieve analysis		Hydrometer analysis	Specific gravity	Moisture content	Liquid limit	Plastic limit	Proctor compaction	CBR	SO_3	SiO_2	AL_2O_3	Fe_2O_3	LOS	CaO	TSS	γd		CBR	LWD				
																	CCM	SRM						
Specification / Mix.	*AASHTO M145*	*ASTM 3282*	*ASTM D7928*	*ASTM D854*	*ASTM D1557*	*ASTM D4318*	*ASTM D4318*	*ASTM D1557*	*ASTM D1883*	*BS EN 1377-3*							*ASTM D2937*	*ASTM D1556*	*ASTM D4429*	*ASTM E2583*				
A-1-b-8 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-1-b-12 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-1-b-16 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-3-8 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-3-12 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-3-16 NOP	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	6	6	6	6
A-7-6-8 NOP	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	4	--	6	6
A-7-6-12 NOP	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	4	--	6	6
A-7-6-16 NOP	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	4	--	6	6
Al-Takahe district	✓	✓		✓	✓	✓	✓	✓	✓	✓						✓	✓			✓	20	--	--	20
Al-Emam Ali district	✓	✓		✓	✓	✓	✓	✓	✓	✓						✓	✓			✓	6	--	--	6

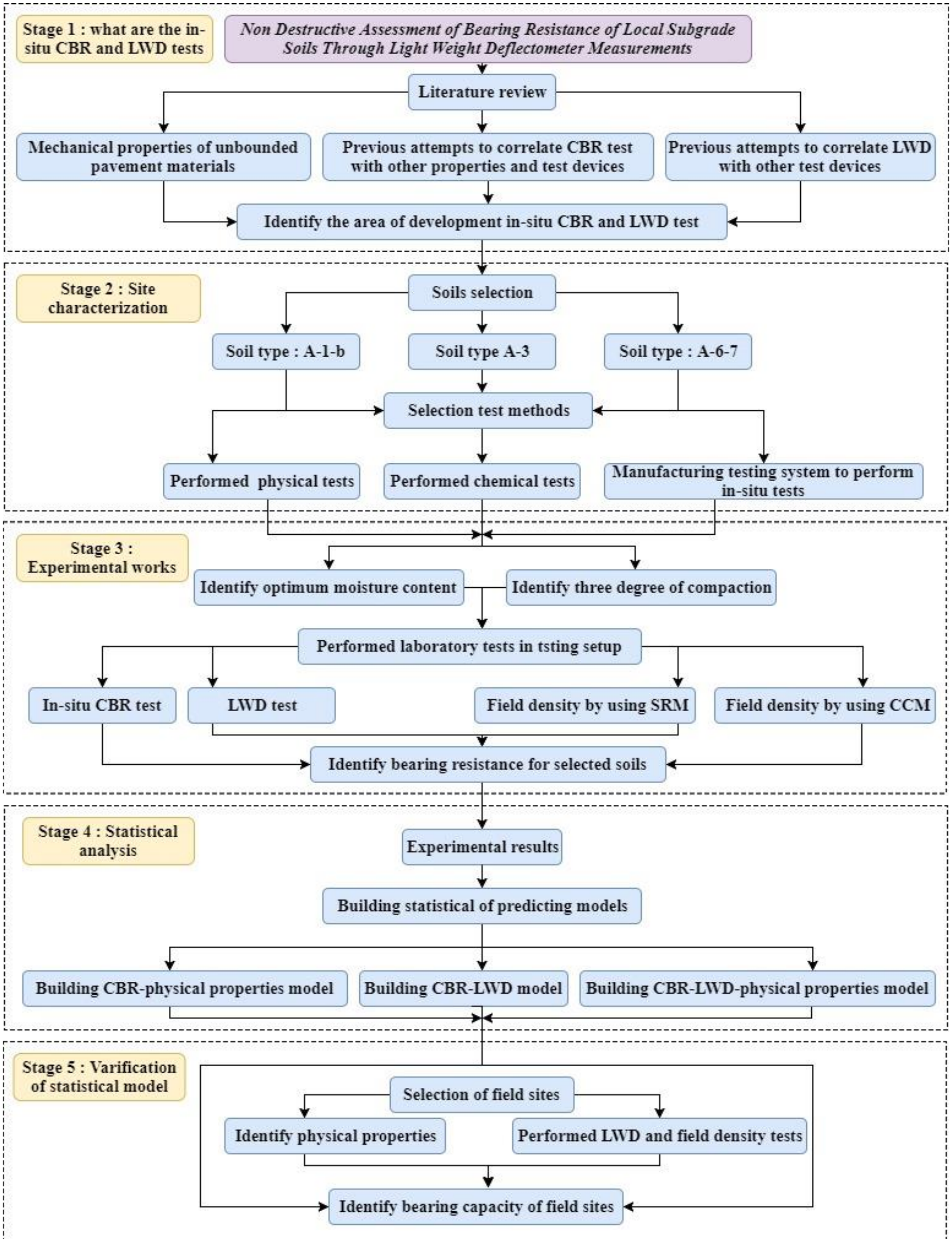


Figure 3.25: Schematic diagram of the research methodology.

3.7 Summary

This chapter explains the characterization of the sites selection, types of subgrade soils used in this research, loading system manufacturing, the methods used to test materials, and research methodology. On the other hand, tests are divided to basic physical and chemical tests and in-situ test. The basic physical tests were conducted to identify the basic characteristics of subgrade such as grain size particles distribution, standard and modified Proctor test, and, laboratory CBR test. While the chemical tests were performed for detection on chemical elements in the soil and identified the extension of their effect on the properties of the soil such as, gypsum, and Total Soluble Salts. The laboratory tests were included in-situ CBR, LWD, field densities were conducted by two test methods CCM and SRM to identify the bearing capacity of subgrade soil.

The methodology of this research included a comparison between different in-situ tests and correlation between them to identifying the acceptable and logical relationship to predict the bearing capacity of subgrade soil by using dynamic measurement obtained from light weight deflectometer test and basic physical properties of subgrade.

Chapter Four

LABORATORY TESTS RESULTS

4.1 Introduction

This chapter presents and discusses the results of experimental works for three types of subgrade soils: A-1-b, A-3, and A-7-6. Field density by CCM and SRM, CBR, and LWD were performed on these types of subgrade soils. 108 test results were collected from experimental works by CBR and LWD tests, while 48, 36 test results were collected by CCM and SRM to obtain the dry density and moisture content, respectively.

The investigation focuses on the influence of parameters such as type of soil, degree of compaction on the bearing ratio of different types of soils. The bearing ratio was obtained directly by CBR test and predicted by LWD parameters such as surface deflection on well compacted subgrade soil, dynamic modulus, and degree of compatibility. Two criteria were considered to achieve this purpose in the preparation of soil sample:

- Optimum moisture content obtained according to ASTM D1557 was adopted in soil mix process.
- Three degrees of compaction were adopted depending on numbers of compactor passing, i.e., 8, 12, and 16 NOP.

4.2 Field Density Results

Two test methods were adopted in this experimental work, core cutter method (CCM) according to ASTM D2937 and sand replacement method (SRM) according to ASTM D1556. Results of wet unit weight (γ_w), dry unit weight (γ_d) and degree of compaction (D_{oc}) for selected types of soil are explained below:

4.2.1 Subgrade Soil Type A-1-b

As mentioned previously two methods were adopted to determine γ_w, γ_d, and D_{oc} for 8 NOP for A-1-b subgrade soil. After completed tests for six test points, the results are shown in Table (4.1). These results illustrate that the γ_w obtained by CCM is varied from 1.693 gm/cm^3 to 1.858 gm/cm^3 with average value of 1.772 gm/cm^3, while the results obtained by SRM record higher value than that determined by CCM. These values are varied from 1.758 gm/cm^3 to 2.043 gm/cm^3 with average of 1.904 gm/cm^3. γ_d value is varied from 1.512 gm/cm^3 to1.637 gm/cm^3 with average of 1.576 gm/cm^3 by CCM and varied from 1.582 % to 1.775 % with an average 1.682 gm/ cm^3. Finally, D_{oc} was recorded average value of 84 % by CCM and 86% by SRM. After increasing numbers of compactor passing from 8 NOP to 12 NOP, then to 16 NOP the same trend is recognized with higher densities, as can be seen in Tables (4.2, and 4.3), The difference between values for tests results obtained by SRM than CCM is due to the SRM affected by several factors including: size of testing hole, density of Ottawa sand, and method of obtaining a disturbed soil sample from the hole. While, it is quite changeling to keep a granular soil sample intact when using cone cutter method.

Table 4.1: Summary of field density results by CCM and SRM for subgrade soil A-1-b, 8-NOP.

Test point No.	**Core cutter method**			**Sand replacement method**		
	γ_w (gm/cm^3)	**γ_d (gm/cm^3)**	**D_{oc} %**	**γ_w (gm/cm^3)**	**γ_d (gm/cm^3)**	**D_{oc} %**
P_1	1.820	1.622	86.07	1.914	1.704	87.22
P_2	1.719	1.557	82.60	1.912	1.686	86.30
P_3	1.695	1.517	80.47	1.890	1.655	84.72
P_4	1.848	1.637	86.84	2.043	1.775	90.86
P_5	1.858	1.609	85.35	1.910	1.689	86.44
P_6	1.693	1.512	80.20	1.758	1.582	80.99
Average	**1.772**	**1.576**	**84%**	**1.904**	**1.682**	**86%**

Table 4.2: Summary of field density results by CCM and SRM for subgrade soil A-1-b, 12-NOP.

Test point No.	Core cutter method			Sand replacement method		
	γ_w (gm/cm^3)	γ_d (gm/cm^3)	D_{oc} %	γ_w (gm/cm^3)	γ_d (gm/cm^3)	D_{oc} %
P_1	1.985	1.720	91.24	2.038	1.766	89.97
P_2	1.947	1.683	89.29	2.023	1.749	89.51
P_3	1.995	1.737	92.14	2.069	1.777	90.94
P_4	2.000	1.721	91.30	2.149	1.878	95.57
P_5	1.970	1.696	89.98	2.005	1.751	89.61
P_6	1.989	1.723	91.42	2.069	1.806	92.45
Average	**1.981**	**1.713**	**91%**	**2.059**	**1.788**	**91%**

Table 4.3: Summary of field density results by CCM and SRM for subgrade soil A-1-b, 16-NOP.

Test point No.	Core cutter method			Sand replacement method		
	γ_w (gm/cm^3)	γ_d (gm/cm^3)	D_{oc} %	γ_w (gm/cm^3)	γ_d (gm/cm^3)	D_{oc} %
P_1	2.120	1.831	97.13	2.124	1.853	98.28
P_2	2.087	1.829	97.05	2.123	1.843	97.77
P_3	2.137	1.832	97.20	2.218	1.855	98.39
P_4	1.959	1.791	95.03	2.127	1.828	96.95
P_5	2.142	1.859	98.62	2.170	1.860	98.68
P_6	2.137	1.863	98.84	2.076	1.861	98.75
Average	**2.097**	**1.834**	**97%**	**2.140**	**1.850**	**98%**

4.2.2 Subgrade Soil Type A-3

After completed tests for six test points were conducted for A-3 subgrade with 8 NOP, the results are listed in Table (4.4). These results illustrate the wet unit weight obtained by CCM is varied from 1.875 gm/cm^3 to 1.958 gm/cm^3 with average value of 1.930 gm/cm^3, while the results obtained by SRM record values varied from 1.924 gm/cm^3 to 1.983 gm/cm^3 with average of 1.958 gm/ cm^3. Dry unit weight value is varied from 1.772 gm/cm^3 to 1.858 gm/cm^3 with average of 1.826 gm/cm^3 by CCM and varied from 1.815 gm/cm^3 to 1.861 gm/cm^3 with an average of 1.851 gm/ cm^3. Finally, degree of compaction records an average value of 86 % by CCM and 87 % by SRM. After increasing numbers of compactor passing from 8 NOP to 12 NOP, then to 16 NOP, the results were recorded higher values than 8 NOP, as can be seen in Tables (4.5, and 4.6).

Table 4.4: Summary of field density results by CCM and SRM for subgrade soil A-3, 8-NOP.

Test point No.	**Core Cutter Method**			**Sand Replacement Method**		
	γ_w (gm/cm^3)	**γ_d (gm/cm^3)**	**D_{oc} %**	**γ_w (gm/cm^3)**	**γ_d (gm/cm^3)**	**D_{oc} %**
P_1	1.950	1.847	86.53	1.960	1.859	87.05
P_2	1.875	1.772	83.02	1.955	1.856	86.94
P_3	1.934	1.810	84.76	1.924	1.815	84.99
P_4	1.958	1.858	87.03	1.983	1.861	87.18
P_5	1.916	1.818	85.15	1.955	1.856	86.94
P_6	1.950	1.847	86.53	1.969	1.859	87.07
Average	**1.930**	**1.826**	**86%**	**1.958**	**1.851**	**87%**

Table 4.5: Summary of Field density results by CCM and SRM for subgrade soil A-3, 12-NOP.

Test point No.	**Core Cutter Method**			**Sand Replacement Method**		
	γ_w (gm/cm^3)	**γ_d (gm/cm^3)**	**D_{oc} %**	**γ_w (gm/cm^3)**	**γ_d (gm/cm^3)**	**D_{oc} %**
P_1	2.007	1.905	89.23	2.018	1.910	89.46
P_2	2.017	1.905	89.22	1.993	1.904	89.19
P_3	2.088	1.977	92.61	1.977	1.885	88.29
P_4	1.966	1.845	86.40	2.051	1.948	91.23
P_5	1.895	1.799	84.24	2.025	1.936	90.66
P_6	2.027	1.919	89.88	2.018	1.914	89.64
Average	**2.000**	**1.892**	**89%**	**2.014**	**1.916**	**90%**

Table 4.6: Field density results by CCM and SRM for subgrade soil A-3, 16-NOP.

Test point No.	**Core Cutter Method**			**Sand Replacement Method**		
	γ_w (gm/cm^3)	**γ_d (gm/cm^3)**	**D_{oc} %**	**γ_w (gm/cm^3)**	**γ_d (gm/cm^3)**	**D_{oc} %**
P_1	2.129	2.016	94.43%	2.110	1.987	93.09%
P_2	2.092	1.974	92.46%	2.086	1.968	92.16%
P_3	2.158	2.023	94.74%	2.167	2.032	95.16%
P_4	2.176	2.039	95.49%	2.167	2.039	95.52%
P_5	2.091	1.980	92.76%	2.100	1.971	92.30%
P_6	2.169	2.042	95.66%	2.196	2.055	96.26%
Average	**2.136**	**2.012**	**94%**	**2.137**	**2.009**	**94%**

4.2.3 **Subgrade Soil Type A-7-6**

According to ASTM D2937 was specified the validity of using CCM for cohesive subgrade soils with particle size less than 38 mm. for this reason CCM was adopted to determine γ_w, γ_d, and D_{oc}. The results of four test points were performed on subgrade soil type A-7-6 indicate that γ_w varied from 1.722 gm/cm^3 to 1.746 gm/cm^3, with an average of 1.733 gm/cm^3, γ_d ranged from 1.447 gm/cm^3 to 1.463 gm/cm^3 with an average 1.455 gm/cm^3, and average value of degree of compaction was 85%, when 8NOP was selected to compact subgrade layers. see Table (4.7). At the same trend the results were recorded for 12 NOP and 16 NOP as shown in both Tables (4.8) and (4.9).

Table 4.7: Summary of field density results by CCM for subgrade soil A-7-6, 8-NOP.

Test point No.	**γ_w (gm/cm^3)**	**γ_d(gm/cm^3)**	**D_{oc} %**
P_1	1.722	1.447	84.12
P_2	1.746	1.460	84.91
P_3	1.727	1.463	85.06
P_4	1.736	1.451	84.35
Average	**1.733**	**1.455**	**85%**

Table 4.8: Summary of field density results by CCM for subgrade soil A-7-6, 12-NOP.

Test point No.	**γ_w (gm/cm^3)**	**γ_d(gm/cm^3)**	**D_{oc} %**
P_1	1.797	1.516	88.15
P_2	1.794	1.512	87.89
P_3	1.808	1.508	87.70
P_4	1.797	1.505	87.48
Average	**1.799**	**1.510**	**88%**

Table 4.9: Summary of field density results by CCM for subgrade soil A-7-6, 16-NOP.

Test point No.	**γ_w (gm/cm^3)**	**γ_d(gm/cm^3)**	**D_{oc} %**
P_1	1.868	1.570	91.30
P_3	1.893	1.564	90.91
P_4	1.851	1.518	88.26
P_6	1.886	1.552	90.23
Average	**1.875**	**1.551**	**90%**

The results of dry density which is determined by CCM and SRM methods for three types of soil demonstrated that the dry density and degree of compaction were influenced by the number of compactor passing as shown in Figures (4.1) and (4.2). Figures show the increment in NOP that lead to increase in dry density and degree of compaction and record higher values for three types of soils at 16 NOP than 12 NOP, and lastly 8 NOP, and shows A-3 subgrade soil is more influenced with the increasing of NOP than A-1-b, and lastly A-7-6.

The reason of this the increament is due to the effect of gypsum content in subgrade soil with percentage approximately 11% that agreed with Ahmad (2013) attributed this behavior to the role of gypsum particles as a filling material to the intergranular voids of the soil matrix, and Kuttah (2015) claimed that adding gypsum to soil is increase the maximum dry unit weight of the soil and decrease the optimum moisture content, but only for gypsum content ranging between 0% to 15%. After investigation can observe the field moisture content had an average 5.90% and record decreased with percentage 30% than optimum moisture content, that means complete agreement with the previous finding.

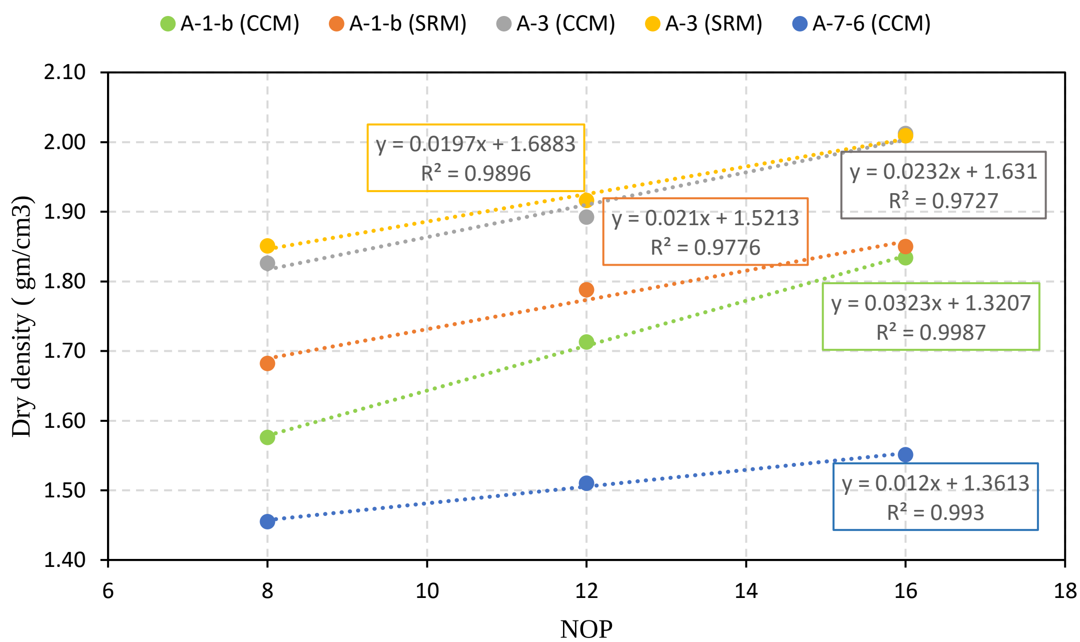


Figure 4.1: Relationship between dry density and NOP obtained by CCM and SRM for three types of soil.

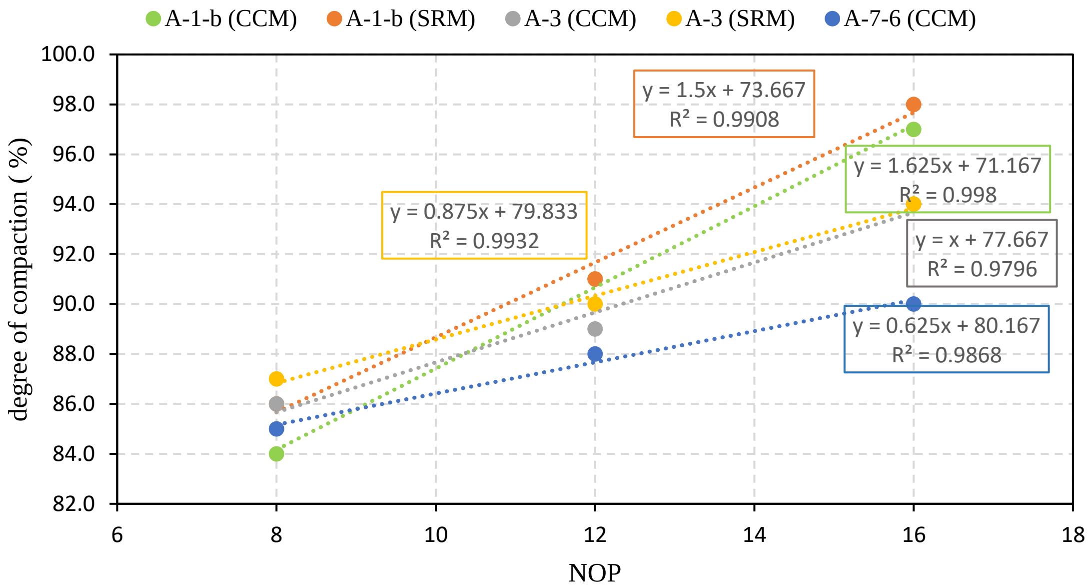


Figure 4.2: Relationship between D_{OC} and NOP obtained by CCM and SRM for three types of soil.

4.3 CBR Test Results

The standard testing procedure proposed by the American Society for Testing and Materials ASTM D 4429, was followed to determine CBR value as shown in Figures (4.3) to (4.5). The soil samples collected from three field sites were compacted at optimum moisture and at three compacted levels (low, medium, and high) for three layers with thickness of 20 cm each, similar to in-situ conditions.

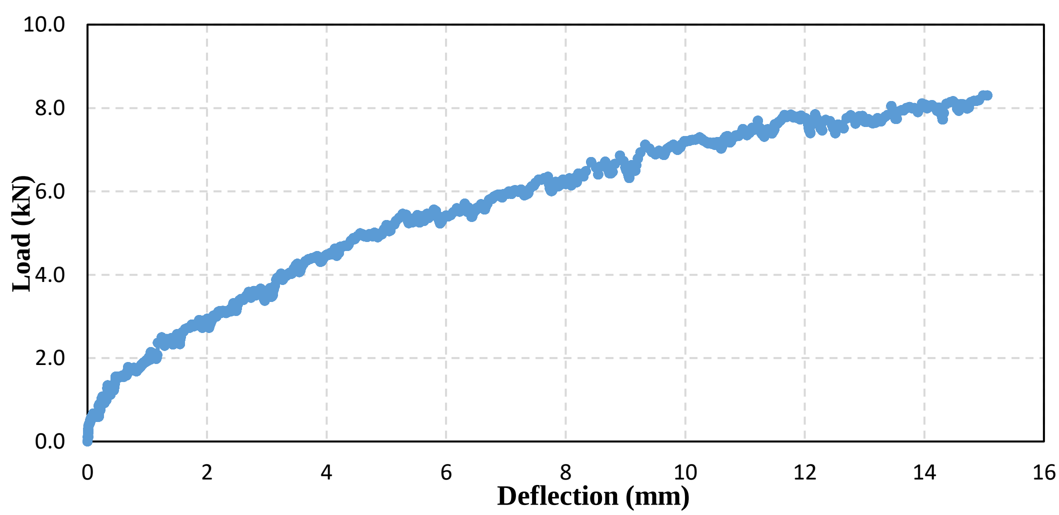


Figure 4.3: Actual load deflection curve.

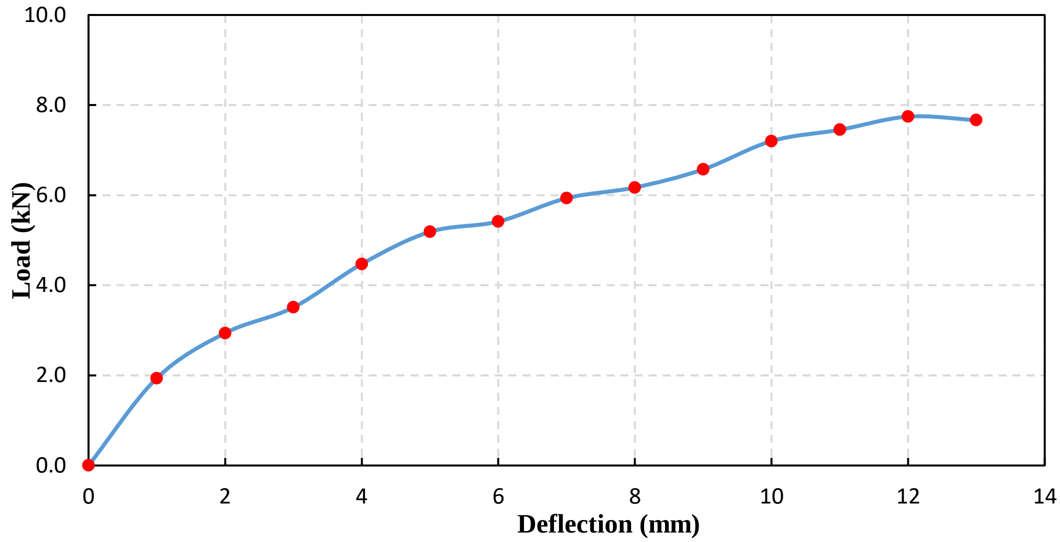


Figure 4.4: Reduced Load - Penetration Curve.

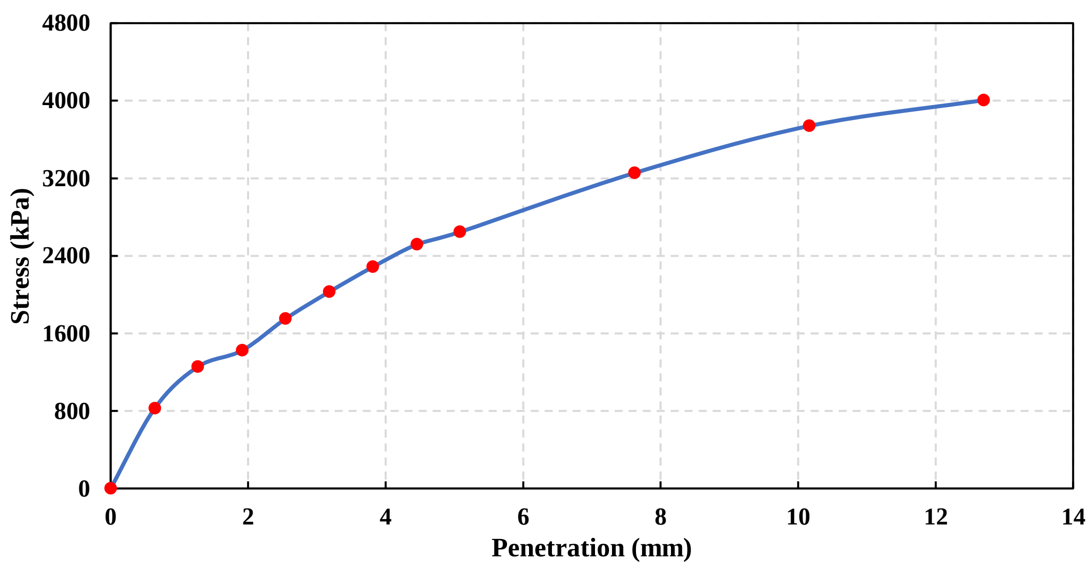


Figure 4.5: Stress-Penetration Curve.

4.3.1 **Subgrade Soil Type A-1-b**

The results collected from A-1-b soil type are varied from 16.671% to 22.680% with an average of 19.806% when number of passing equal 8 NOP. After increasing compacted level to 12 NOP, the CBR value vary from 21.392% to 25.687% with an average of 24.040%. Finally, when compacted level was increased to 16 NOP, that caused increased in CBR value at range from 27.519% to 38.569% with an average of 31.603%. Table (4.10) presents CBR results of A-1-b subgrade soil type.

Table 4.10: Summary of CBR Test for A-1-b subgrade soil.

Test point No.	**CBR%**		
	8-NOP	**12-NOP**	**16-NOP**
P1	21.547	23.637	28.457
P2	20.062	21.392	27.940
P3	16.735	24.920	29.529
P4	22.680	26.583	27.519
P5	21.140	22.021	38.569
P6	16.671	25.687	37.607
Average	**19.806**	**24.040**	**31.603**

Collected results for CBR value at three degrees of compaction indicate that the bearing ratio for A-1-6 subgrade soil was influenced by the increase in the amount of dry density. From this test procedure and data collected, the CBR value for A-1-b subgrade soil equal to 25.8% and 26.2% as obtained from Figure (4.6) and (4.7) which represent CBR value corresponding to dry density for each compactor levels obtained by CCM and SRM respectively.

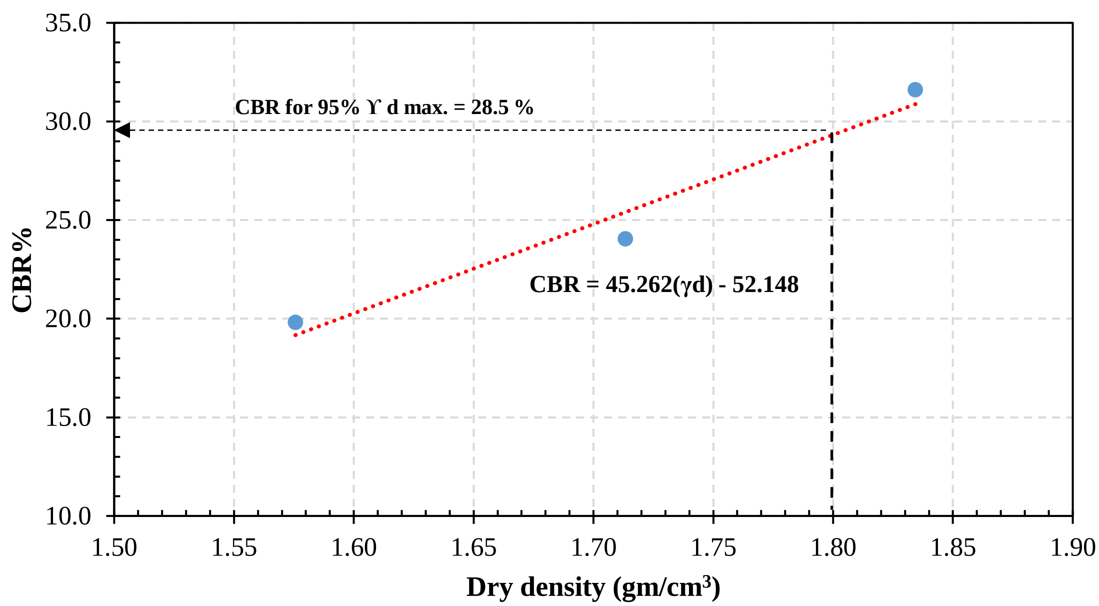


Figure 4.6: CBR chart for A-1-b subgrade soil by CCM.

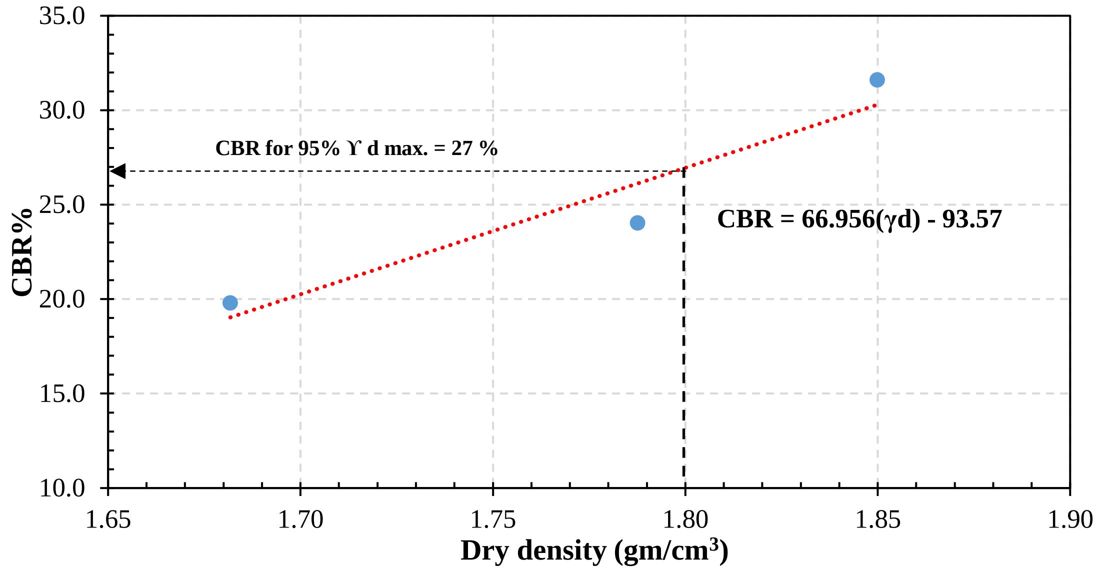


Figure 4.7: CBR chart for A-1-b subgrade soil by SRM.

4.3.2 Subgrade Soil Type A-3

The results collected from A-3 soil type which was provided from AL-Fares zone indicate that CBR values measured according to standard specification requirement ASTM D4429 at equivalent field conditions varied from 10.951 % to 15.195 % with an average of 13.043 % when number of passing equal 8-NOP. After increasing compacted level to 12-NOP, the CBR value varies from 19.714 % to 30.610 % with an average of 23.890 %. Finally, when compacted level was increased to 16-NOP, that caused increased in CBR value at range from 46.521% to 55.084 % with an average of 50.891%. Table (4.11) presents CBR results of A-3 subgrade soil type.

Table 4.11: Summary of CBR Test for A-3 subgrade soil.

Test point No.	**CBR%**		
	8-NOP	**12-NOP**	**16-NOP**
P_1	13.797	23.131	48.917
P_2	10.951	22.503	46.521
P_3	11.649	30.610	52.515
P_4	15.195	21.724	53.865
P_5	12.432	19.714	48.445
P_6	14.235	25.660	55.084
Average	**13.043**	**23.890**	**50.891**

Collected results for CBR value at three degrees of compaction indicate that the bearing ratio for A-3 subgrade soil was increased by the increment in the amount of dry density. From this test procedure and data collected, the CBR value for A-3 subgrade soil equal to 54% as obtained from Figure (4.8) and (4.9) which represent CBR value corresponding to dry density for each compactor levels obtained by CCM and SRM respectively. The results indicate that the CBR value for granular soils includes (A-1-b and A-3) which obtained in steel box is lower than the result of laboratory remolded samples results as shown in Table (3.2). the difference between these results is due to the difference in the environmental condition such as the temperature and evaporation effect, boundary condition of the laboratory tests, compaction equipment which is effect in the degree of compactions and hence will change in particles size distribution for compacted soils.

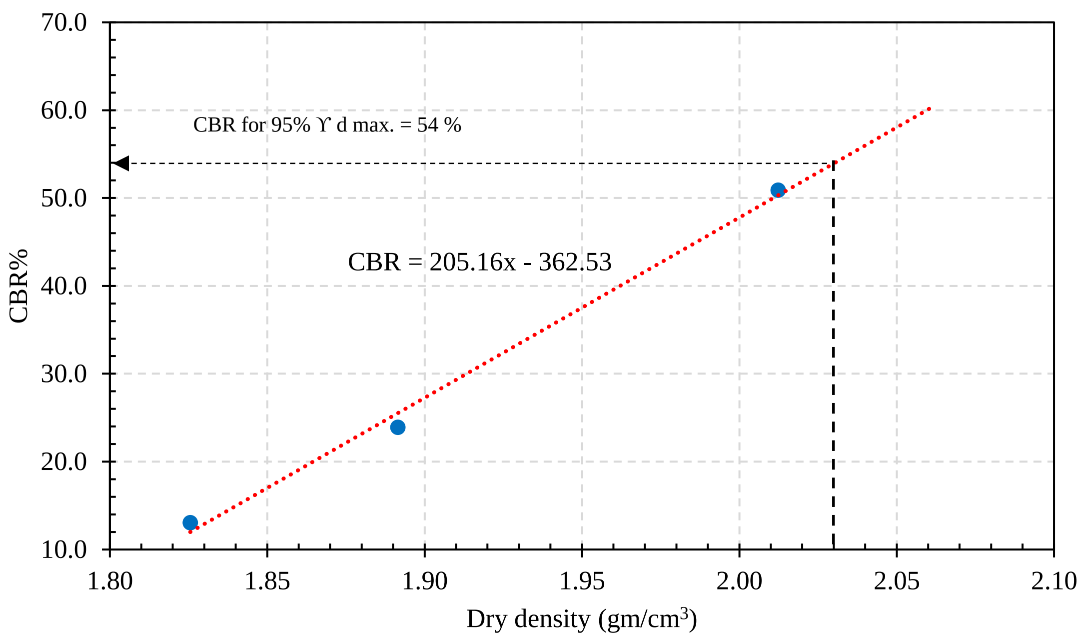


Figure 4.8: CBR chart for A-3 subgrade soil by CCM.

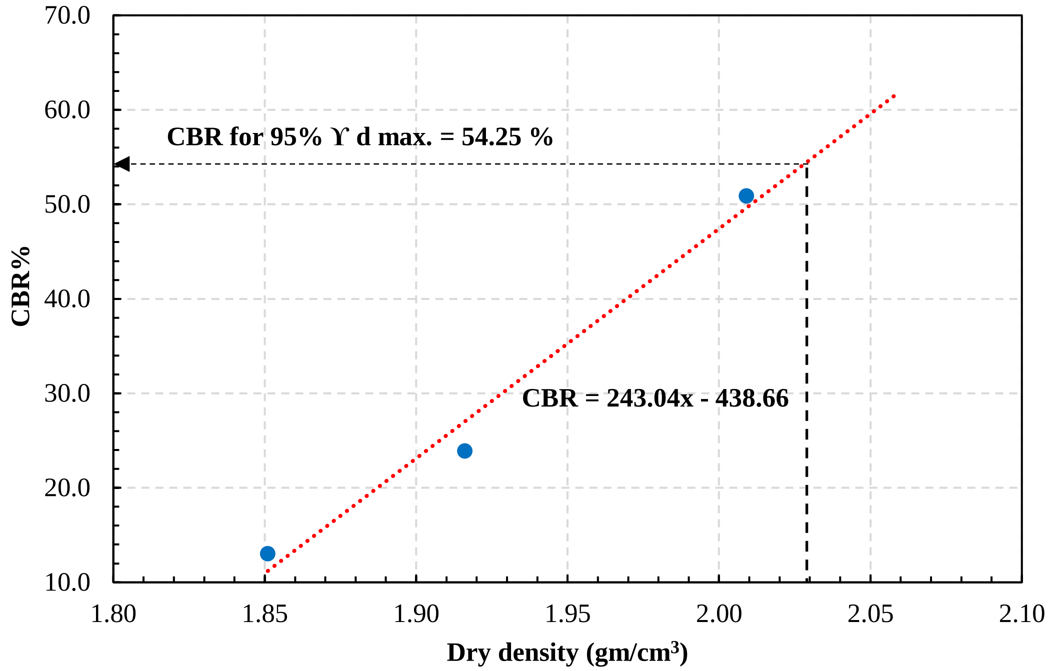


Figure 4.9: CBR chart for A-3 subgrade soil by SRM.

4.3.3 Subgrade Soil Type A-7-6

The results collected from A-7-6 soil type which was provided from AL-Rofaee zone indicate that CBR values varied from 5.995 % to 7.989 % with an average of 7.155 % when number of passing equal to 8-NOP. After increasing compacted level to 12-NOP, the CBR value varies from 9.462 % to 14.548 % with an average of 11.006 %. Finally, when compacted level was increased to 16-NOP, that caused increased in CBR value at range from 14.323 % to 19.967 % with an average of 16.821 %. Table (4.12) presents CBR results of A-7-6 subgrade soil type.

Table 4.12: Summary of CBR Test for A-7-6 subgrade soil.

Test point No.	CBR%		
	8-NOP	12-NOP	16-NOP
P1	5.996	14.548	18.122
P2	7.665	10.087	14.350
P3	7.011	10.207	19.967
P4	7.989	10.062	16.346
P5	6.614	11.674	14.323
P6	7.653	9.462	17.819
Average	**7.155**	**11.006**	**16.821**

Collected results for CBR value at three degrees of compaction indicate that the bearing ratio for A-7-6 subgrade soil increased by the increment that occurred in the amount of dry density and recorded lower values than A-1-b and A-3. Soil samples classified according to grain size particles distribution as fine grained soil which having F_{200} greater than 50% and coarse grained soil which having F_{200} less than 50 %, it was observed as presented previously in Table (3.2), that an increase in fines led to increase in the amount of optimum moisture content and decrease in the maximum dry density, hence the CBR value tend to decrease (Rahman at el. 2017). These lower values agreed with George and Kumar (2018). CBR value for A-7-6 subgrade soil can be obtained from Figure (4.10) which represent CBR value corresponding to dry density for each compactor levels obtained by CCM.

The relation between CBR and number of compactors passing on subgrade soil for the three types of soil are summarized in Figure (4.11). The granular soils exhibit higher bearing resistance, also, the Figures (4.11) and (4.12) shows that the increase in compaction effort from 8 to 12 then to 16, lead to an increase in the degree of compaction and record higher CBR values. A-3 subgrade soil shows more influence with the increasing of compaction effort than A-1-b, and lastly A-7-6.

The increment in bearing resistance for A-3 subgrade soil might be due to the effect of gypsum content in this type of soil, that agreed completely with Ahmed (2013) reported that the compressive strength values for poorly graded sandy soil samples stabilized with recycled gypsum increased from 14.42 kPa to 25.43, 81.99 and 331.18 kPa due to adding 5%, 10% and 20% content of recycled gypsum, respectively. This can be explained by the addition of recycled gypsum to the soil causing cementation or hardening of soil particles; thus, cohesion strength between soil particles is developed.

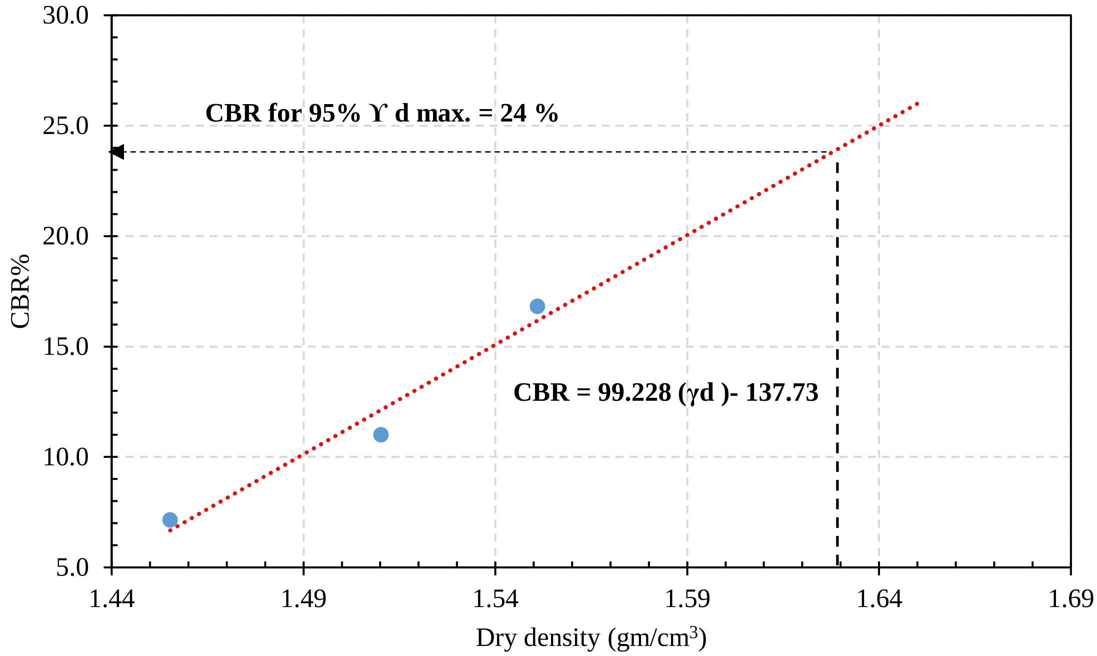


Figure 4.10: CBR chart for A-7-6 subgrade soil by CCM.

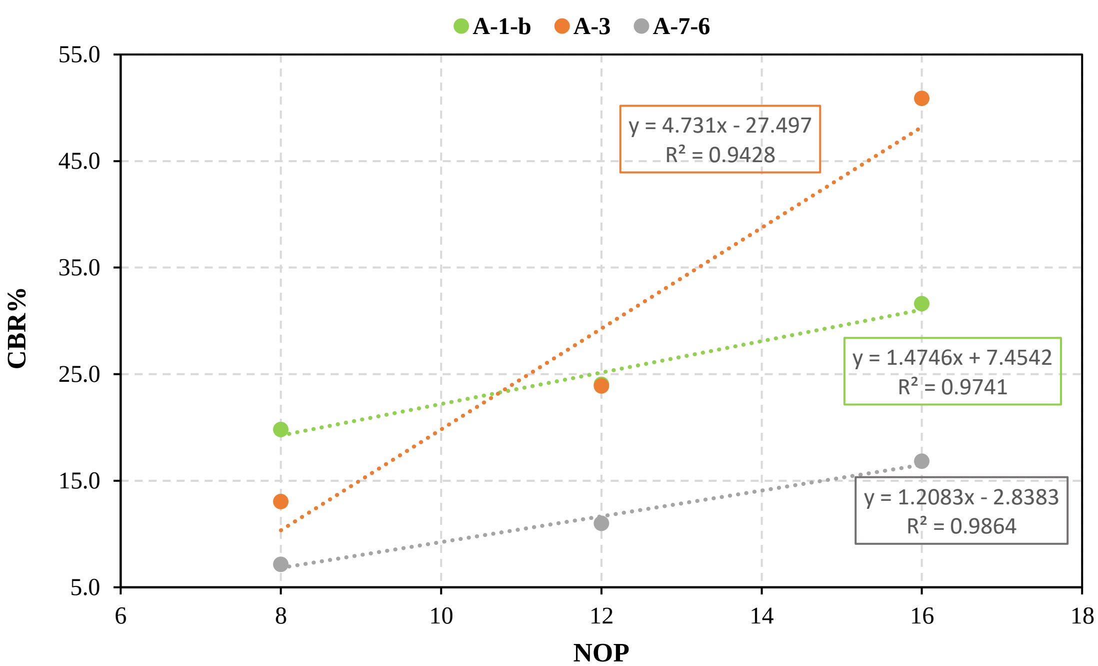


Figure 4.11: Relationship between CBR and NOP for three types of soil.

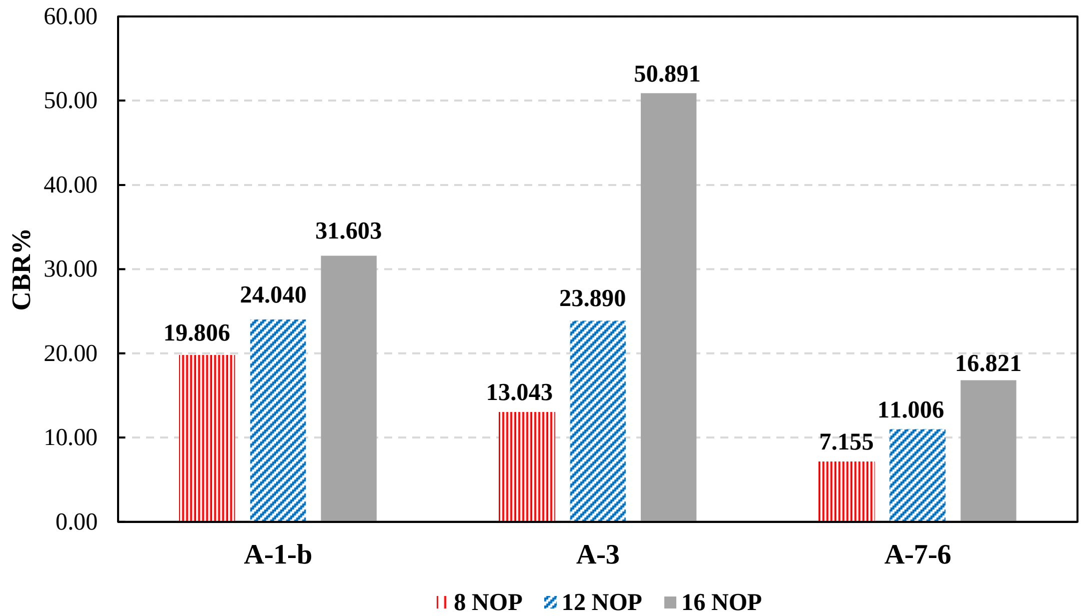


Figure 4.12: Average of bearing resistance values for three types of soil.

4.4 LWD Test Results

The dynamic properties of subgrade layers were obtained by using the LWD test for the three different types of soils as mentioned previously. LWD parameters measured during this research were surface deflection (mm), dynamic modulus (kPa), and degree of compatibility (ms). LWD test was performed according to standard specification requirement ASTM D 2583 for selected types of soil and three degrees of compaction for each soil type. Collected LWD test results were illustrated in the subsection below:

4.4.1 Subgrade Soil Type A-1-b

The results of six LWD test points were performed on a well compacted subgrade soil type A-1-b. The data extracted from the integration process indicate that surface deflection varied from 0.644 mm to 0.953 mm, with an average deflection of 0.786 mm, the values of dynamic modulus ranged from 23.61 MPa to 34.94 MPa with an average of 29.225 MPa, and average value of degree of compatibility of subgrade soil was 4.115 ms, when 8NOP was selected to compact subgrade layers. see Table (4.13). At the same trend the results were recorded for 12 NOP and 16 NOP with average 0.69 mm, 33.018 MPa, 3.672 ms and 0.632 mm, 36.122 MPa, 3.234 ms for S_d, E_d, and D_c respectively. See Tables (4.14) and (4.15).

Table 4.13: Summary of LWD test results of subgrade soil A-1-b, 8-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	0.73	0.716	0.72	0.722	31.16	4.245
P2	0.832	0.829	0.829	0.830	27.11	4.718
P3	0.912	0.879	0.874	0.888	25.34	3.969
P4	0.691	0.676	0.666	0.678	33.19	3.673
P5	0.665	0.628	0.639	0.644	34.94	3.423
P6	0.977	0.955	0.928	0.953	23.61	4.660
Average	**0.801**	**0.781**	**0.776**	**0.786**	**29.225**	**4.115**

Table 4.14: Summary of LWD test results of subgrade soil A-1-b, 12-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	0.748	0.735	0.732	0.738	30.49	3.599
P2	0.847	0.849	0.847	0.848	26.53	4.186
P3	0.654	0.649	0.637	0.647	34.78	3.319
P4	0.617	0.596	0.609	0.607	37.07	3.771
P5	0.662	0.652	0.663	0.659	34.14	3.862
P6	0.652	0.644	0.628	0.641	35.1	3.295
Average	**0.697**	**0.688**	**0.686**	**0.690**	**33.018**	**3.672**

Table 4.15: Summary of LWD test results of subgrade soil A-1-b, 16-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	0.675	0.665	0.638	0.659	34.14	2.903
P2	0.731	0.712	0.711	0.718	31.34	3.371
P3	0.663	0.659	0.644	0.655	34.35	3.459
P4	0.716	0.696	0.684	0.699	32.19	3.071
P5	0.554	0.541	0.533	0.543	41.44	3.487
P6	0.545	0.501	0.513	0.520	43.27	3.112
Average	**0.647**	**0.629**	**0.621**	**0.632**	**36.122**	**3.234**

The results of CBR and LWD parameters such as surface deflection and dynamic modulus clarified in Figure (4.13) and Figure (4.14). 1st figure shows a reduction in the S_d value with higher value of CBR %, which varied from 0.52 mm to 0.953 mm. while 2nd figure shows an increment in the E_d value with higher value of CBR% with value varied from 23.61 Mpa to 43.27 Mpa. These results indicate a good relationship between S_d vs. CBR and E_d vs. CBR.

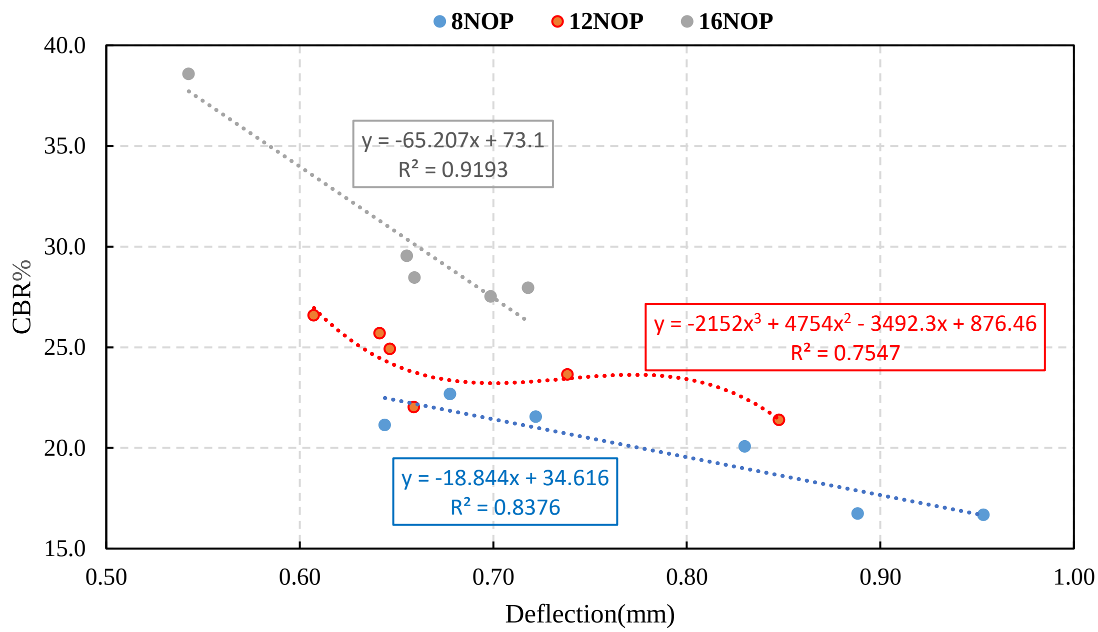


Figure 4.13: Relationship between CBR and surface deflection for A-1-b subgrade soil.

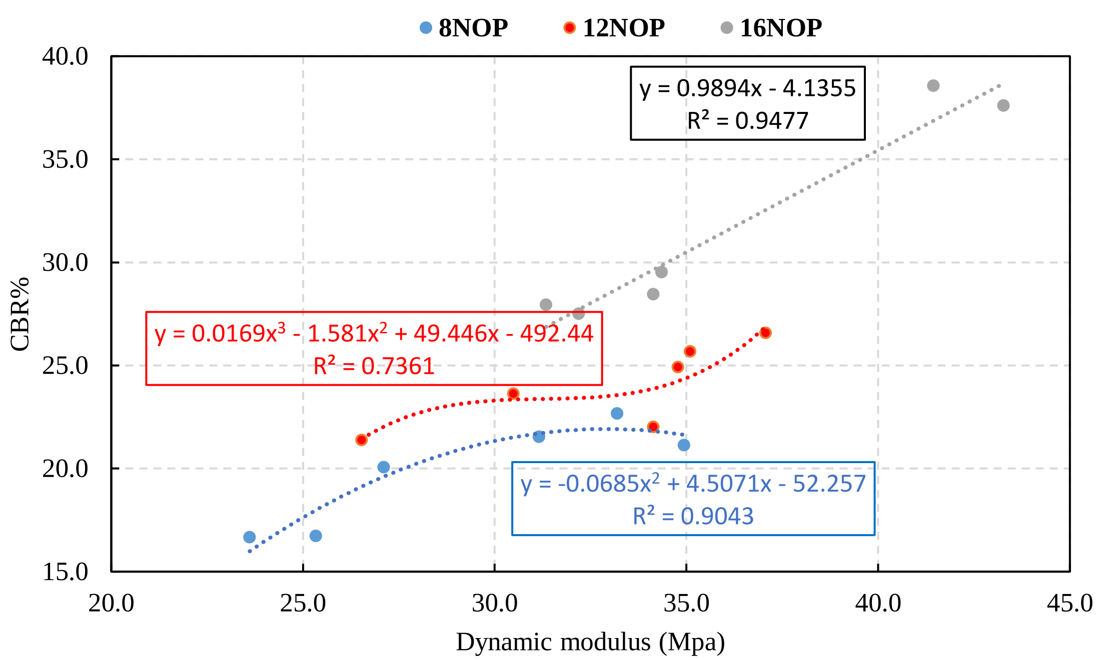


Figure 4.14: Relationship between CBR and surface deflection for A-1-b subgrade soil.

4.4.2 Subgrade Soil Type A-3

The results of six LWD test points were performed on subgrade soil type A-3. Test results indicate that S_d varied from 0.760 mm to 0.956 mm, with an average 0.872 mm, E_d ranged from 23.54 MPa to 29.61 MPa with an average 26.005 MPa, and average value of D_c of subgrade soil was 4.34 ms, when 8NOP was selected to compact subgrade layers. see Table (4.16). After increasing number of compactor passing from 8 NOP to 12 NOP, then to 16 NOP the same trend is recognized with higher values, as can be seen in Tables (4.17, and 4.18). The relation between CBR and LWD parameters showed a same behavior of A-1-b soil type which present decreasing in the surface deflection and increasing in the dynamic modulus with the increasing in CBR value. See Figures (4.14) and (4.15).

Table 4.16: Summary of LWD test results of subgrade soil A-3, 8-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	0.876	0.86	0.865	0.867	25.95	4.578
P2	0.98	0.944	0.945	0.956	23.54	4.516
P3	0.971	0.934	0.923	0.943	23.86	4.446
P4	0.782	0.741	0.756	0.760	29.61	3.981
P5	0.944	0.919	0.914	0.926	24.3	4.552
P6	0.8	0.783	0.764	0.782	28.77	3.964
Average	**0.892**	**0.864**	**0.861**	**0.872**	**26.005**	**4.340**

Table 4.17: Summary of LWD test results of subgrade soil A-3, 12-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	0.663	0.652	0.64	0.652	34.51	4.159
P2	0.676	0.662	0.646	0.661	34.04	3.242
P3	0.621	0.597	0.595	0.604	37.25	3.239
P4	0.695	0.652	0.672	0.673	33.43	4.374
P5	0.724	0.696	0.685	0.702	32.05	4.216
P6	0.641	0.627	0.613	0.627	35.89	3.877
Average	**0.670**	**0.648**	**0.642**	**0.653**	**34.528**	**3.851**

Table 4.18:Summary of LWD test results of subgrade soil A-3, 16-NOP.

Test point No.	Surface Deflection (mm)				Ed (MPa)	Dc (ms)
	S_1	S_2	S_3	Mean		
P1	0.486	0.471	0.474	0.477	47.17	2.868
P2	0.587	0.576	0.576	0.580	38.79	3.241
P3	0.457	0.451	0.451	0.453	49.67	2.772
P4	0.423	0.423	0.413	0.420	53.57	2.962
P5	0.511	0.532	0.5	0.514	43.77	2.829
P6	0.405	0.385	0.384	0.391	57.54	2.887
Average	**0.478**	**0.473**	**0.466**	**0.473**	**48.418**	**2.927**

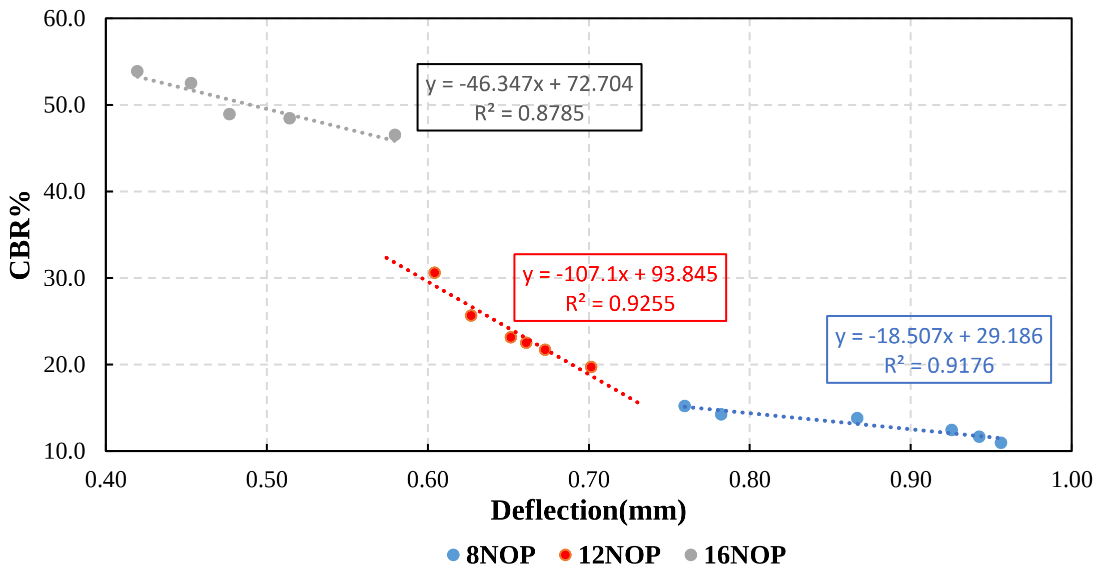


Figure 4.15: Relationship between CBR and surface deflection for A-3 subgrade soil.

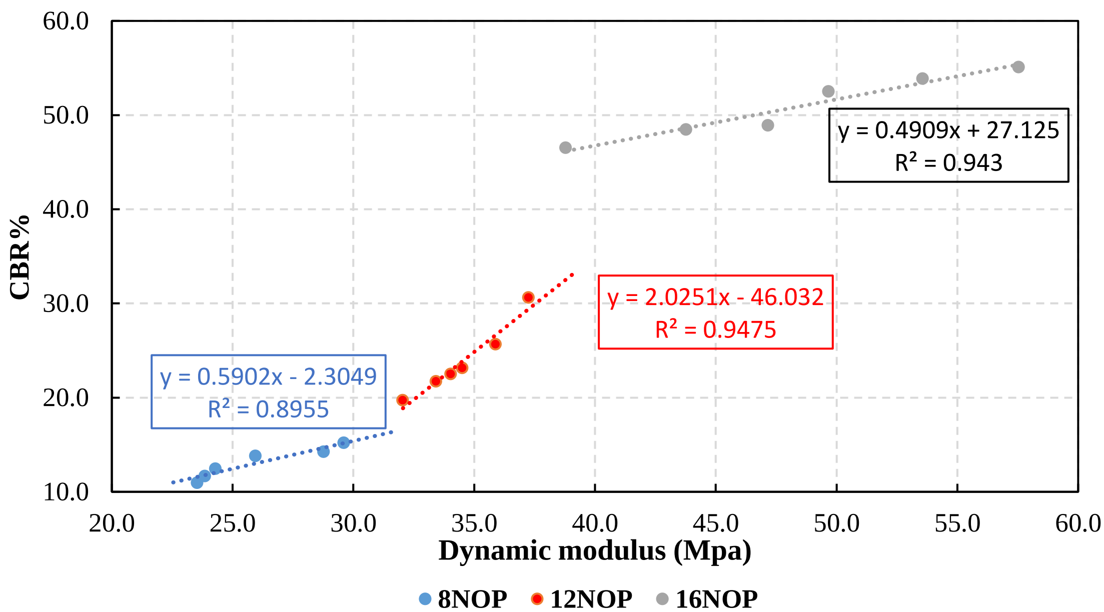


Figure 4.16: Relationship between CBR and dynamic modulus for A-3 subgrade soil.

4.4.3 **Subgrade Soil Type A-7-6.**

The results of six LWD test points were performed on subgrade soil type A-7-6. Test results indicate that vertical displacements varied from 1.706 mm to 1.814 mm, with an average 1.75 mm, dynamic modulus ranged from 12.4 MPa to 13.19 MPa with an average 12.862 MPa, and average value of degree of compatibility was 4.787 ms, when 8NOP was selected to compact subgrade layers, see Table (4.19). After increasing NOP from 8 NOP to 12 NOP, then to 16 NOP, the surface deflection, dynamic modulus, and degree of compatibility recorded higher values than the results was obtained at 8NOP, as can be seen in Tables (4.20, and 4.21). The results of CBR and LWD parameters clarified in Figures (4.17) and (4.18) and shows a reduction in the S_d value and an increment in the E_d with higher value of CBR %.

Table 4.19: Summary of LWD test results of subgrade soil A-7-6, 8-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P1	1.862	1.795	1.786	1.814	12.4	4.459
P2	1.748	1.741	1.72	1.736	12.96	4.591
P3	1.751	1.752	1.739	1.747	12.88	5.072
P4	1.708	1.702	1.709	1.706	13.19	4.794
P5	1.767	1.745	1.714	1.742	12.92	4.538
P6	1.767	1.746	1.752	1.755	12.82	5.266
Average	**1.767**	**1.747**	**1.737**	**1.750**	**12.862**	**4.787**

Table 4.20: Summary of LWD test results of subgrade soil A-7-6, 12-NOP.

Test point No.	**Surface Deflection (mm)**				**E_d (MPa)**	**D_c (ms)**
	S_1	**S_2**	**S_3**	**Mean**		
P_1	1.43	1.403	1.375	1.403	16.04	4.198
P_2	1.547	1.541	1.54	1.543	14.58	4.769
P_3	1.483	1.469	1.468	1.473	15.27	4.501
P_4	1.611	1.591	1.573	1.592	14.13	4.742
P_5	1.437	1.434	1.44	1.437	15.66	4.327
P_6	1.651	1.629	1.588	1.623	13.86	4.419
Average	**1.527**	**1.511**	**1.497**	**1.512**	**14.923**	**4.493**

Table 4.21: Summary of LWD test results of subgrade soil A-7-6, 16-NOP.

Test point No.	Surface Deflection (mm)				E_d (MPa)	D_c (ms)
	S_1	S_2	S_3	Mean		
P1	1.269	1.37	1.368	1.336	16.84	4.544
P2	1.467	1.447	1.44	1.451	15.51	4.681
P3	1.316	1.294	1.292	1.301	17.29	4.107
P4	1.381	1.39	1.373	1.381	16.29	4.539
P5	1.466	1.456	1.434	1.452	15.5	4.58
P6	1.387	1.36	1.358	1.368	16.45	4.395
Average	**1.381**	**1.386**	**1.378**	**1.382**	**16.313**	**4.474**

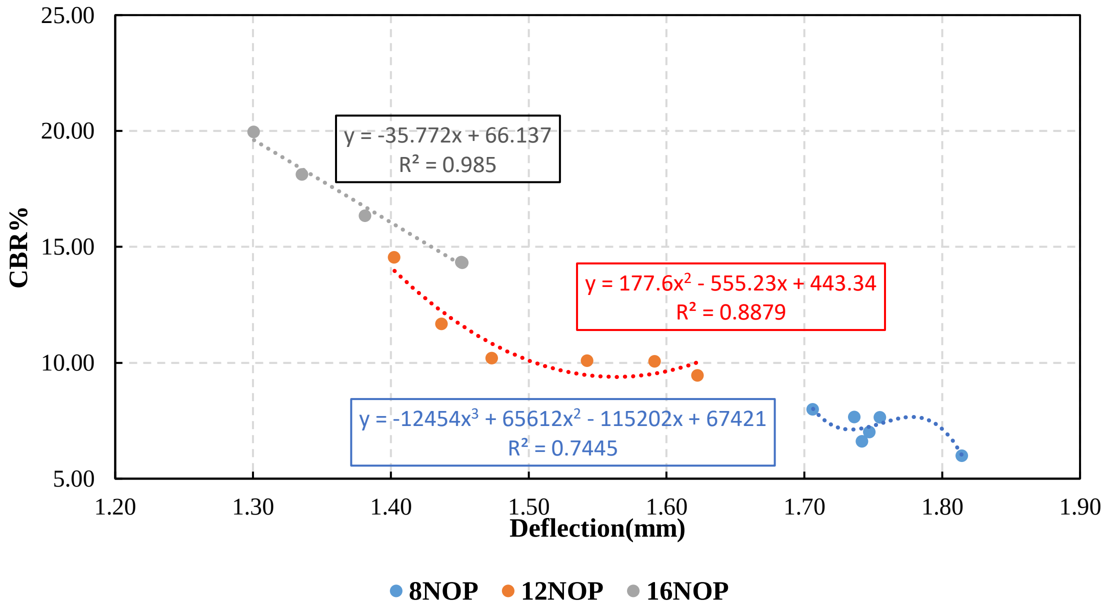


Figure 4.17: Relationship between CBR and surface deflection for A-7-6 subgrade soil.

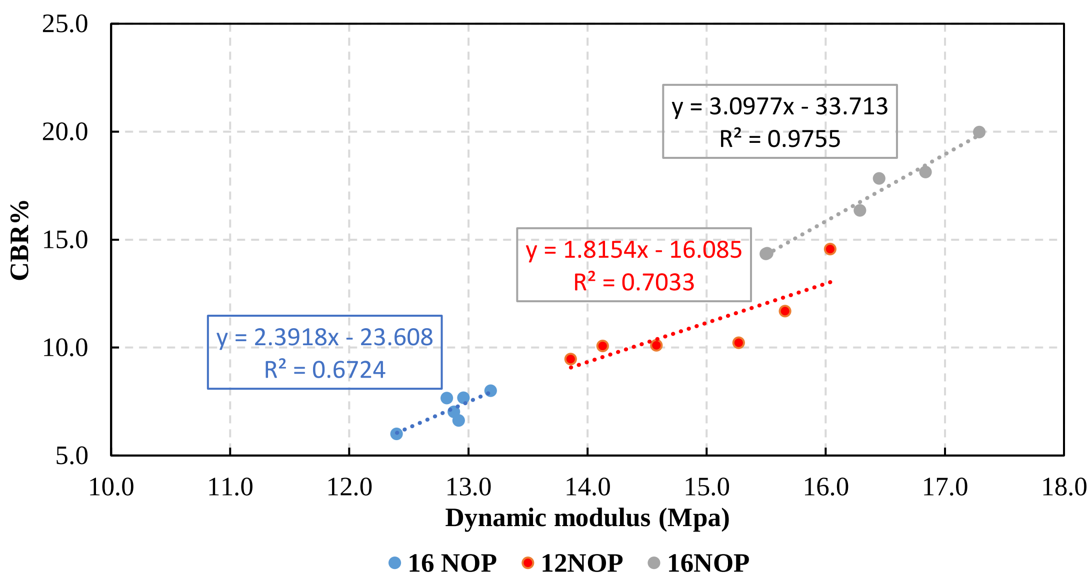


Figure 4.18: Relationship between CBR and dynamic modulus for A-7-6 subgrade soil.

After data collected for CBR and LWD tests, a good relationship was significant between CBR value for three types of soils and LWD parameters such represented by surface deflection, dynamic modulus and degree of compatibility at three compacted levels. Figure (4.19) clarifies the relationship between compaction effort and surface deflection and shows reduction in surface deflection with increasing in compaction effort for three type of soil, and illustrate A-7-6 subgrade soil records higher value of surface deflection than A-3, and lastly A-7-6.

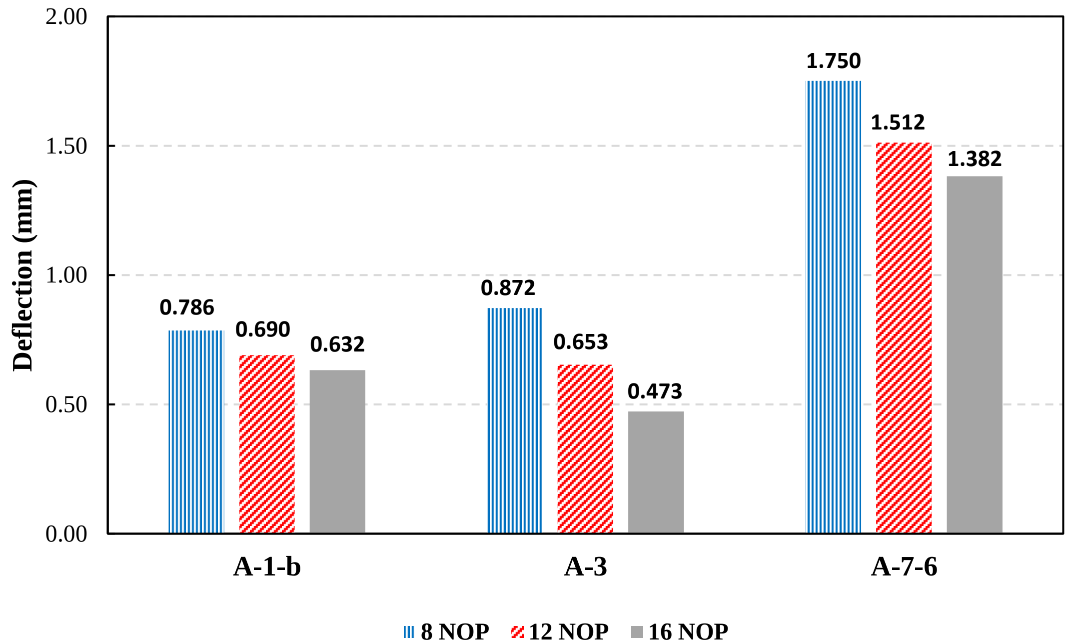


Figure 4.19: Relationship between surface deflection and degree of compaction.

While Figure (4.20) shows the effect of the compaction effort in dynamic modulus value for three types of soil, the increament in compaction effort that lead to increase in dynamic modulus and record higher dynamic modulus value for three types of soils at 16 NOP than 12 NOP, and lastly 8 NOP, and shows A-3 subgrade soil is more influenced with the increasing of compaction effort than A-1-b, and lastly A-7-6.

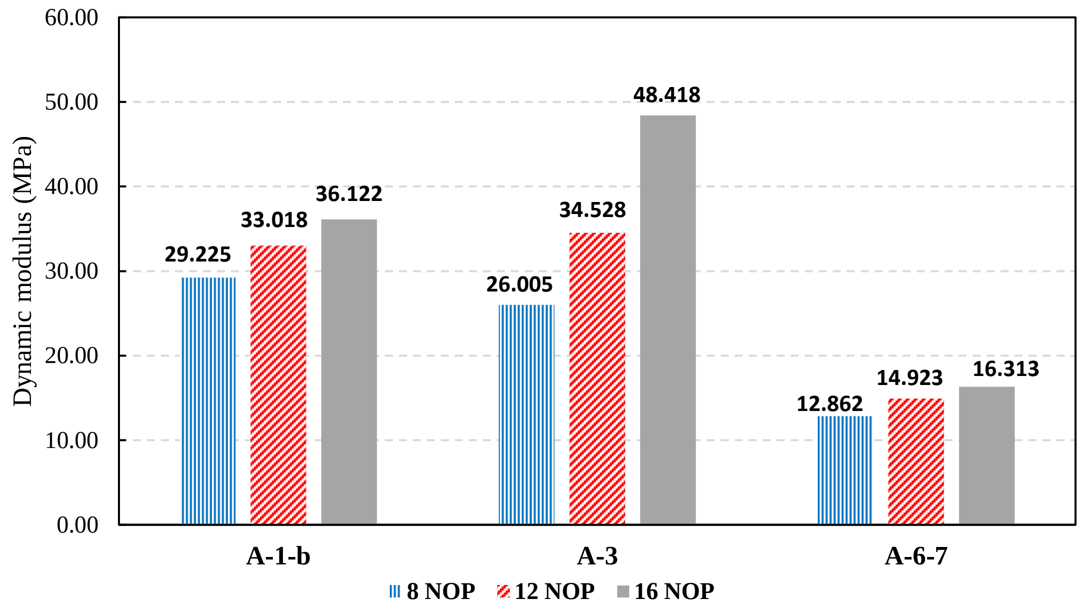


Figure 4.20: Relationship between dynamic modulus and degree of compaction.

Finally, Figure (4.21) shows the effect of the compaction effort with the degree of compatibility, a figure shows reduction in degree of compatibility with increasing in compaction effort for three type of soil, and illustrate A-7-6 subgrade soil records higher value than A-3, and lastly A-7-6.

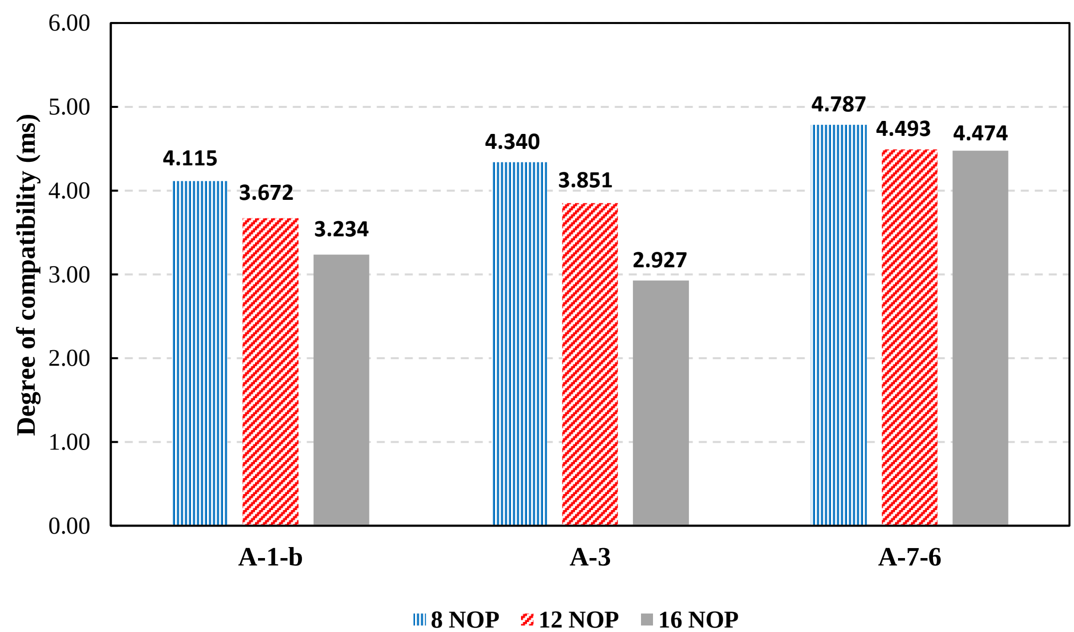


Figure 4.21: Relationship between degree of compatibility and degree of compaction.

4.5 **Summary**

The results of the conducted testing program for evaluating subgrade soils using different types and different tests, can be summarized in the following points:

1. Bearing ratio of subgrade soil was influenced with their basic physical properties such as particles size distribution, moisture content, and Atterberg limits in addition to test condition such as soaked and unsoaked, compaction effort, and confinement condition.
2. Bearing ratio of subgrade soil was influenced with the chemical elements such as gypsum content.
3. LWD parameters include surface deflection, dynamic modulus and degree of compaction influenced with the basic physical properties and degree of compaction for subgrade soils.
4. Test results clarify that there is correlation between CBR, LWD parameters and physical properties and need a statistical analyses program to analyze these results and find acceptable and logical correlation between these parameters as demonstrated in next chapter.

Chapter Five

STATISTICAL ANALYSIS

5.1 Introduction

A statistical model is a formalization of the relationships between variables in the form of mathematical equations. It describes how one or more random variables are related to one or more other variables. Statistical methods are used to improve the experimental methods, in which, instead of selecting one starting mix proportion and then adjusting by trial and error for achieving the optimum solution (Padmanaban, Kandasamy and Natesan, 2009). In this part of the study, the overall objective is to develop a predictive equation that correlate a dependent variable with an independent variable.

The collected results are 54 for each test of CBR, LWD, and 84 results obtained from field densities (γ_{df}) determined by two test methods (i.e., core cutter method and sand replacement method) for granular and clayey soil, respectively. Additionally, 24 and 12 parameters were obtained by conducting basic physical soil tests represented by (D_{10}, D_{30}, D_{60}, C_C, C_u, $G._S$, L.L, P.I, γ_{dmax}, O.M.C, $CBR_{unsoacked}$, and $CBR_{soacked}$) for granular soils and clayey soil respectively. However, because of the limitated space of this thesis one parameter only were selected (namely, CBR) represented as dependent variables, while the parameters (S_d, E_d, D_c, W_c, and γ_{df}). Three statistical models were developed to predict CBR value. The 1st model was developed using LWD parameters (E_d, S_d, D_c) as independent variables, the 2nd model was represented by considering (Wc, and γ_{df}) as independent variables for unbounded granular material clayey soils. The third model was proposed by taking several parameters W_c, and γ_{df} and LWD parameters as independent variables for both soil types. See Table (5.1)

Table 5-1: Dependent and independent variables considered in statistical analysis

Abbreviation	***Description***	***Unit***
	Dependent variable	
CBR	Calerfonia Bearing Ratio	%
	Independent variable	
S_d	Surface deflection	mm
E_d	Dynmic modulus	MPa
D_c	Degree of compatibility	ms
W_c	Water content	%
γ_{df}	Field dry density	gm/cm^3

In this study, a Statistical Package for The Social Science (SPSS) software (Version 25), SPSS is a window-based program was utilized to perform data entry, analysis and creating tables and graphs. It is capable of handling a large amount of data and can perform all of the analysis covered in the text and more. Therefore, the following section explained the basic concepts of statistical analyses, and discloses the analysis process for building and validating the prospective models.

5.2 Basic Concepts of Statistical Analysis

SPSS software provides many concepts of statistical analysis. The main concepts and their definitions are demonstrated below (Blunch, 2012):

5.2.1 Correlation Between Variables

Correlation is a statistical method that explains how strong the linking between two variables. The correlation coefficient is an index used to measure the accuracy of correlation. The value of the correlation coefficient ranges between -1 and 1. Correlation classified into three sets based on the numbers of variables and the relation between them. These three sets are; (1) negative and positive, (2) linear and nonlinear, and (3) simple and multiple. On the other hand, the correlation degree is classified into five types according to correlation coefficient value and expressed as perfect, high degree, moderate degree, low degree, and no correlation.

5.2.2 Regression Analysis.

Regression analysis is a statistical procedure for investigating any possible relationship between an independent and a dependent variable to predict the future values of the dependent variables. Three main types of regression analysis expressed by linear regression analysis, multiple regression analysis, and nonlinear regression analysis.

5.2.3 Some Definitions about Statistical and Goodness of Fit

To evaluate the performance of any model, it is required to understand some statistical parameters of models, which are stated as follows:

- **Coefficient of Determination (R^2):**

Expressed as R-squared, is a statistical measure of how close the data are to the fitted regression line. It is also known as the coefficient of multiple determination for multiple regression. The value of R^2 is always between 0% and 100%. When the model shows none of the variability of the response data around its mean, R-squared will equal to 0%, while if the model shows all the variability of the response data around its mean, in this case, R-squared will equal to 100%. In general, the higher the R-squared, the better model fits your data.

$$R^2 = 1 - \frac{SS_E}{SS_T} \qquad \text{--} \qquad (5.1)$$

Where :
SS_E: Error or residual sum of squares.
SS_T: Total of Sum squares.

- **The Residual (e):**

The difference between the observed value of the dependent variable (y) and predicted value (ŷ) is called the residual (e). Each data point has one residual.

$$e = y - \hat{y} \qquad \text{--} \qquad (5.2)$$

- **Analysis of Variance (ANOVA):**

The ANOVA is a parametric statistical technique used to compare datasets. It is used to compare means and the relative variance between them. However, analysis of variance (ANOVA) is best applied where more than 2 samples are meant to be compared.

- **Confidence interval:**

The confidence interval describes the amount of uncertainty associated with a sample estimate of a population parameter. The confidence interval can take any number of probabilities, with the most common being 95%.

5.3 Prediction Model

SPSS software was used to analyze and build predictive models. For the simplification purposes, linear models were firstly generated, unfortunately, all linear models were failed to represent the observations, for many trails it was found that all models were nonlinear. Three sets of nonlinear models were selected to correlate bearing ratio to LWD parameters, basic physical properties, and a combination of LWD parameters and basic physical properties. This selection was based on the importance of parameters, as it represents the bearing capacity of subgrade soil.

5.3.1 Building of CBR-LWD Model

For unbounded granular soils and clayey soils, it was assumed that the CBR is influenced by three LWD variables: surface deflection (S_d), dynamic modulus (E_d) and degree of compatibility (D_c). The results collected in experimental work from CBR and LWD tests divided randomly into 70% to generate the model and 30% to validate the model. The results of statistical analysis for both soil models are explained as follow:

5.3.1.1 Correlation CBR - LWD Parameters For Granular Soils

CBR value and LWD parameters (S_d, E_d, and D_c) were employed to build a model for predicting bearing ratio for granular soils. Analysis results of modeling are shown in Tables (5.2) to (5.6), this analysis includes bivariate Pearson correlation, test of normility, models expression, parameters estimation, ANOVA and goodness fitting between observed and predicted values.

Table (5.2) explains the bivariate Pearson correlation between variables was illustrated the correlation was presented between dependent variables (CBR) and independent variables (LWD parameters), and shows if the sign is positive stat, that mean the higher score on dependent is associated with higher score on independent variables. While the sign negative stat that mean the lower score of dependent variable with higher score of independent variables. In addition this table shows the E_d has the most significant correlation to bearing soil capacity, then S_d, and lastly D_c.

Table 5.2: Correlation between Variables CBR-LWD parameters-granular soils.

Correlations					
		CBR	S_d	E_d	D_c
CBR	***Pearson Correlation***	1	-.889**	.938**	-.803**
	Sig. (2-tailed)	/	.000	.000	.000
	N	36	36	36	36
S_d	***Pearson Correlation***	-.889**	1	-.960**	.828**
	Sig. (2-tailed)	.000	/	.000	.000
	N	36	36	36	36
E_d	***Pearson Correlation***	.938**	-.960**	1	-.789**
	Sig. (2-tailed)	.000	.000	/	.000
	N	36	36	36	36
D_c	***Pearson Correlation***	-.803**	.828**	-.789**	1
	Sig. (2-tailed)	.000	.000	.000	/
	N	36	36	36	36
**. Correlation is significant at the 0.01 level (2-tailed).					

Table (5.3) shows the test of normality for dependent and independent variables, an ecceptable distribution for input data.

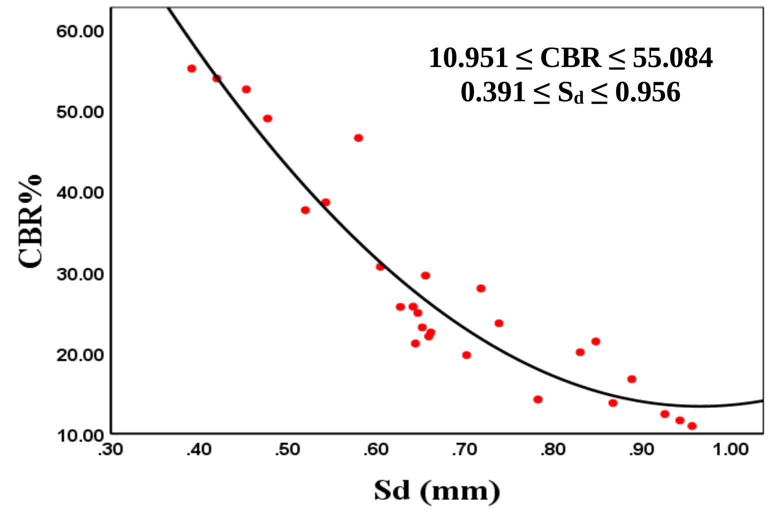
Table 5.3: Test of normality between Variables CBR-LWD parameters-granular soils.

Tests of Normality						
	Kolmogorov-Smirnov[a]			***Shapiro-Wilk***		
	Statistic	***df***	***Sig.***	***Statistic***	***df***	***Sig.***
CBR	.182	36	.004	.878	36	.001
S_d	.101	36	.200*	.965	36	.307
E_d	.168	36	.012	.918	36	.011
D_c	.103	36	.200*	.944	36	.070
*. This is a lower bound of the true significance.						
a. Lilliefors Significance Correction						

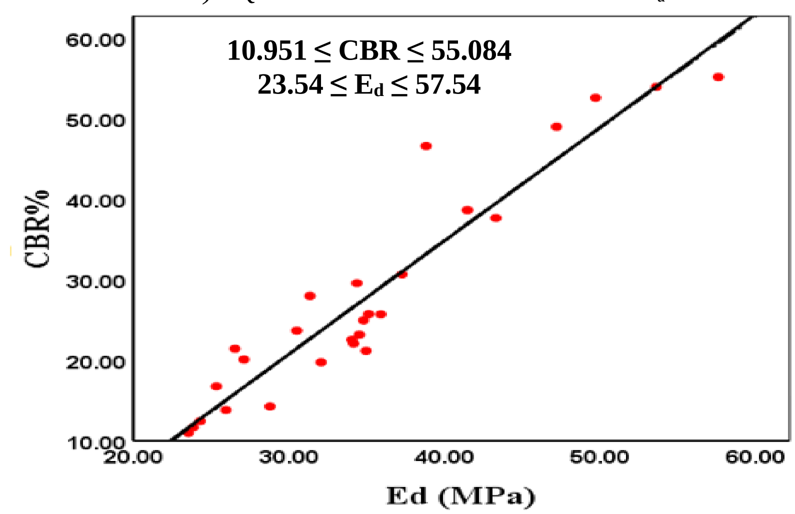
The analysis was implemented by separately investigating correlation of CBR value to each LWD variables. Three nonlinear correlations were developed using the principles of selected regression model where this model represented higher R^2 value among other models such as (linear, inverse, logarithmic, quadratic, cubic, power, … etc.) as shown in Table (5.4) and models' expression for these relations were shown in Figure (5.1).

Table 5.4: Summary of models and coefficients for Nonlinear CBR-LWD-granular soils parameters.

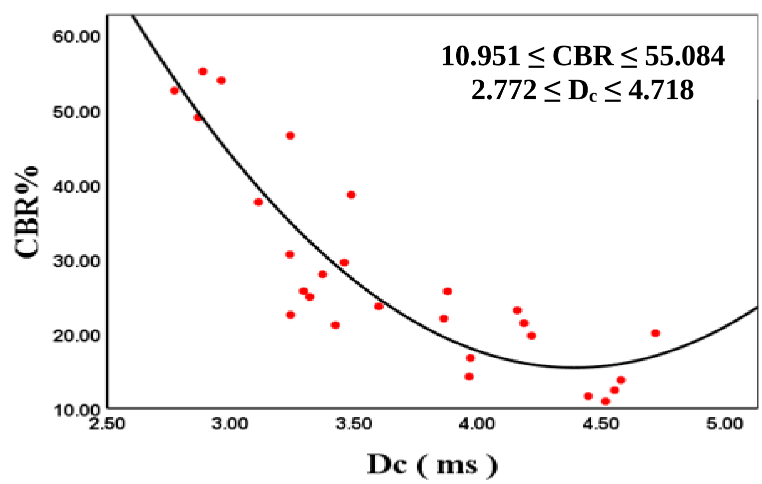
Ind. variable	***D. variable***	***Models expression***	R^2	***R-Adjusted***	***Std. Error***	***Estimated parameters***
CBR	S_d	$CBR = b_0 + (b_1 * S_d) + (b_2 * S_d{}^2)$	0.90	0.890	4.393	$b_0 = 140.477$ $b_1 = -263.295$ $b_2 = 136.363$
	E_d	$CBR = b_0 + (b_1 * E_d) + (b_2 * E_d{}^2)$	0.89	0.887	4.496	$b_0 = -22.667$ $b_1 = 1.471$ $b_2 = -0.001$
	D_c	$CBR = b_0 + (b_1 * D_c) + (b_2 * D_c{}^2)$	0.80	0.779	6.290	$b_0 = 299.796$ $b_1 = -129.5741$ $b_2 = 14.760$



a) Quadratic relation between CBR and S_d



b) Quadratic relation between CBR and E_d



c) Quadratic relation between CBR-D_c

Figure 5.1: Models' expression for Nonlinear CBR-LWD-granular soils parameters.

The developed model and its limitation with a confidence interval of 95% are illustrated in Table (5.5).

Table 5.5: Nonlinear CBR-LWD-granular soils parameters modeling.

Developed model	$CBR = b_0 + (b_1 * E_d^2) + (b_2 * D_c)$			
Parameter Estimates				
Parameter	***Estimate***	***Std. Error***	***95% Confidence Interval***	
			Lower Bound	***Upper Bound***
b_0	30.139	9.816	9.999	50.278
b_1	.014	.002	.010	.018
b_2	-5.529	2.101	-9.840	-1.218
Solution	$CBR\ Predicted = 30.139 + 0.014\ E_d^2 – 5.529\ D_c$			

ANOVA results are listed in Table (5.6) which explains that the MSE is low and residual sum of squares (SSE) is lower than Regression sum of squares (SSR), which is sustained the significant of the model. Additionally, the high value of the R^2 (0.899) indicates a good prediction.

Table 5.6: ANOVA for nonlinear CBR-LWD-granular soil model.

ANOVA[a]			
Source	***Sum of Squares***	***df***	***Mean Squares***
Regression	28391.170	3	9463.723
Residual	520.479	27	19.277
Uncorrected Total	28911.650	30	
Corrected Total	5084.896	29	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = 0.898			

A conclusion can be drawn that the developed model for predicting CBR in terms of LWD parameters from the remain records of validation is statistically reliable. Figure (5.2) illistitrates the relationship between the predicted and observed values and it can be seen the adequacy of the model and this figure indicates that an acceptable scatter can be recognized between predicted and observed CBR values with R^2 (0.94), and Figure (5.3) shows scatter plot for residual and predicted CBR value.

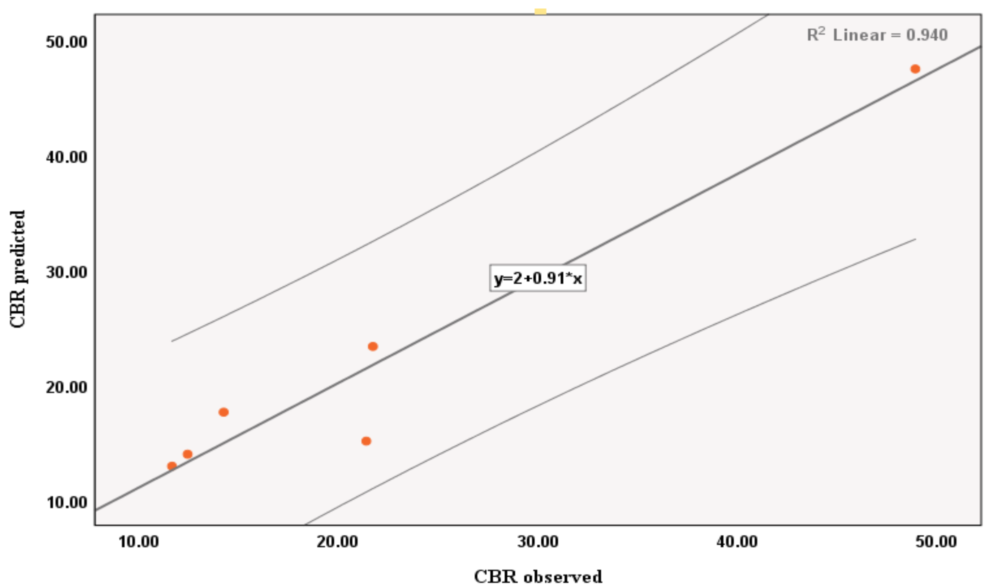


Figure 5.2: Comparisons between Measured and Predicted CBR for granular soils.

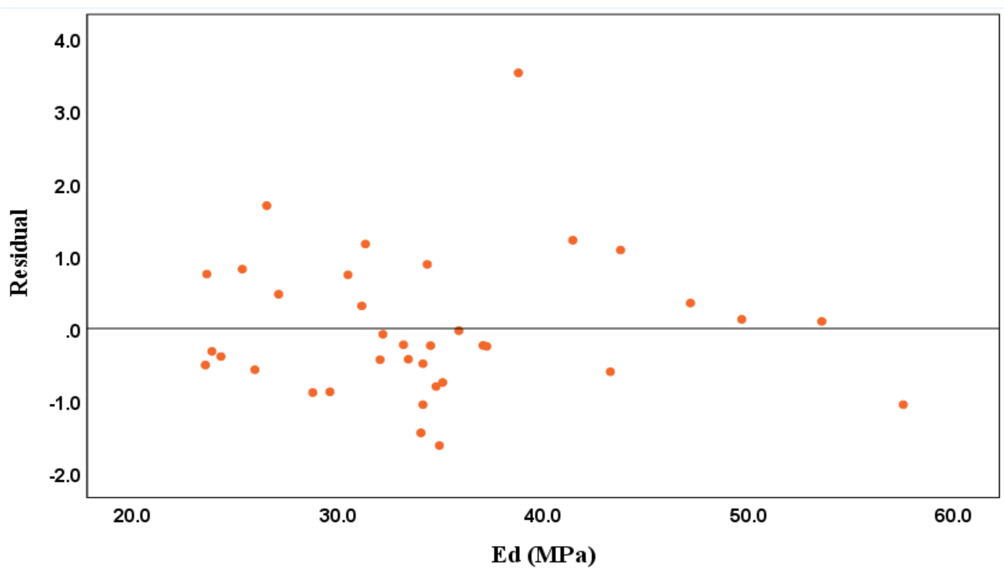


Figure 5.3: Scatter plot for residual and predicted CBR - granular soil.

5.3.1.2 **Correlation CBR and LWD Parameters For Clayey Soils**

CBR values and LWD parameters (S_d, E_d, and D_c) were employed to build a model between these parameters for clayey soils as same procedure of granular soil model. The results of regression analysis are shown in Tables (5.7) to (5.11). The bivariate Pearson correlation between variables shows that the independent variables had a correlation with dependent variable (CBR) and demonstrate that the E_d has the most significant correlation to CBR value, then S_d, and lastly D_c. See Table (5.5).

Table 5.7: Correlation between Variables for CBR-LWD parameters-clayey soils.

Correlations					
		CBR	***S_d***	***E_d***	***D_c***
CBR	***Pearson Correlation***	1	-.929**	.948**	-.571*
	Sig. (2-tailed)	/	.000	.000	.013
	N	18	18	18	18
S_d	***Pearson Correlation***	-.929**	1	-.997**	.587*
	Sig. (2-tailed)	.000	/	.000	.010
	N	18	18	18	18
E_d	***Pearson Correlation***	.948**	-.997**	1	-.600**
	Sig. (2-tailed)	.000	.000	/	.009
	N	18	18	18	18
D_c	***Pearson Correlation***	-.571*	.587*	-.600**	1
	Sig. (2-tailed)	.013	.010	.009	/
	N	18	18	18	18
**. Correlation is significant at the 0.01 level (2-tailed).					
*. Correlation is significant at the 0.05 level (2-tailed).					

Table (5.8) shows the test of normality for dependent and independent variables, and shows an ecceptable distribution for input data

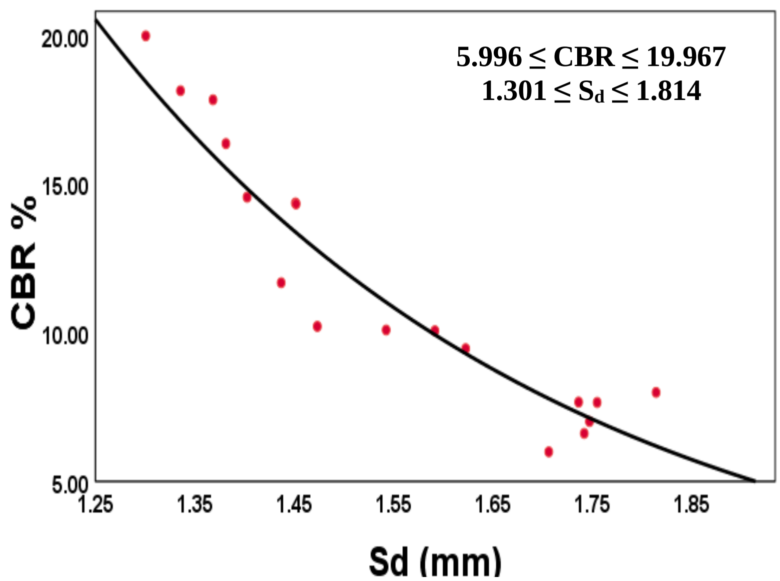
Table 5.8: Test of normality between Variables CBR-LWD parameters-clayey soils.

Tests of Normality						
	Kolmogorov-Smirnov[a]			*Shapiro-Wilk*		
	Statistic	*df*	*Sig.*	*Statistic*	*df*	*Sig.*
S_d	.171	18	.177	.915	18	.105
E_d	.162	18	.200*	.924	18	.150
D_c	.158	18	.200*	.951	18	.448
CBR	.185	18	.105	.921	18	.135
*. This is a lower bound of the true significance.						
a. Lilliefors Significance Correction						

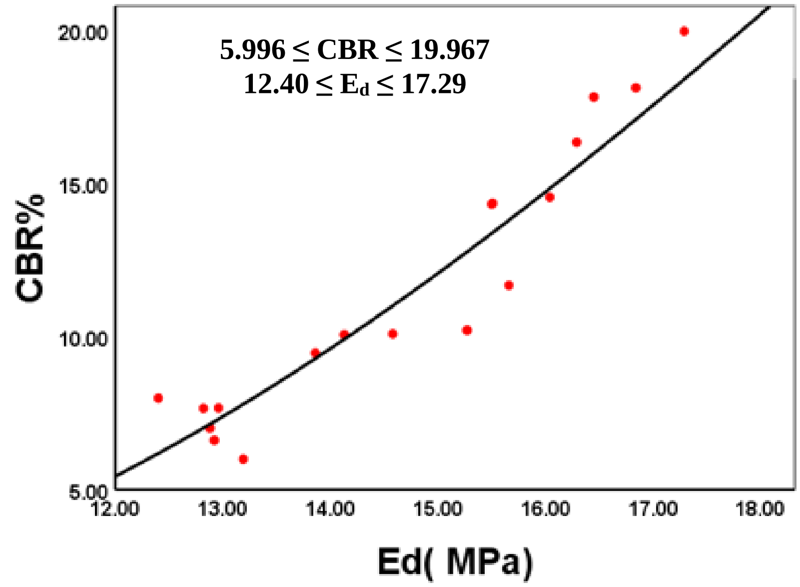
The analysis was implemented by separately investigating the correlation of CBR value to each LWD variables for clayey, model expression, R^2, R-adjusted, and standard of error for three nonlinear relations were shown in Table (5.9) and Figure (5.4).

Table 5.9: Summary of models and coefficients for Nonlinear CBR-LWD-clayey soils parameters.

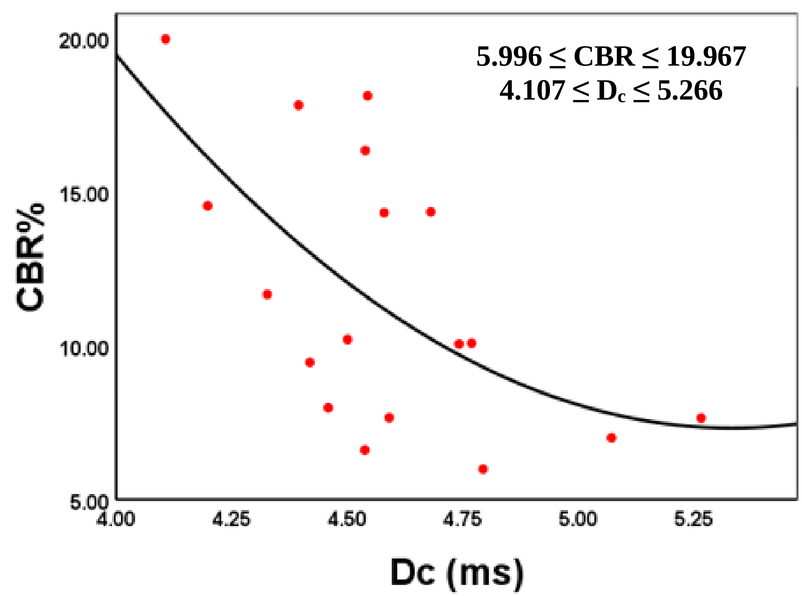
Ind. variable	***D. variable***	***Models expression***	R^2	***R adjusted***	***Std. Error***	***Estimated parameters***
CBR	S_d	$CBR = b_0 * (b_1{**}\ S_d)$	0.915	0.907	0.088	$b_0 = 240.288$ $b_1 = 0.146$
	E_d	$CBR = e^{(b_0 + (b_1/ E_d))}$	0.913	0.904	0.089	$b_0 = 5.321$ $b_1 = -43.35$
	D_c	$CBR = b_0 + (b_1 * D_c) + (b_2 * D_c^{\ 2}) + (b_3 * D_c^{\ 3})$	0.19	0.011	2.798	$b_0 = 198.83$ $b_1 = -77.397$ $b_2 = 7.863$



a) Compound relation between CBR-S_d



b) S relations between CBR-E_d



c) Cubic relationship between CBR-D_c

Figure 5.4: Models' expression for Nonlinear CBR-LWD-granular soils parameters.

The developed model and its limitation with confidence interval of 95% are illustrated in Table (5.10).

Table 5.10: Nonlinear CBR-LWD parameters modeling for clayey soils.

Developed model	$CBR = b_0 * E_d^2 + b_1$			
Parameter Estimates				
Parameter	**Estimate**	**Std. Error**	**95% Confidence Interval**	
			Lower Bound	**Upper Bound**
b_0	.090	.007	.075	.105
b_1	-8.011	1.542	-11.279	-4.743
Solution	CBR Predicted = $0.09\ E_d^2 - 8.011$			

Table (5.11) discloses that the sum of regression is higher than the sum of residue which is sustained the significant of the model. While, from the same table, the high value of the R-Square (0.914) indicates a perfect prediction.

Table 5.11: ANOVA for nonlinear (CBR-LWD-clayey soils) modeling.

ANOVA[a]			
Source	**Sum of Squares**	**df**	**Mean Squares**
Regression	2749.056	2	1374.528
Residual	28.399	16	1.775
Uncorrected Total	2777.455	18	
Corrected Total	329.933	17	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = 0.914			

A conclusion can draw the developed model for CBR-LWD parameters for clayey soil is acceptable. Figure (5.5) indicates that acceptable scatter can be recognized between predicted and observed operability values with R^2 (0.99), furthermore, almost all value within the significant level boundaries. While Figure (5.6) shows the scatter plot for residual and independent variable E_d and indicate model inadequacy due to low quantity in variables and outliers can have a dramatic impact on a regression model.

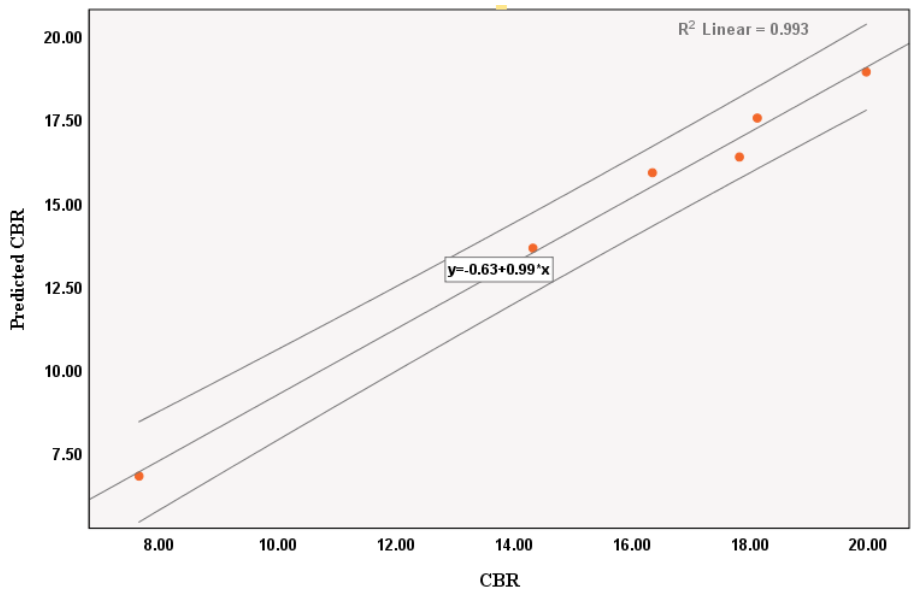


Figure 5.5: Comparisons between Measured and Predicted CBR for clayey soils

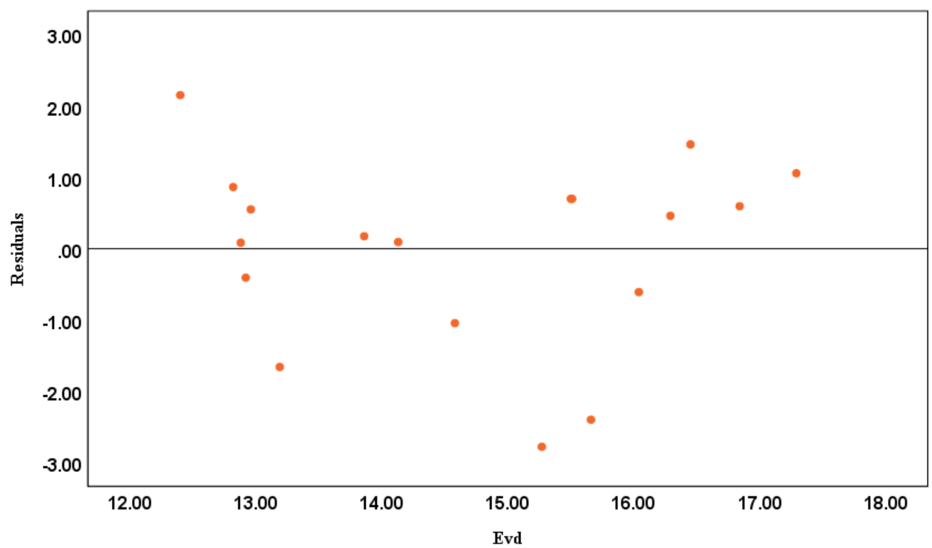


Figure 5.6: Scatter plot for residual and predicted CBR – clayey soils

5.3.2 **Building of CBR-Physical Properties Models**

As mentioned previously, (W_c, and γ_{df}) for granular and clayey soils were selected to develop a statistical model for predicting CBR. This selection is based on the most important parameters and their impact on the bearing capacity of the soils. The results were divided randomly 70% to generate and 30% to validate the model. The analysis results for both soils types were explained as follow:

5.3.2.1 Correlation CBR – Basic Physical Properties For Granular Soils

Correlating CBR for granular soils with (W_c), and (γ_{df}) was performed. The results of regression analysis are shown in Tables (5.12) to (5.15). The bivariate Pearson correlation between variables is shown in Table (5.12) and demonstrates that (γ_{df}) has the most significant correlation to CBR value.

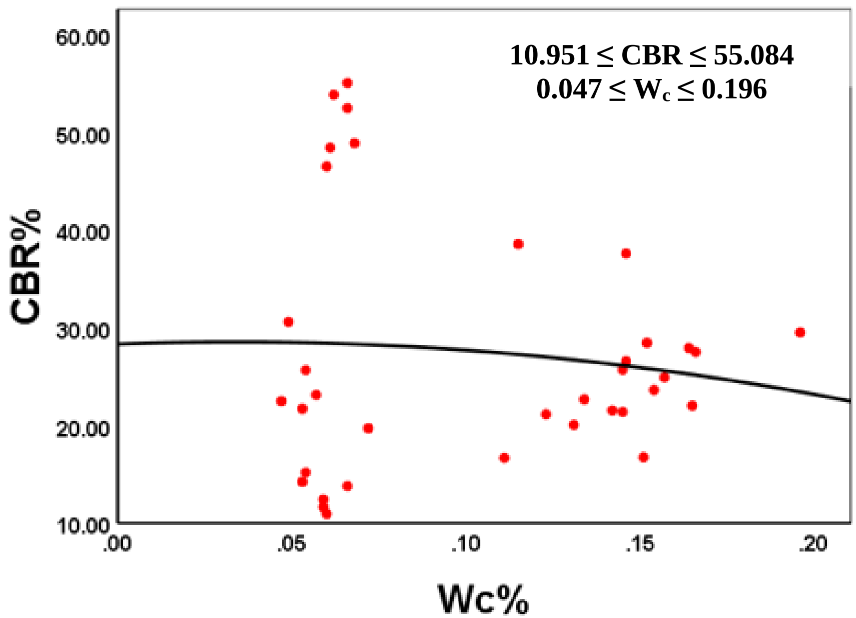
Table 5.12: Correlation between Variables CBR-basic physical properties-granular soils.

Correlation				
		CBR	***Wc***	***γdf***
CBR	***Pearson Correlation***	1	-.109	.751**
	Sig. (2-tailed)	/	.527	.000
	N	36	36	36
W_c	***Pearson Correlation***	-.109	1	-.416*
	Sig. (2-tailed)	.527	/	.012
	N	36	36	36
γ_{df}	***Pearson Correlation***	.751**	-.416*	1
	Sig. (2-tailed)	.000	.012	/
	N	36	36	36
**. Correlation is significant at the 0.01 level (2-tailed).				
*. Correlation is significant at the 0.05 level (2-tailed).				

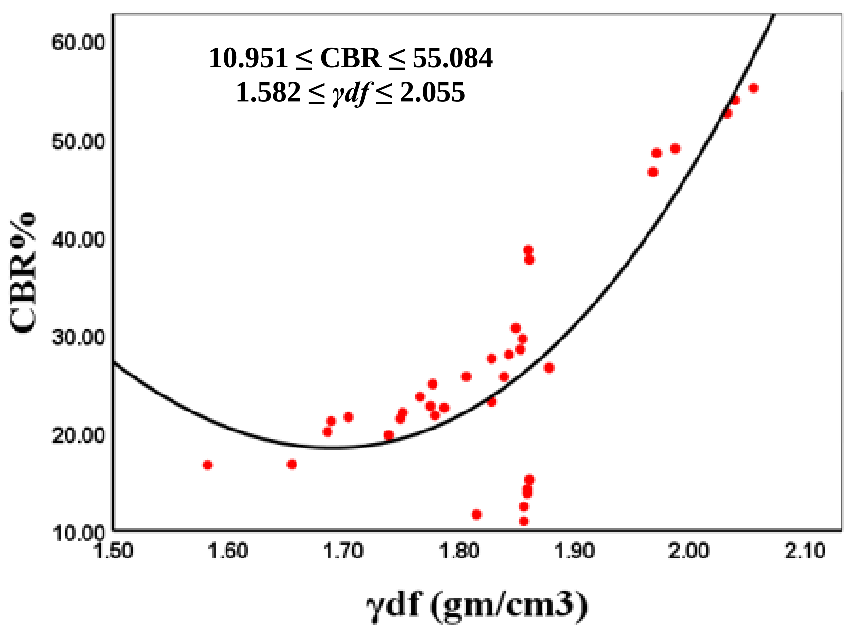
Three nonlinear regression were explained in Table (5.13) and Figure (5.7) and illustrates model expression, coefficient of determination (R^2), R-adjusted, and standard of error for each parameter separately.

Table 5.13: Summary of models and coefficients for Nonlinear CBR- physical properties for granular soils.

Ind. variable	***D. variable***	***Models expression***	***R^2***	***R adjusted***	***Std. Error***	***Estimated parameters***
CBR	**W_c**	$CBR = b_0 + (b_1 * W_c) + (b_2 * W_c^{2})$	0.19	0.178	2.798	$b_0 = 22.059$ $b_1 = -998.940$ $b_2 = 167.35$
	γ_{df}	$CBR = b_0 + (b_1 * \gamma_{df}) + (b_2 * \gamma_{df}^{2}) + (b_3 * \gamma_{df}^{3})$	0.70	0.675	6.498	$b_0 = 203.569$ $b_1 = 0$ $b_2 = -199.475$ $b_3 = 79.989$



a) Quadratic relation between CBR-Wc



b) Cubic relation between CBR-γdf

Figure 5.7: Models' expression for Nonlinear CBR-physical properties-granular soils parameters.

Table (5.14) shows the parameter of the developed model and its limitation with confidence interval.

Table 5.14: Nonlinear model of CBR-physical properties-granular soils parameters.

Developed model	$CBR = b_0 + (b_1 * W_c) + (b_2 * \gamma_{df}{}^2) + (b_3 * \gamma_{df})$			
Parameter Estimates				
Parameter	***Estimate***	***Std. Error***	***95% Confidence Interval***	
			Lower Bound	***Upper Bound***
b_0	767.523	211.132	333.535	1201.510
b_1	56.757	25.438	4.469	109.046
b_2	274.774	63.136	144.996	404.552
b_3	-911.870	231.298	-1387.309	-436.431
Solution	$CBR\ Predicted = 768+57W_c+275\ \gamma_{df}{}^2-912\ \gamma_{df}$			

Table (5.15) discloses that the sum of regression is higher than the sum of residue which is sustained the significant of the model. While, from the same table, the high value of the R-Square (0.801) indicates a good prediction.

Table 5.15: ANOVA for nonlinear (CBR- physical properties-granular soils parameters) model.

ANOVA[a]			
Source	***Sum of Squares***	***df***	***Mean Squares***
Regression	27460.656	4	6865.164
Residual	851.069	26	32.733
Uncorrected Total	28311.725	30	
Corrected Total	4266.909	29	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = 0.801			

A conclusion can draw that the developed model for CBR- physical properties parameters for granular soil is acceptable. Figure (5.8) indicates that acceptable scatter can be recognized between predicted and observed operability values with R^2 (0.735), furthermore, almost all value except one value out the significant level boundaries. Figure (5.9) shows the scatter plot for residual and independent variable γ_{df}.

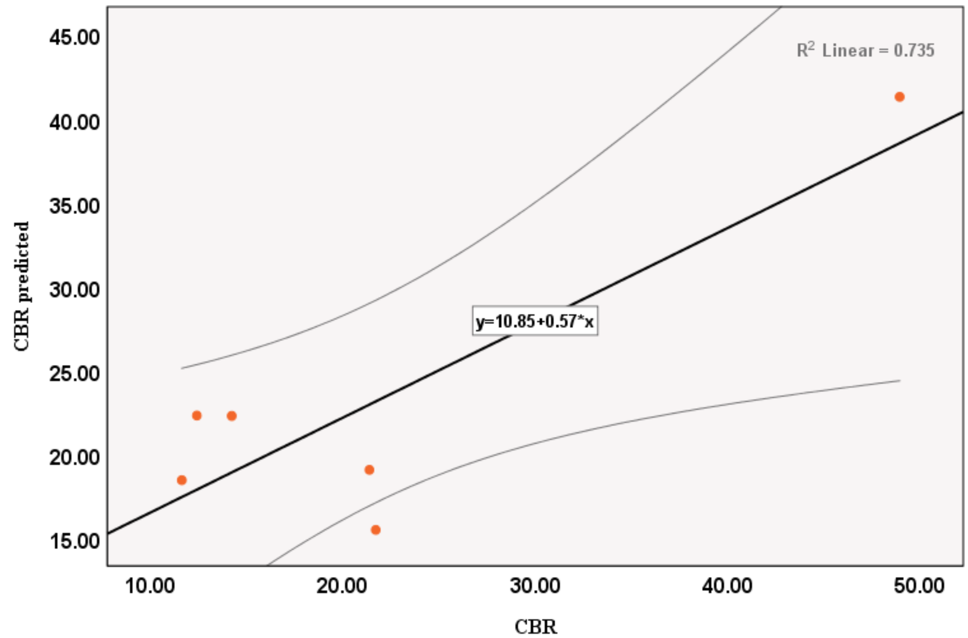


Figure 5.8: Comparisons between Measured and Predicted CBR for granular soils

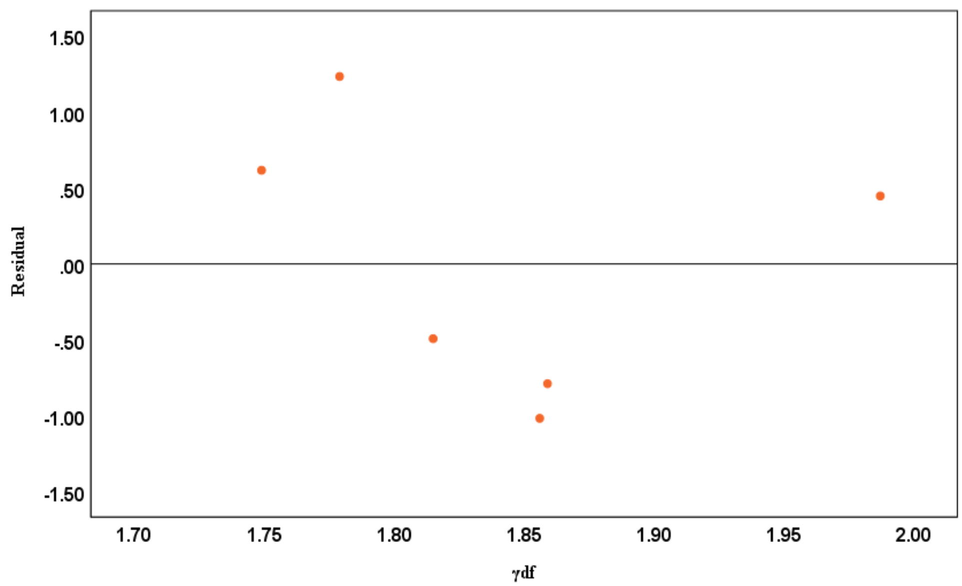


Figure 5.9: Scatter plot for residual and predicted CBR – granular soils

5.3.2.2 **Correlation CBR – Basic Physical Properties For Clayey Soils:**

Correlating bearing ratio for clayey soils to moisture content (W_c), and field density (γ_{df}) was conducted. The analysis results of correlation are shown in Tables (5.16) to (5.19). The bivariate Pearson correlation between variables was explained in Table (5.16) and shows that the (γ_{df}) has the most significant correlation to CBR value then the (W_c).

Table 5.16: Correlation between CBR and basic physical properties of clayey soils.

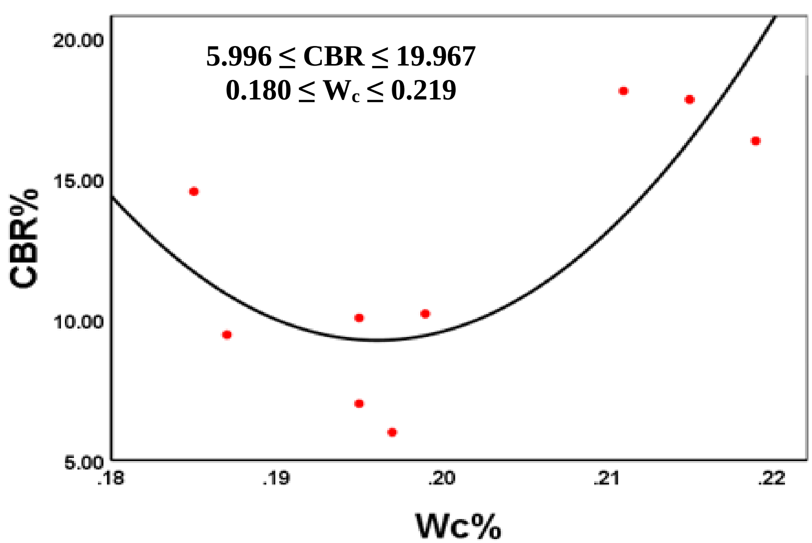
Correlations				
		W_c	γ_{df}	CBR
W_c	Pearson Correlation	1	.435	.507
	Sig. (2-tailed)	/	.157	.093
	N	12	12	12
γdf	Pearson Correlation	.435	1	.930**
	Sig. (2-tailed)	.157	/	.000
	N	12	12	12
CBR	Pearson Correlation	.507	.930**	1
	Sig. (2-tailed)	.093	.000	/
	N	12	12	12

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

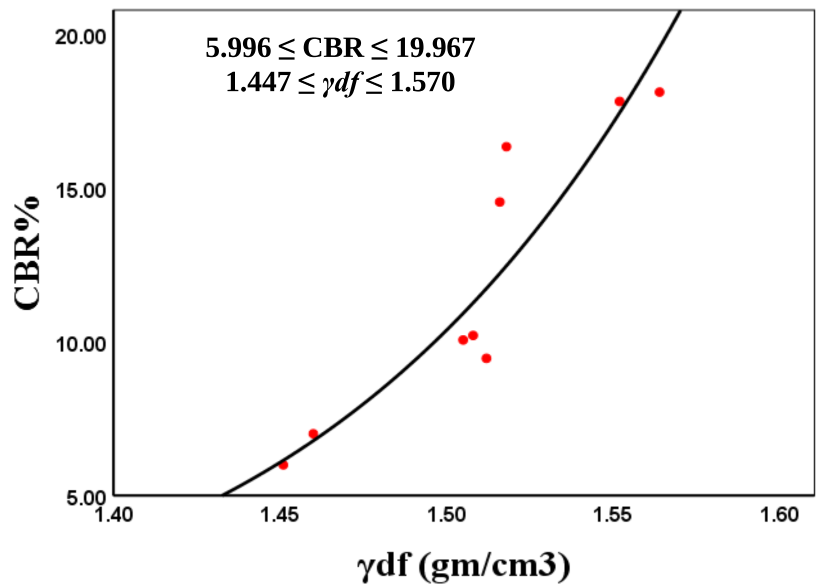
Three nonlinear regressions were explained in Table (5.17) and Figure (5.10) and illustrates model expression, R-square, R-adjusted, and standard of error for each parameter separately.

Table 5.17: Summary of models and coefficients for Nonlinear CBR-basic physical properties for clayey soils.

Ind. variable	D. variable	Models expression	R^2	R-adjusted	Std. Error	Estimated parameters
CBR	W_c	CBR = $b_0 + (b_1 * W_c) + (b_2 * W_c^{\ 2})$	0.647	0.529	3.171	$b_0 = 771.023$ $b_1 = -7770.013$ $b_2 = 19813.67$
	γ_{df}	CBR = $e^{(b_0 + (b_1/\gamma_{df})}$	0.867	0.846	0.158	$b_0 = 17.854$ $b_1 = -23.271$



a) *Quadratic relation between CBR-Wc*



b) S-relation between CBR-γdf

Figure 5.10: Models' expression for Nonlinear CBR-physical properties-clayey soils parameters.

The parameters of the developed model and its limitation with a confidence Interval of 95% are shown in Table (5.18).

Table 5.18: Nonlinear CBR-physical properties-clayey soils parameters modeling.

Developed model	$CBR = b_0 + b_1 * W_c^2 + b_2 * \gamma_{df}^2 + b_3 * \gamma_{df} + b_4 * W_c + b_5 * W_c * \gamma_{df}$			
Parameter Estimates				
Parameter	***Estimate***	***Std. Error***	***95% Confidence Interval***	
			Lower Bound	***Upper Bound***
b_0	1093.977	1328.394	-3133.566	5321.520
b_1	12175.854	6457.954	-8376.239	32727.948
b_2	697.661	1109.237	-2832.425	4227.747
b_3	-1457.857	2019.526	-7884.891	4969.177
b_4	-635.058	12973.836	-41923.595	40653.480
b_5	-2758.476	8161.748	-28732.800	23215.847
Solution	$CBR\ Predicted = 1094 + 12176\ W_c^2 + 698 * \gamma_{df}^2 - 1458 * \gamma_{df} - 635 * W_c - 2758 * W_c * \gamma_{df}$			

ANOVA illustrates in Table (5.19) and discloses that the sum of regression is higher than the sum of residue which is sustained the significant of the model. While, from the same table, the high value of the R-Square (0.949) indicates a perfect prediction.

Table 5.19: ANOVA for nonlinear (CBR- physical properties-clayey soils parameters) modeling.

ANOVA[a]			
Source	***Sum of Squares***	***df***	***Mean Squares***
Regression	1496.159	6	249.360
Residual	8.639	3	2.880
Uncorrected Total	1504.799	9	
Corrected Total	170.789	8	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = 0.949			

A conclusion can draw that the developed model for CBR-basic physical properties parameters for clayey soil is acceptable. Figure (5.11) indicates that acceptable scatter can be recognized between predicted and observed operability values with R^2 is (0.95), furthermore, almost all value within the significant level boundaries. Figure (5.11) shows the scatter plot for residual and independent variable γd_f.

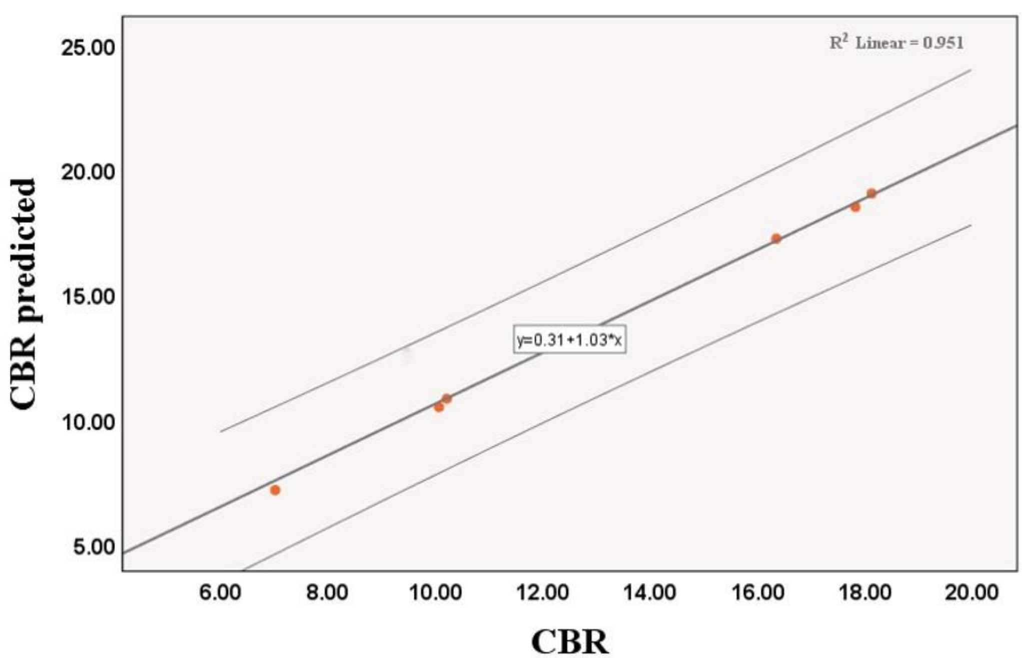


Figure 5.11: Comparisons between Measured and Predicted CBR for clayey soils

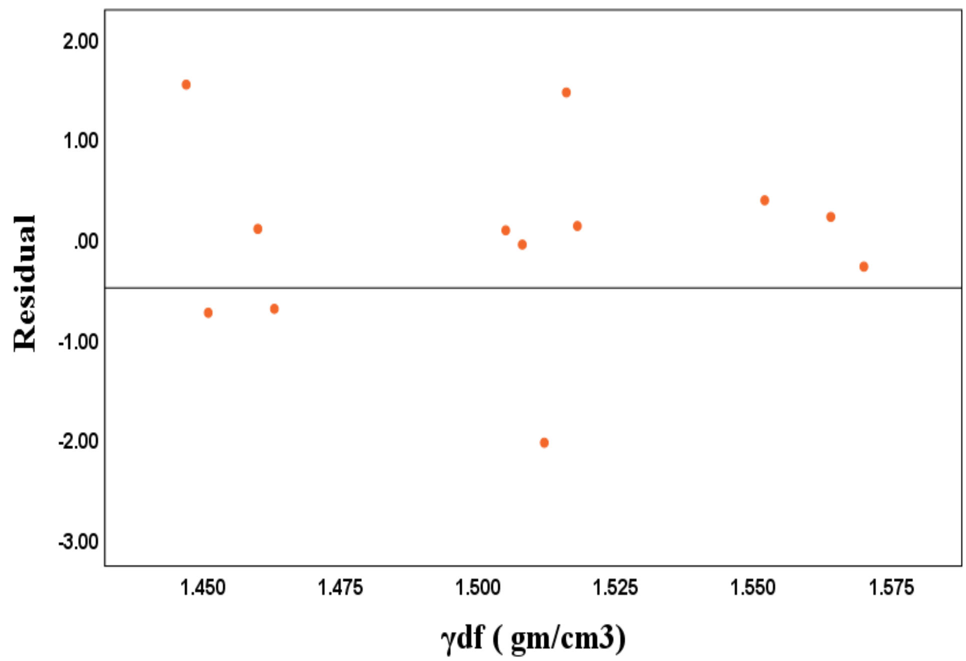


Figure 5.12: Scatter plot for residual and predicted CBR –clayey soils

5.3.3 **Building of CBR-LWD-Physical Properties Models**

As mentioned previously, (S_d, E_d, W_c and γ_{df}) were selected to build two sets of modeling for unbounded granular materials and clayey soils. This section was represented the last two sets of modeling for both types of soils (sandy and clayey soils). These models were demonstrated the correlation of CBR value to LWD parameters and basic physical properties. The results were divided randomly 70% to generate the model and 30% to validate the model. The analysis results for both soils models were explained as follow:

5.3.3.1 **Correlation CBR – LWD - Physical Properties For Granular Soils**

CBR and (S_d, E_d, W_c and γ_{df}) was employed to build a model for predicting CBR value. The analysis results of modeling are shown in Tables (5.20) to (5.23). Table (5.23) explains the bivariate Pearson correlation between variables and demonstrates that the E_d has the most significant correlation to, S_d, γ_{df} and lastly W_c.

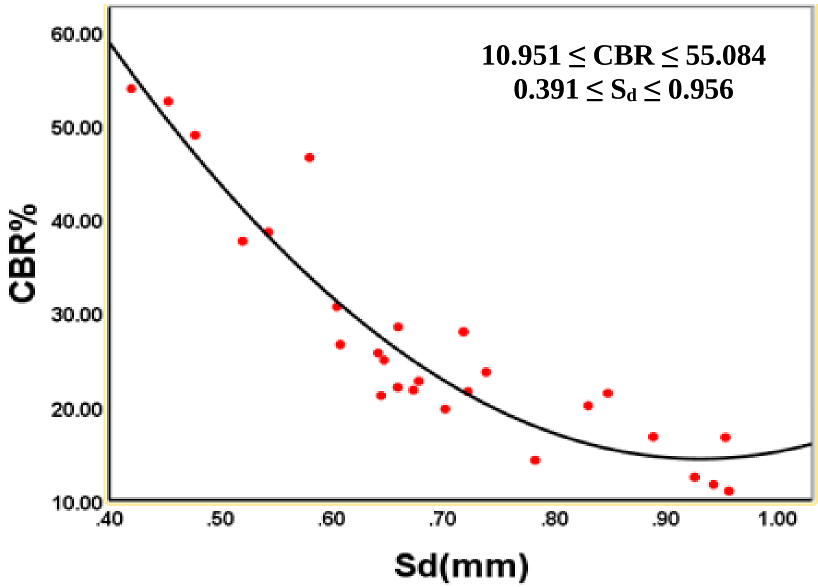
Table 5.20: Correlation between CBR, LWD parameter and basic physical properties-granular soils.

Correlations						
		CBR	***S_d***	***E_d***	***W_c***	***γ_{df}***
CBR	***Pearson Correlation***	1	-.888**	.938**	-.109	.751**
	Sig. (2-tailed)		.000	.000	.527	.000
	N	36	36	36	36	36
S_d	***Pearson Correlation***	-.888**	1	-.903**	.078	-.662**
	Sig. (2-tailed)	.000		.000	.650	.000
	N	36	62	62	36	36
E_d	***Pearson Correlation***	.938**	-.903**	1	-.168	.746**
	Sig. (2-tailed)	.000	.000		.329	.000
	N	36	62	62	36	36
W_c	***Pearson Correlation***	-.109	.078	-.168	1	-.416*
	Sig. (2-tailed)	.527	.650	.329		.012
	N	36	36	36	36	36
γ_{df}	***Pearson Correlation***	.751**	-.662**	.746**	-.416*	1
	Sig. (2-tailed)	.000	.000	.000	.012	
	N	36	36	36	36	36

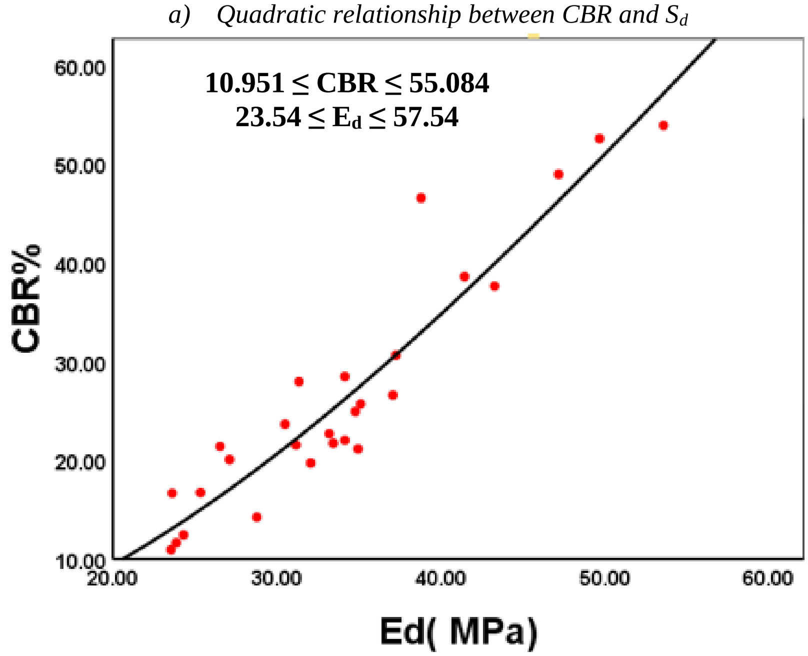
Three nonlinear regressions were explained in Table (5.21) and illustrates R^2, R-adjusted, and standard of error for each parameter separately. Model expression was shown in Figure (5.13).

Table 5.21: Summary of models and coefficients for CBR, LWD parameter and physical properties-granular soils.

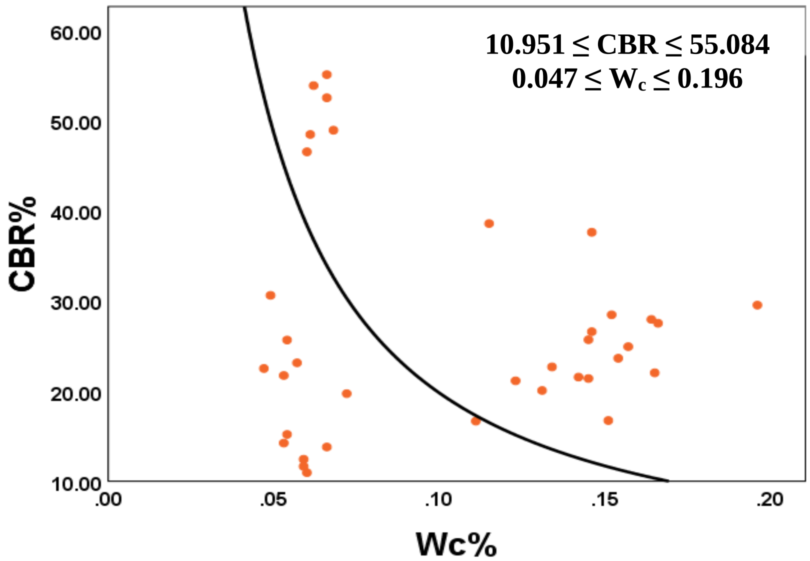
Ind. variable	***D. variable***	***Models expression***	***R^2***	***R adjusted***	***Std. Error***	***Estimated parameters***
CBR	***S_d***	$CBR = b_0 + (b_1 * S_d) + (b_2 * S_d{}^2)$	0.885	0.875	4.308	b_0 = 150.986 b_1 = -293.541 b_2 = 157.708
	E_d	$CBR = b_3 + (b_4 * E_d) + (b_5 * E_d{}^2) + (b_6 * E_d{}^3)$	0.884	0.875	4.314	b_0 = -0.57 b_1 = 0.028 b_2 = 0 b_3 = 0
	W_c	$CBR = b_3 + (b_4 * Wc) + (b_5 * Wc^2) + (b_6 * Wc^3)$	0.10	0.09	12.141	b_0 = -21.659 b_1 = 1690.781 b_2 = -15927.734 b_3 = 44546.875
	γd_f	$CBR = b_0 + (b_1 * \gamma_{df}{}^2) + (b_2 * \gamma_{df}{}^3)$	0.623	0.591	7.789	b_0 = 203.569 b_1 = -199.475 b_2 = 79.989



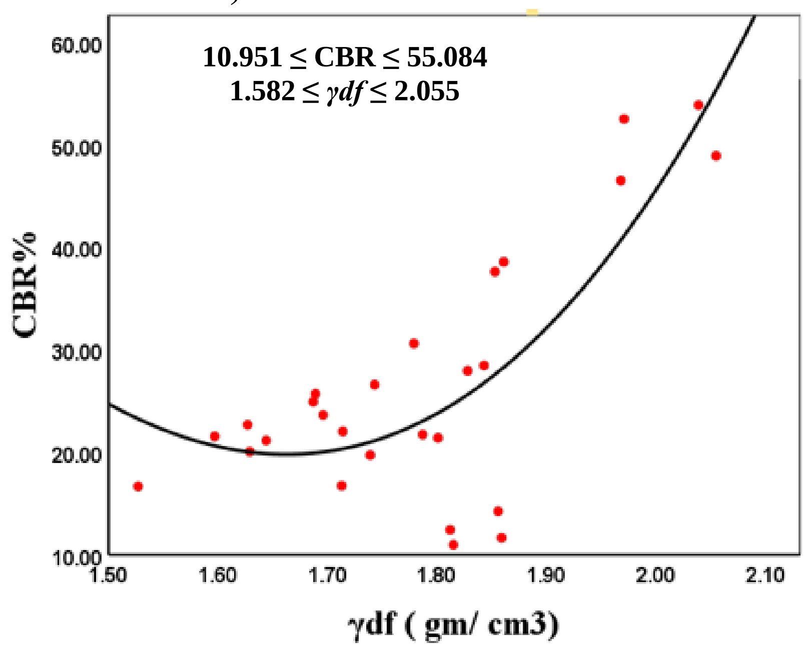
a) Quadratic relationship between CBR and S_d



b) Cubic relation between CBR and E_d



c) Cubic relation between CBR-Wc



d) Cubic relationship between CBR-γdf

Figure 5.13: Models' expression for Nonlinear CBR, LWD parameter and basic physical properties-granular soils.

The parameters of the developed model and its limitation with a confidence interval of 95% are shown in Table (5.22).

Table 5.22:Nonlinear CBR, LWD parameter and basic physical properties-granular soils.

Developed model	$CBR= b_0 + b_1*E_d + b_2 * S_d* \gamma_{df} + b_3 * W_c$			
Parameter Estimates				
Parameter	***Estimate***	***Std. Error***	***95% Confidence Interval***	
			Lower Bound	***Upper Bound***
b_0	1.743	.207	1.318	2.167
b_1	229.477	123.495	-24.371	483.325
b_2	-34.541	8.122	-51.237	-17.846
b_3	-143.481	82.555	-313.176	26.215
Solution	$CBR= 1.74 E_d + 229 S_d W_c - 35 – 143W_c$			

ANOVA Table discloses that the sum of regression is higher than the sum of residue which is sustained the significant of the model. While, from the same table, the high value of the R-Square (0.881) indicates a perfect prediction, see Table (5.23).

Table 5.23: ANOVA for nonlinear CBR, LWD parameter and basic physical properties-granular soils model.

ANOVA[a]			
Source	***Sum of Squares***	***df***	***Mean Squares***
Regression	27805.907	4	6951.477
Residual	505.817	26	19.455
Uncorrected Total	28311.725	30	
Corrected Total	4266.909	29	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = .881.			

A conclusion can draw that the developed model for CBR - LWD parameters - basic physical properties parameters for granular soil is acceptable. See Figure (5.14). The scatter plot for residual and independent variable E_d was shown in Figure (5.15)

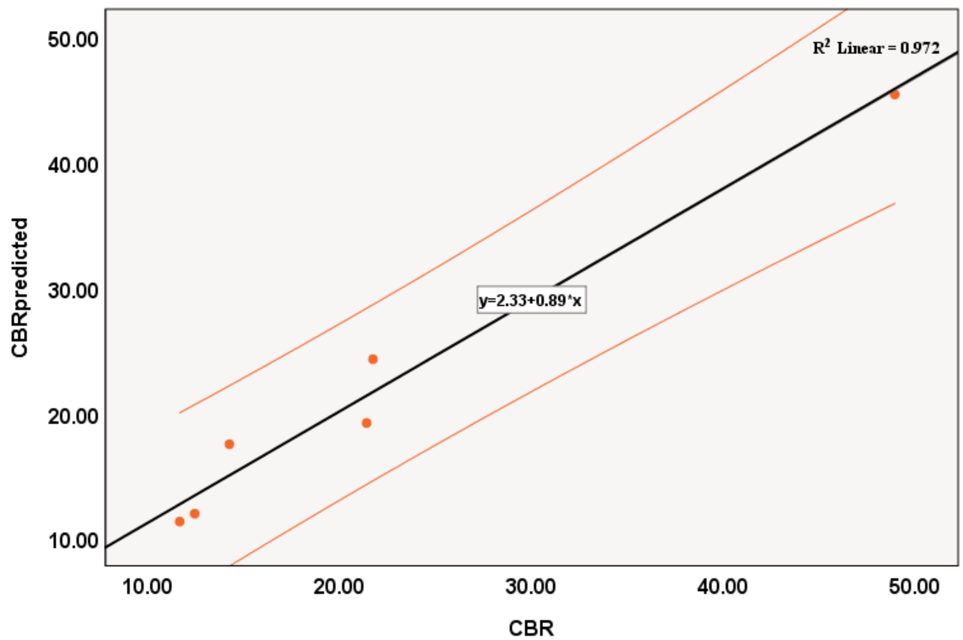


Figure 5.14: Comparisons between Measured and Predicted CBR for granular soils

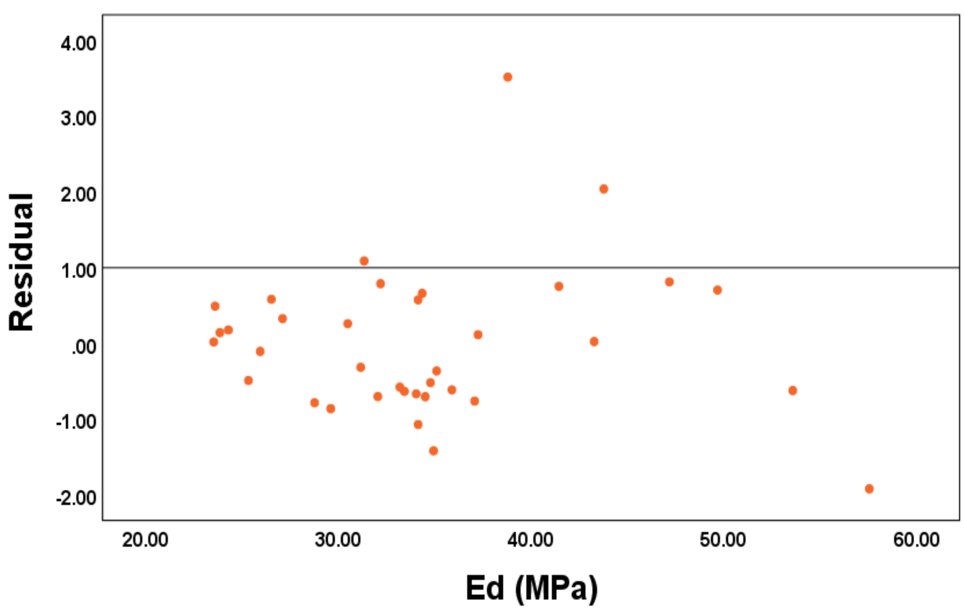


Figure 5.15: Scatter plot for residual and predicted CBR – granular soils

5.3.3.2 **Correlation CBR –LWD- Physical Properties For Clayey Soils**

Modeling bearing ratio for clayey soils to S_d, E_d, and γ_{df} was conducted. The analysis results of modeling are shown in Tables (5.24) to (5.27). The bivariate Pearson correlation between variables was explained in Table (5.24) and demonstrates that the E_d has the most significant correlation to CBR value then S_d, γ_{df} and W_c.

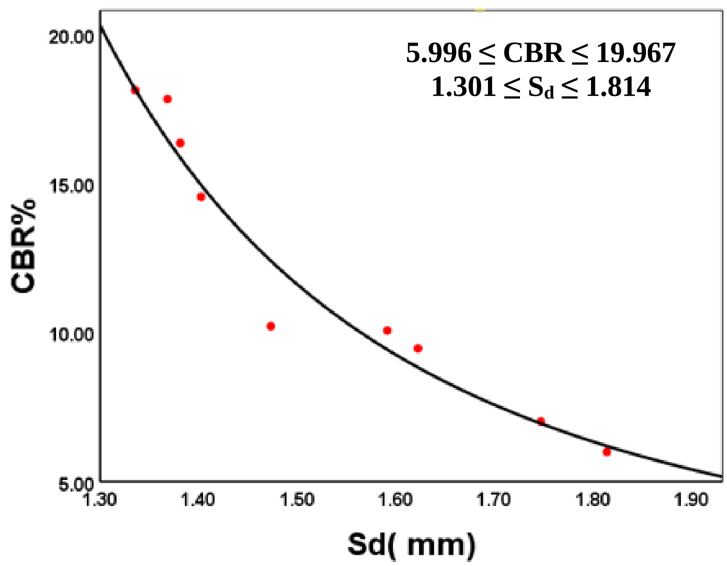
Table 5.24: Correlation between CBR, LWD parameter and basic physical properties- clayey soils.

Correlations						
		S_d	E_d	W_c	γ_{df}	CBR
S_d	Pearson Correlation	1	-.997**	-.497	-.931**	-.958**
	Sig. (2-tailed)		.000	.100	.000	.000
	N	12	12	12	12	12
E_d	Pearson Correlation	-.997**	1	.501	.934**	.972**
	Sig. (2-tailed)	.000		.097	.000	.000
	N	12	12	12	12	12
W_c	Pearson Correlation	-.497	.501	1	.435	.507
	Sig. (2-tailed)	.100	.097		.157	.093
	N	12	12	12	12	12
γ_{df}	Pearson Correlation	-.931**	.934**	.435	1	.930**
	Sig. (2-tailed)	.000	.000	.157		.000
	N	12	12	12	12	12
CBR	Pearson Correlation	-.958**	.972**	.507	.930**	1
	Sig. (2-tailed)	.000	.000	.093	.000	
	N	12	12	12	12	12

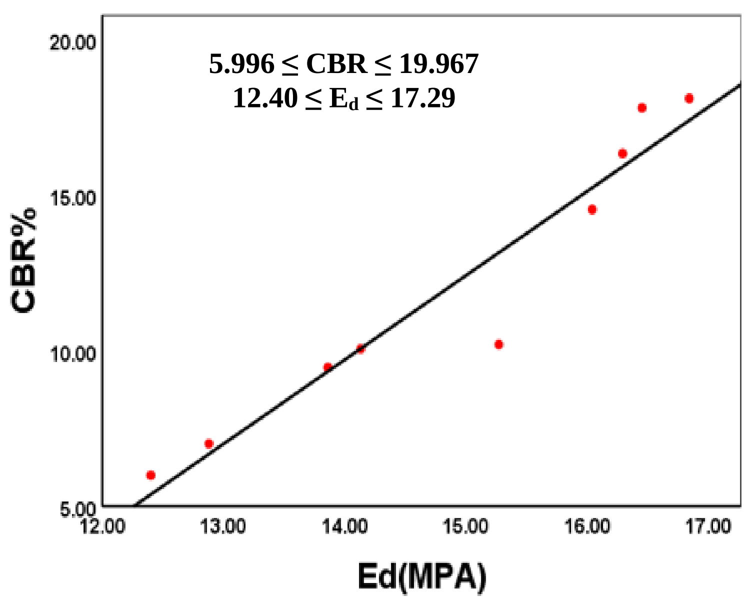
Three nonlinear regression was explained in Table (5.25) and illustrates R^2, R- adjusted, and standard of error for each parameter separately. While the model's expression was illustrated in Figure (5.11).

Table 5.25: Summary of models for Nonlinear CBR, LWD parameter and physical properties- clayey soils.

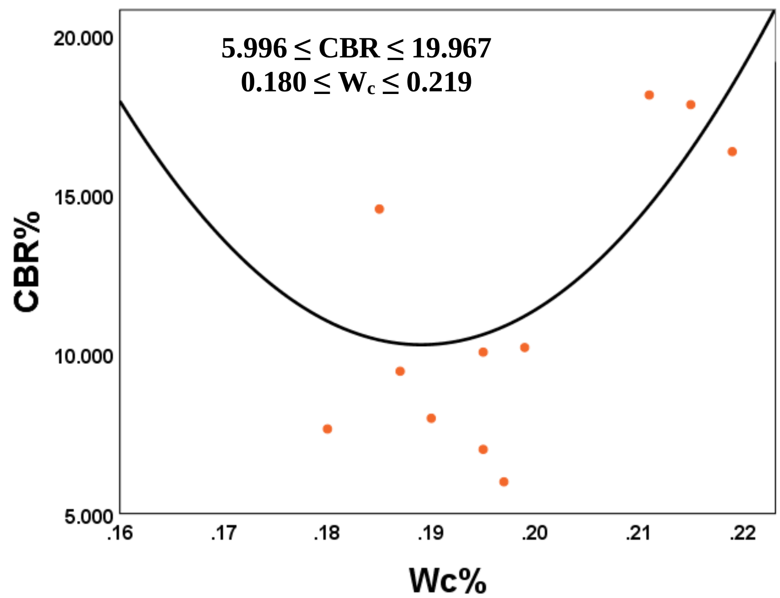
Ind. variable	D. variable	Models expression	R^2	R adjusted	Std. Error	Estimated parameters
CBR	S_d	CBR = Exp (b_0 + (b_1 /S_d))	0.957	0.951	0.089	b_0 = -1.176 b_1 = -293.541 b_2 = 157.708
	E_d	CBR = b_0 + (b_1 * E_d)	0.927	0.916	4.314	b_0 = -28.233 b_1 = 2.711
	W_c	CBR=b_0 + (b_1*W_c) + (b_2*W_c^2)	0.317	0.165	4.523	b_0 = 333.958 b_1= -3426.364 b_2= 9057.403
	γ_{df}	CBR = Exp (b_0+ (b_1 / γ_{df}))	0.623	0.617	7.789	b_0 = -1.176 b_1 = 5.442



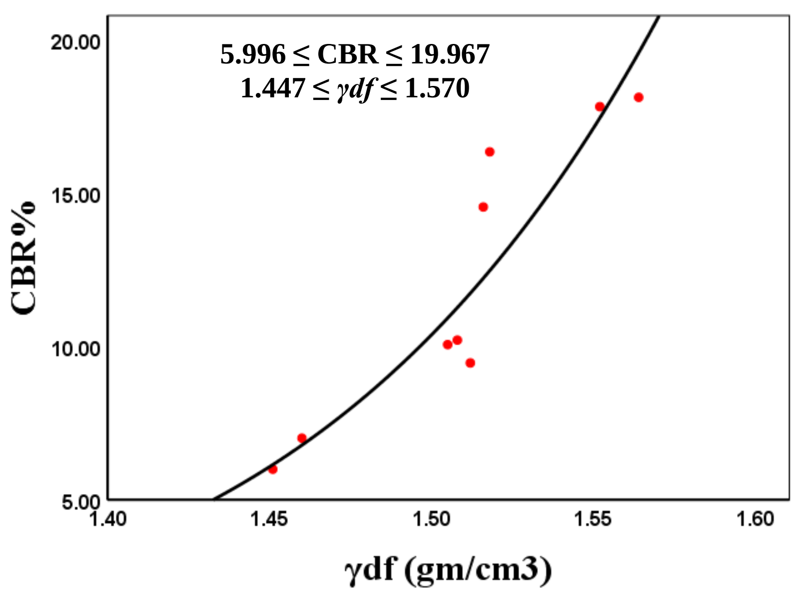
a) S-relation between CBR and Sd



b) Linear relation between CBR and Ed



a. Quadratic relation between CBR and Wc



c) S-relation between CBR-γ_{df}

Figure 5.16: Models' expression for Nonlinear CBR, LWD parameter and basic physical properties- clayey soils

Table (5.23) shows the parameter of the developed model and its limitation with a confidence interval of 95%.

Table 5.26:Nonlinear CBR, LWD parameter and basic physical properties- clayey soils model.

Developed model	$\mathrm{CBR} = \frac{(b_0 * Ed)}{Sd} + \mathrm{b}_1$			
Parameter Estimates				
Parameter	**Estimate**	**Std. Error**	**95% Confidence Interval**	
			Lower Bound	**Upper Bound**
b_0	2.091	.205	1.607	2.575
b_1	-8.698	2.085	-13.627	-3.769
Solution	$\mathrm{CBR} = \frac{(2.091 * Ed)}{Sd} - 8.698$			

ANOVA table was disclosed that the sum of regression is higher than the sum of residue which is sustained the significant of the model. While, from the same table, the high value of the R-Square (0.937) indicates a prediction, thus from this value, as shown in Table (5.24).

Table 5.27: ANOVA for nonlinear CBR, LWD parameter and basic physical properties- clayey soils) model.

ANOVA[a]			
Source	**Sum of Squares**	**df**	**Mean Squares**
Regression	1494.062	2	747.031
Residual	10.736	7	1.534
Uncorrected Total	1504.799	9	
Corrected Total	170.789	8	
Dependent variable: CBR			
a. R squared = 1 - (Residual Sum of Squares) / (Corrected Sum of Squares) = .937.			

A conclusion can draw that the developed model for CBR-LWD-basic physical properties parameters for granular soil is acceptable. See Figure (5.11) which indicates that acceptable scatter can recognize between predicted and observed values R^2(0.99), furthermore, almost all value within the significant level boundaries. The scatter plot for residual and independent variable E_d was shown in Figure (5.18)

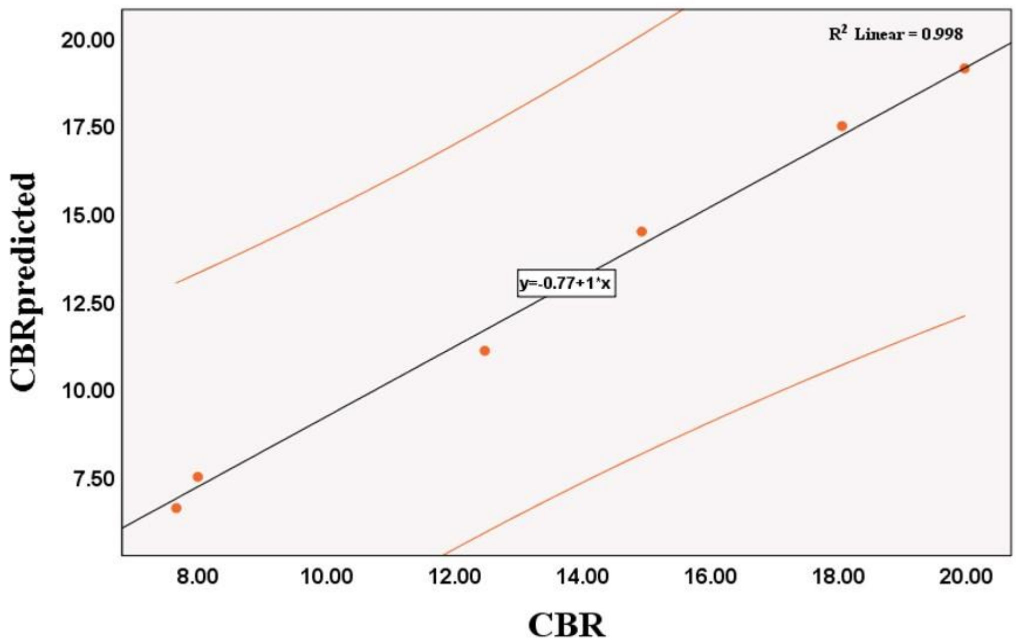


Figure 5.17: Comparisons between Measured and Predicted CBR -clayey soils.

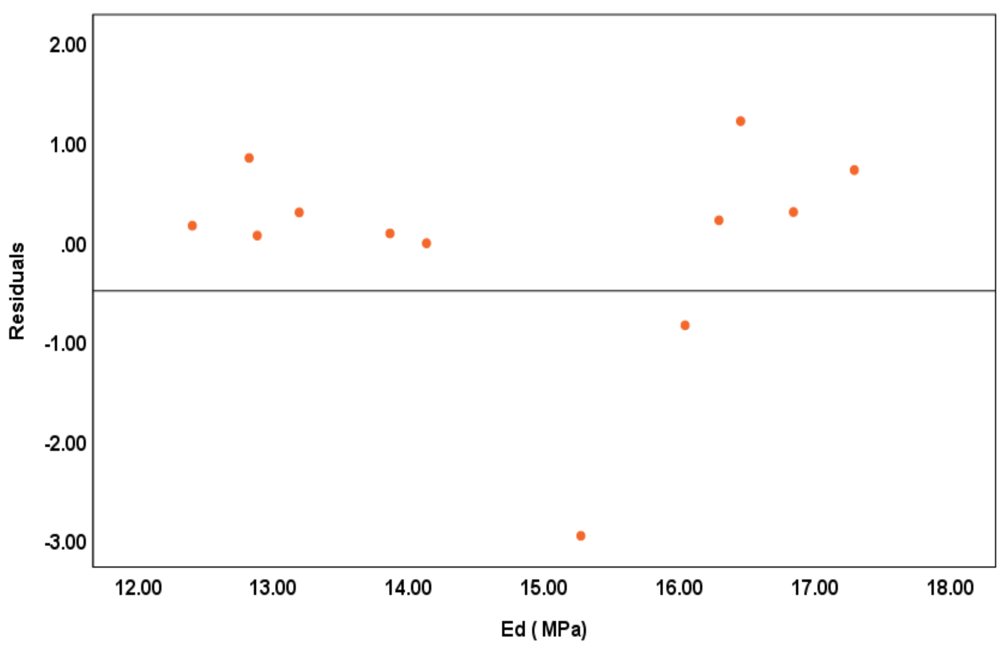


Figure 5.18: Scatter plot for residual and predicted CBR – clayey soils

5.4 Summary

Statistical analysis modeling is a very important approach which helps to understand the significant of the different parameters such as the LWD parameters and physical properties to characterizing the California bearing ratio. However, this chapter proved the ability to start up an acceptable model to predict bearing ratio for two main groups of soil, the sandy soil and clayey soil. Nevertheless, it is two edges weapon, whereas the limiting input could lead to misleading result. Therefore, the statistical model is significant within its boundary conditions.

Chapter Six

VARIFICATION OF STATISTICAL ANALYSIS

6.1 Introduction.

This chapter included conducting field tests for two highway sections in different projects within Karbala city. The field works include a series of laboratory and in-situ tests to validate predicted models which were built by statistical analysis program, to achieve this purpose that required evaluating of selected local subgrade soils and identifying their engineering and physical properties.

6.2 Site Characterizations

Subgrade soils evaluated in this work were collected from two roadway projects in Kerbala city. The 1st site project was AL-Emam Ali district, and 2nd was AL-Takahe district. Two main groups of testing-techniques were performed in each site: laboratory tests and field tests. In each field investigation, different number of stations were selected for each 100 m with ranged (6-20) station depending on constructed length of highways.

The dry unit weight and moisture content were obtained using the field density by core cutter method. LWD test was performed at the same location of the field density test point to evaluate the reliability of this device. After preparing laboratory specimens, physical and chemical tests were implemented including: sieve analysis, modified proctor test, Atterberg limits, California bearing ratio test (CBR), and chemical tests such as total soluble salt, sulfur trioxide, and gypsum in term of ($CaSO_4$ $2H_2O$). The following subsections provide a description of the location and geology of each testing site which was investigated in this research.

6.2.1 AL-Takahe District

AL-Takahe district is located at west of Karbala city includes an arterial two-lane roadway with length 2000 m and width 40 m. A left side of this an arterial roadway was constructed in 2019. This project was selected to conduct proposed field measurements. Permission to perform the field tests which proposed in this research was granted by the Government of Karbala City. Field density by CCM and LWD were performed every 100 m on the center of left side for constructed roadway, which start from station 00+00 and end at station 20+00. Twenty test points were performed for both the CCM and LWD test, the LWD test was conducted in the same location of the field density test, as clarified in Figure (6.1). After completing field tests, the disturbed soil samples have been collected from the field and tested in a laboratory for identifying their basic properties, see Table (6.1) and Figures (6.2) and (6.3).

Table 6.1: Average Physical Characteristics of Subgrade Soil - AL-Takahe zone

Property	***AL-Takahe district***	***AL-Emam Ali district***	***Specification***
Physical characteristics			
Site coordination	408595.011, 3605065.385	404229.144, 3611462.039	/
USCS classification	poorly graded sand with silt (SP-SM)	poorly graded sand (SP)	ASTM D2487
AASHTO classification	A-1-b	A-3	AASHTO M145
Dry Unit Weight	21.28 kN/m^3	20.89 kN/m^3	ASTM D1557
O.M.C	8 %	10.4 %	ASTM D4643
D_{10}, D_{30}, D_{60}	0.14,0.29, 0.53	0.17, 0.29, 0.402	ASTM D2487
C_u, C_c	3.79, 1.13	2.36, 1.23	
Gravel Fraction, GF	4.60 %	1.52 %	
Fine Content	5.30 %	4.01 %	ASTM D2487
CBR – unsoaked, soaked	57 %, 47%	38 %, 26%	ASTM D1883
Liquid limit	0 %	0 %	ASTM D4318
Plasticity Index	N. P	N. P	
Chemical characteristics			
SO_3	1.27 %	1.04 %	BS1377-3:1990
$CaSO_4\ 2H_2O$	3.73 %	1.77 %	
Total Soluble Salts	1.22 %	3.80 %	

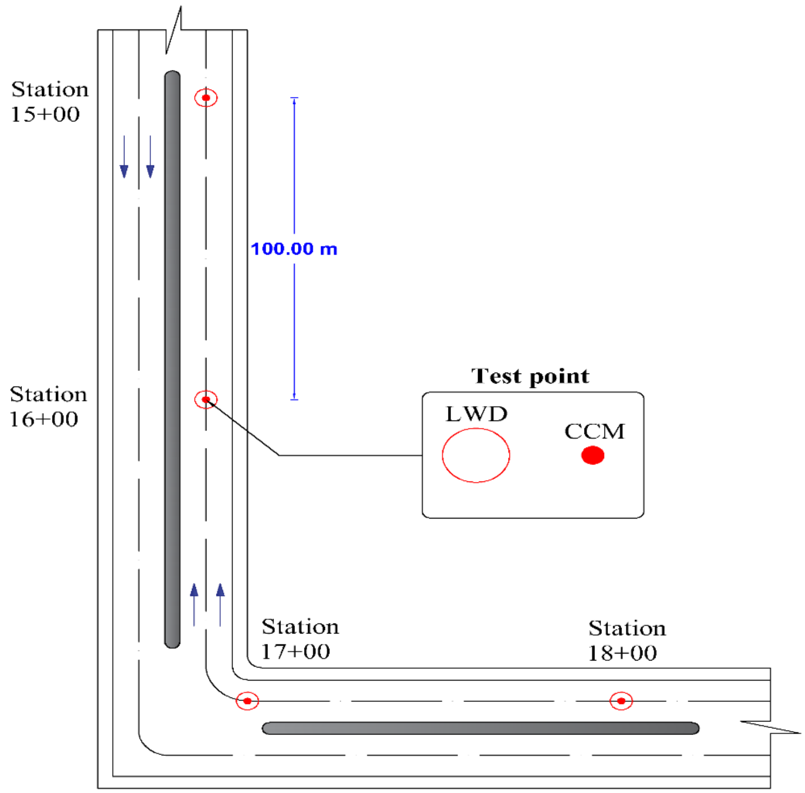


Figure 6.1: Schematic diagram of field test points - AL-Takahe district.

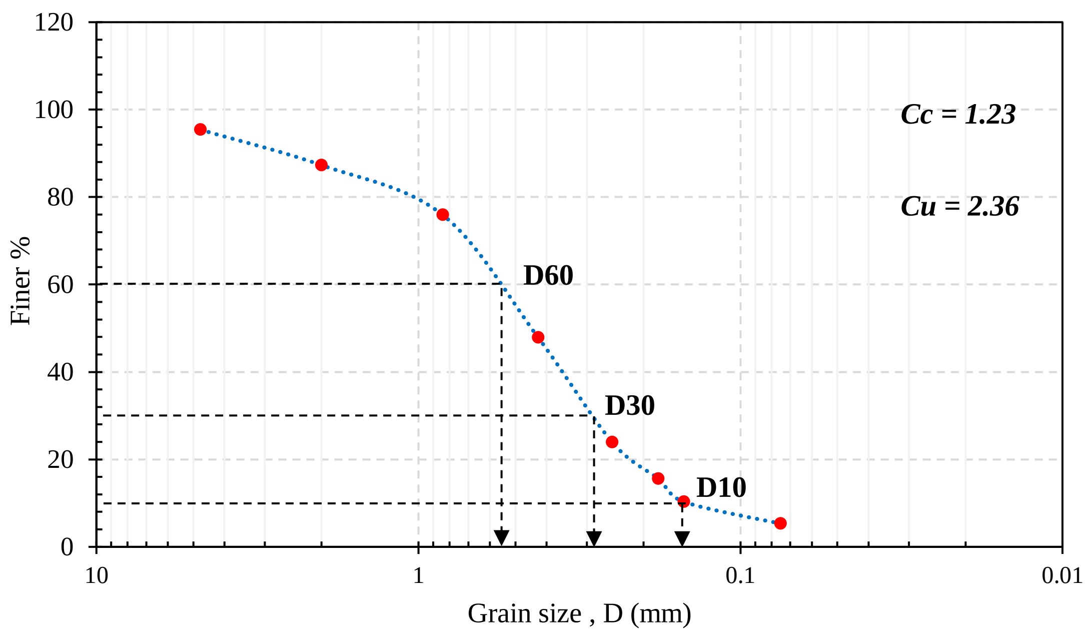


Figure 6.2: Grain size distribution of subgrade soil - AL-Takahe district.

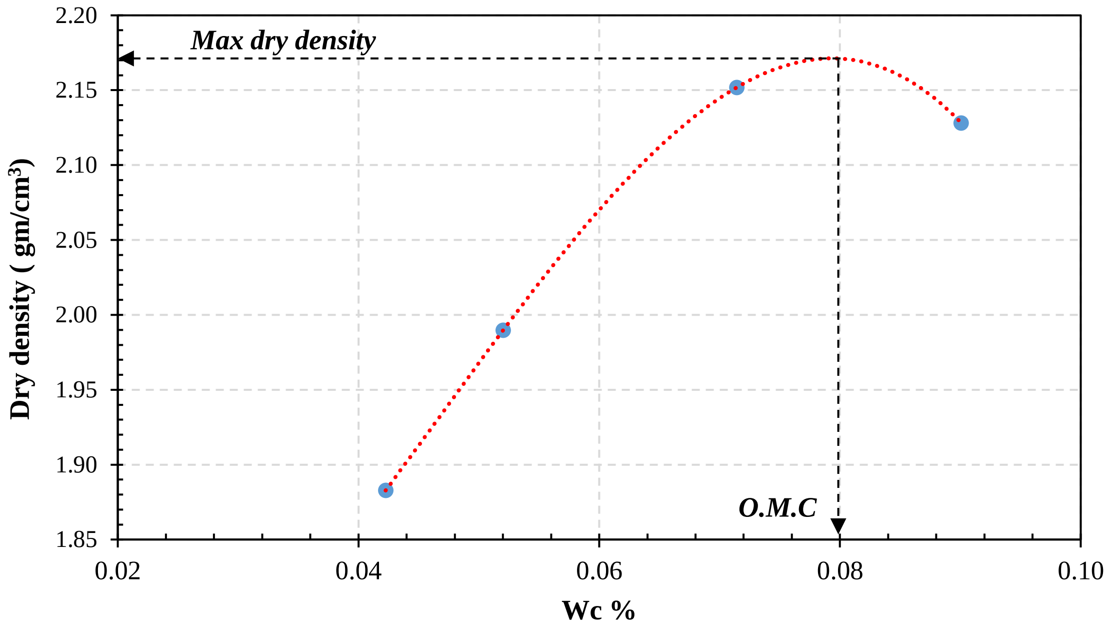


Figure 6.3: Modified proctor test of subgrade soil ‑ AL-Takahe district.

6.2.2 AL-Emam Ali district

AL-Emam Ali district is located at southwestern of Karbala city, consist of four local streets with length 150 m and width 12 m and the collector street with length 200 m and width 12 m. In February 2019, A construction project was conducted in AL-Emam Ali district which include paved roads. CCM and LWD test methods were conducted at the middle location of each road, which start from the first street at station 00+100 and end at station 00+800 as shown in Figure (6.4). The basic physical and chemical characteristics of the subgrade layer were clarified in Table (6.1) and Figures (6.5) and (6.6)

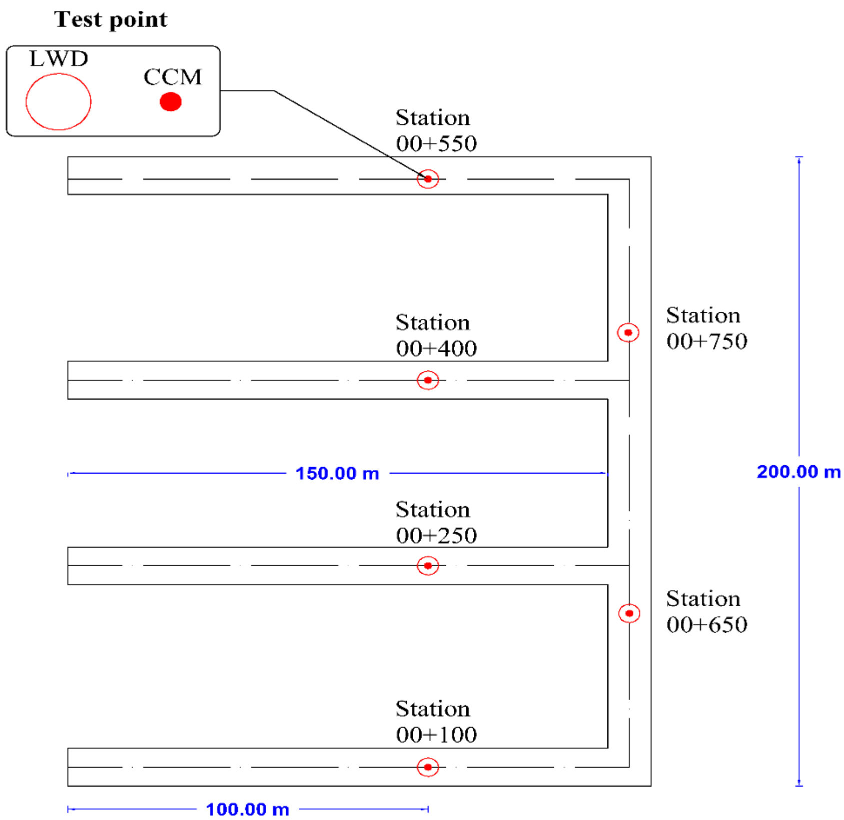


Figure 6.4: Schematic diagram of field test points - AL-Emam Ali district.

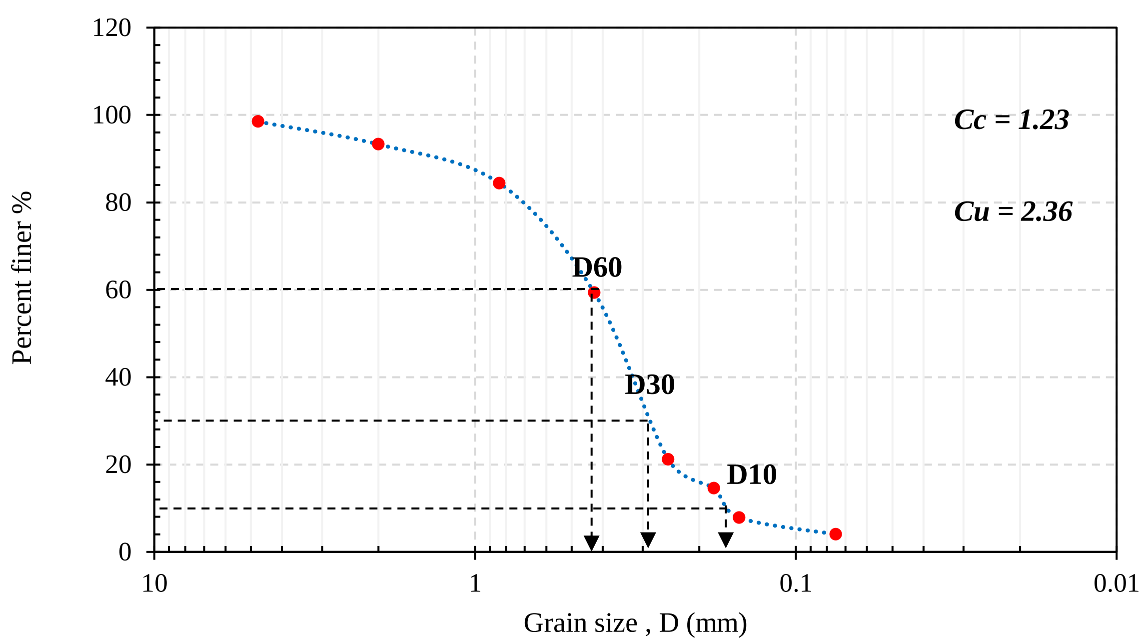


Figure 6.5: Grain size distribution of subgrade soil -AL-Emam Ali district.

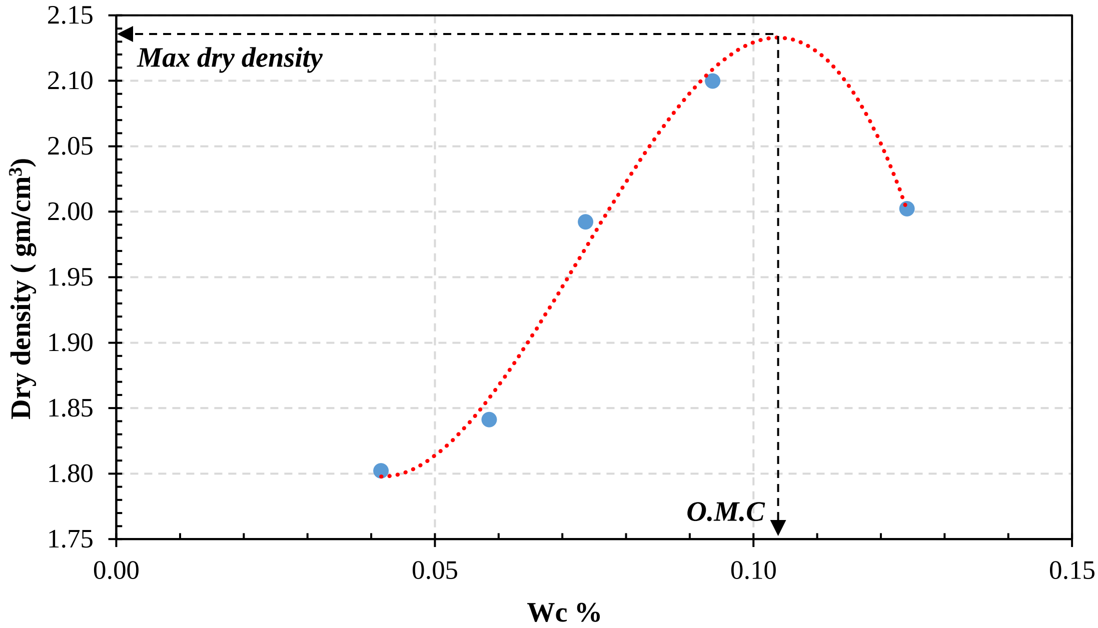


Figure 6.6: Modified proctor test of subgrade soil -AL-Emam Ali district.

6.3 Field Density Test Results

Dry unit weight (gm/cm^3), wet unit weight (gm/cm^3), moisture content (%), and degree of compaction (%) were obtained for compacted natural subgrade layers by field density test according to CCM at two field sites, 1st site was AL-Takahe zone, and 2nd site was AL-Emam Ali district. The test was conducted according to standard specification requirements for 26 test points as shall be presented in the next subsections:

6.3.1 Al-Takahe District

The results of twenty field density points were obtained by CCM are listed in Table (6.2), the data collected from field density tests indicate that wet unit weight was at range (1.829-2.01) gm/cm^3 with average 1.919 gm/cm3, moisture content was varied from 0.5 % to 8.91% with average 4.912%, dry unit weight was varied from 1.755 gm/ cm3 to 1.927 gm/ cm3, with average 1.830 gm/cm3, and lastly the degree of compaction was at range (80.88%-88.80%) with average 84.20%.

Table 6.2: Summary of field density Results for Subgrade Soils of AL-Takahe district.

Station No.	***γ_w (gm/cm^3)***	***W_c %***	***γ_d (gm/cm^3)***	***D_{oc} %***
00+100	1.952	8.91	1.792	82.58
00+200	1.990	6.76	1.864	85.90
00+300	2.010	8.00	1.861	85.76
00+400	1.883	6.90	1.761	81.15
00+500	1.988	6.40	1.868	86.08
00+600	1.951	5.80	1.844	84.98
00+700	1.910	7.00	1.785	82.26
00+800	1.927	4.80	1.839	84.75
00+900	1.837	4.70	1.755	80.88
01+00	1.844	3.50	1.782	82.12
01+100	2.010	8.00	1.861	85.76
01+200	1.951	8.80	1.793	82.63
01+300	1.854	2.40	1.811	83.46
01+400	1.977	2.60	1.927	88.80
01+500	1.854	0.50	1.845	85.02
01+600	1.854	1.50	1.827	84.19
01+700	1.829	0.90	1.813	83.55
01+800	1.951	3.50	1.885	86.87
01+900	1.922	5.60	1.820	83.87
02+00	1.893	1.67	1.862	85.81
Average	**1.919**	**4.912**	**1.830**	**84.32**

6.3.2 **AL-Emam Ali District**

The results of six field density points were obtained by CCM are listed in Table (6.3), the data collected from field density tests indicate that wet unit weight was at range (2.141-2.198) gm/cm^3 with average 2.177 gm/cm^3, moisture content was varied from 6.76 % to 8.91 % with average 7.895 %, dry unit weight was varied from 2.005 gm/ cm^3 to 2.041 gm/cm^3, with average 2.018 gm/cm^3, and lastly the degree of compaction was at range (95.0 %-96.7 %) with average 95.62 %.

Table 6.3: Summary of field density Results for Subgrade Soils of AL-Emam Ali district.

Station No.	γ_w ***(gm/cm³)***	W_c ***%***	γ_d ***(gm/cm³)***	D_{oc} ***%***
00+100	2.198	8.91	2.018	95.60
00+250	2.141	6.76	2.005	95.00
00+400	2.175	8.00	2.014	95.50
00+550	2.182	6.90	2.041	96.70
00+650	2.178	8.00	2.017	95.60
00+750	2.189	8.80	2.012	95.30
Average	**2.177**	**7.895**	**2.018**	**95.62**

6.4 LWD Test Results

The dynamic properties of subgrade layers were obtained by implementing the LWD test at two field sites as mentioned previously. The LWD parameters measured during this research are surface deflection (mm), dynamic modulus (MPa), and degree of compatibility (ms).

6.4.1 AL-Takahe District.

The results of the twenty LWD tests performed on a well compacted subgrade surface were calculated by averaging the values resulted from three consecutive drops. The data extracted from the integration process indicate that vertical displacements varied from 0.162 mm to 0.575 mm, with an average deflection of 0.301 mm. The values of dynamic modulus ranged from 39.13 MPa to 138.89 MPa with an average 86.25 MPa. The average value of degree of compatibility of subgrade soil was 2.50 ms, see Table (6.4).

Table 6.4: Summary of LWD results for subgrade soils of AL-Takahe district.

Station No.	***Surface Deflection (mm)***				***E_d (MPa)***	***D_c (ms)***
	S_1	***S_2***	***S_3***	***Mean***		
00+100	0.284	0.279	0.28	0.281	80.07	3.110
00+200	0.244	0.234	0.264	0.247	91.09	2.354
00+300	0.376	0.391	0.395	0.387	58.14	2.252
00+400	0.409	0.388	0.405	0.401	56.11	2.250
00+500	0.576	0.584	0.566	0.575	39.13	2.982
00+600	0.427	0.423	0.414	0.421	53.44	2.487
00+700	0.209	0.192	0.206	0.202	111.39	2.567
00+800	0.429	0.431	0.43	0.43	52.33	2.473
00+900	0.188	0.186	0.177	0.184	122.28	2.840
01+00	0.214	0.207	0.208	0.21	107.14	2.750
01+100	0.359	0.363	0.351	0.358	62.85	2.503
01+200	0.500	0.502	0.504	0.502	44.82	2.760
01+300	0.285	0.294	0.299	0.293	76.79	2.780
01+400	0.165	0.178	0.171	0.171	131.58	2.118
01+500	0.212	0.226	0.215	0.218	103.21	2.530
01+600	0.374	0.382	0.383	0.38	59.21	2.465
01+700	0.203	0.204	0.199	0.202	111.39	2.088
01+800	0.166	0.156	0.163	0.162	138.89	2.242
01+900	0.193	0.195	0.193	0.194	115.98	2.285
02+00	0.205	0.208	0.206	0.206	109.22	2.163
Average	**0.301**	**0.301**	**0.301**	**0.301**	**86.25**	**2.500**

6.4.2 **AL-Emam Ali District.**

The results of six LWD test points were performed on a well compacted subgrade surface were calculated by averaging the values resulted from three consecutive drops. The data extracted from the integration process indicate that vertical displacements varied from 0.225 to 0.393 mm, with an average deflection of 0.316 mm. The values of dynamic modulus ranged from 57.25 MPa to 100.00 MPa with an average of 73.648 Mpa. The average value of degree of compatibility of subgrade soil was 2.687 ms, see Table (6.5).

Table 6.5: Summary of LWD results for subgrade soils of AL-Emam Ali district.

Station No.	***Surface Deflection (mm)***				***E_d (MPa)***	***D_c (ms)***
	S_1	***S_2***	***S_3***	***Mean***		
00+100	0.32	0.319	0.274	0.304	74.01	2.548
00+250	0.377	0.381	0.37	0.376	59.84	2.436
00+400	0.223	0.229	0.223	0.225	100.00	3.285
00+550	0.398	0.392	0.389	0.393	57.25	2.652
00+650	0.295	0.297	0.29	0.294	76.53	2.734
00+750	0.303	0.302	0.304	0.303	74.26	2.465
Average	**0.319**	**0.320**	**0.308**	**0.316**	**73.648**	**2.687**

6.5 **Comparisons and Verification For Field Test Results**

In order to evaluate a predictive bearing resistance of subgrade soils, stress-penetration curves of laboratory CBR test were generated using the linear-fit regression curve for the granular soils and clayey soils, as shown in Figures (4.6) and (4.8). The results obtained from these generated curves were compared to the predictive results obtained from statistical analyses models. On the other hand, the results obtained by field test were compared with results obtained by laboratory results and the comparison was indicate that the field results were higher than the laboratory results as listed in chapter four. This increment was caused by the increasing in compaction effort and evaporation conditions and hence decreasing in the moisture content in the field condition. The comparison analysis was performed in the following major ways:

1. Dynamic parameters obtained from LWD test, water content, and dry density were compared between field sites results and laboratory tests results.
2. CBR value measured in terms of the stress-penetration curve (4.4) and (4.8) had to be considered as a measured CBR value.
3. Due to the increment in the field results, correction equations were required as clarified in Table (6.6).

4. The results of analytical and statistical analysis demonstrated in Table (6.7) through (6.9)

Table 6.6: Summary of correction equations

Parameters	***symbols***	***Correction equation***
S_d	S_{df}: field surface deflection S_{dc}: corrected surface deflection	$S_{dc} = -\ 0.0073\ S_{df} + 0.6676$
E_d	E_{df}: field dynamic modulus E_{dc}: corrected dynamic modulus	$E_{dc} = -\ 0.0804\ E_{df} + 40.814$
D_c	D_{cf}: field degree of compatibility D_{cc}: corrected degree of compatibility	$D_{cc} = -\ 0.1836\ D_{cf} + 4.0879$
W_c	W_{cf}: field water content. W_{cc}: corrected water content.	$W_{cc} = -\ 0.6756\ W_{cf} + 0.1578$

6.5.1 Comparison Between CBR and LWD parameters

Based on the results of analytical and statistical comparison analysis, a regression model CBR-LWD parameters model (S_d, E_d, and D_c) was adopted to predict a bearing resistance of subgrade soils. The resulting model can be utilized to compute CBR values of subgrades soils as a function of LWD parameters, see equation (6.1). Model reliability was highlighted through comparison of measured and predicted CBR and identifying measured to predicted ratio of CBR value (CBRm/CBRp) was (1.11) and (1.85) to AL-Takahe district and AL-Emam Ali district respectively as shown in Table (6.7).

$$\text{CBR predicted} = 17.225 + 0.051 * E_d + 0.023 * D_c^2 - 0.25 * D_c \qquad \text{-------------------} \qquad (6.1)$$

Table 6.7: Summary of CBRm and CBRp as a function of LWD parameters

Station No.	***CBR m****	***CBR p****	***CBR m*/ CBR p****
AL-Takahe district			
00+100	26.42	27.20	0.97
00+200	31.24	25.85	1.21
00+300	31.04	27.10	1.15
00+400	24.34	27.18	0.90
00+500	31.50	28.71	1.10
00+600	29.90	27.53	1.09
00+700	25.95	25.24	1.03
00+800	29.56	27.56	1.07
00+900	23.94	25.12	0.95
01+00	25.75	25.63	1.00
01+100	31.04	27.16	1.14
01+200	26.48	28.20	0.94
01+300	27.69	26.91	1.03
01+400	35.45	23.96	1.48
01+500	29.96	25.54	1.17
01+600	28.76	27.27	1.05
01+700	27.82	24.76	1.12
01+800	32.64	23.78	1.37
01+900	28.29	24.76	1.14
02+00	31.10	24.92	1.25
Average	**28.94**	**26.22**	**1.11**
AL-Emam Ali district			
00+100	49.97	26.75	1.87
00+250	47.90	27.21	1.76
00+400	49.33	26.62	1.85
00+550	53.62	27.56	1.95
00+650	49.81	26.86	1.85
00+750	49.02	26.65	1.84
Average	**49.941**	**26.94**	**1.85**

The result indicated that the developed model was acceptable and reliable to predict bearing resistance for A-1-b subgrade soils, while the difference between measured and predicted CBR value for A-3 subgrade soil was due to the higher value of measured CBR which was employed to build the model.

6.5.2 Comparison Between CBR-Physical Properties

The 2nd regression model as known CBR-physical properties (W_c and γ_{df}) was selected to predict a bearing resistance of subgrade soils. The resulting model can be utilized to compute CBR values of subgrade soil as a function of W_c and γ_{df}, see equation (6.2). Model reliability was highlighted through comparison of measured and predicted CBR and identifying measured to predicted ratio to CBR value (CBRm/CBRp) was (1.08) and (0.95) to AL-Takahe district and AL-Emam Ali district respectively, see Table (6.8). The results indicated that the developed model was acceptable and reliable to predict bearing resistance for A-1-b, and A-3 subgrade soils.

$$\text{CBR predicted} = 776.99 - 38 * W_c + 275 * \gamma_{df}^{\,2} - 912 * \gamma_{df} \quad \text{-------------------------} \qquad (6.2)$$

Table 6.8: Summary of CBRm and CBRp as a function of Wc and γ_{df}.

Station No.	*CBR m**	*CBR p**	*CBR m*/ CBR p**
AL-Takahe district.			
00+100	26.42	21.82	1.21
00+200	31.24	29.30	1.07
00+300	31.04	28.50	1.09
00+400	24.34	20.59	1.18
00+500	31.50	29.89	1.05
00+600	29.90	27.53	1.09
00+700	25.95	22.05	1.18
00+800	29.56	27.41	1.08
00+900	23.94	21.10	1.13
01+00	25.75	23.17	1.11
01+100	31.04	28.50	1.09
01+200	26.48	21.93	1.21
01+300	27.69	25.77	1.07
01+400	35.45	39.05	0.91
01+500	29.96	29.65	1.01
01+600	28.76	27.52	1.05
01+700	27.82	26.51	1.05
01+800	32.64	33.02	0.99
01+900	28.29	25.33	1.12
02+00	31.10	31.01	1.00
Average	**28.94**	**26.98**	**1.08**

Table 6.8: Summary of CBRm and CBRp as a function of Wc and γ_{df}.(continued)

AL-Emam Ali district.			
00+100	49.97	52.31	0.96
00+250	47.90	50.61	0.95
00+400	49.33	51.87	0.95
00+550	53.62	57.75	0.93
00+650	49.81	52.46	0.95
00+750	49.02	51.18	0.96
Average	**49.941**	**52.70**	**0.95**

6.5.3 Comparison Between CBR-LWD-Physical Properties

The 3rd regression model which developed previously in this research and known as CBR-LWD-physical properties (W_c and γ_{df}) was selected to predict a bearing resistance of subgrade soils. The resulting model can be utilized to compute CBR values of subgrades soils as a function of S_d, E_d, W_c and γ_{df}, see equation (6.3). Model reliability was highlighted through comparison of measured and predicted CBR and identifying measured to predicted ratio to CBR value (CBRm/CBRp) was (1.09) and (1.74) to AL-Takahe district and AL-Emam Ali district respectively as shown in Table (6.9) and showed an acceptable and reliable model for A-1-b, while the difference between measured and predicted CBR value for A-3 subgrade soil was due to the higher value of measured CBR which was employed to build the model.

$$\text{CBR predicted} = -0.139E_d + 1.129 * S_d * W_c - 6.39 * W_c + 36.585 \quad \text{----------------} \quad (6.3)$$

Table 6.9: Summary of CBRm and CBRp as a function of LWD parameters, W_c and γ_{df}.

Station No.	CBR m*	CBR p*	CBR m*/ CBR p*
AL-Takahe district.			
00+100	26.42	27.91	0.95
00+200	31.24	26.51	1.18
00+300	31.04	31.03	1.00
00+400	24.34	31.38	0.78
00+500	31.50	33.78	0.93
00+600	29.90	31.81	0.94
00+700	25.95	23.67	1.10
00+800	29.56	32.03	0.92
00+900	23.94	22.30	1.07
01+00	25.75	24.48	1.05
01+100	31.04	30.37	1.02
01+200	26.48	32.84	0.81
01+300	27.69	28.77	0.96
01+400	35.45	21.13	1.68
01+500	29.96	25.21	1.19
01+600	28.76	31.27	0.92
01+700	27.82	24.05	1.16
01+800	32.64	20.06	1.63
01+900	28.29	23.12	1.22
02+00	31.10	24.30	1.28
Average	**28.94**	**27.30**	**1.09**
AL-Emam Ali district.			
00+100	49.97	28.76	1.74
00+250	47.90	30.86	1.55
00+400	49.33	25.19	1.96
00+550	53.62	31.22	1.72
00+650	49.81	28.46	1.75
00+750	49.02	28.73	1.71
Average	**49.941**	**28.87**	**1.74**

6.6 **Summary**

Three regression models were developed by using statistical analysis and verified developed models at two field sites. Comparison and verification between measured and predicted results were conducted. The results showed that the three developed models were acceptable to predict the bearing resistance for A-1-b subgrade soils. While the modeling of CBR value for A-3 subgrade soil with reference to their basic physical properties is satisfactory for prediction of the bearing resistance of subgrade soils but un satisfactory to predict bearing resistance with reference to the dynamic measurement of LWD parameters inputs.

Chapter Seven

CONCLUSIONS AND RECOMMENDATIONS

7.1 Introduction

In this study, an attempt has been conducted to correlate the conventional test methods such as the California Bearing Ratio (CBR) with dynamic measurement such as light weight deflectometer for evaluating bearing resistance of local subgrade soils.

7.2 Conclusions

According to the achieved research works, the main conclusions for this research are listed below:

1. The subgrade soils at Karbala city is classified into:

- The south zones classify as granular soils type A-1-b and A-3 according to AASHTO classification system, and their CBR value ranges from 13% to 51%.
- The west zones classify as clayey soils type A-7-6, and have bearing capacity varied from 7% to 17%.

2. Regression analyses for developed models show the following:

- CBR value for both granular and clayey soils with reference to the soil's physical parameters input is achievable and satisfactory in terms of prediction and the significance of input parameters with multi nonlinear models.
- The γ_{df} has the most significant correlation than Wc to predict bearing resistance for both granular and clayey soils.
- The prediction CBR value for (LWD) parameters are achievable and satisfactory.

- The E_d is the most significant parameter in predicting CBR than the S_d , and lastly the D_c for granular and clayey soils.

3. The results of comparison and varification show the following:

- Modeling of CBR value for A-1-b subgrade soil with reference to both their basic physical properties and the dynamic measurement of LWD parameters inputs is achievable and satisfactory in terms of prediction the bearing resistance of subgrade soils.
- Modeling of CBR value for A-3 subgrade soil with reference to their basic physical properties is satisfactory for prediction of the bearing resistance of subgrade soils but un satisfactory to predict bearing resistance with reference to the dynamic measurement of LWD parameters inputs.

4. The results of this study suggest LWD can be reliably used to predict CBR values, and hence can be used to evaluate the stiffness/strength parameters of different subgrade soils.

7.3 Recommendations

Within the scope of this research work and the obtained data, several recommendations are presented below:

1. For further practice, LWD test is a vital solution for predicting the strength of subgrade soils by using correlating with conventional tests and the current study is highly recommended this dynamic measurement to determine bearing resistance for local subgrade soils.
2. It is recommended to select more types of subgrade soils as classified according the AASHTO classification system and more parameters such as C_c, C_u, D_{10}, D_{30}, D_{60}, and plasticity indices to develop other statistical models.

3. It is recommended to determine measured CBR by using in-situ test device instead of the approach was adopted in this research to ensure the accuracy for validated results.
4. It is recommended to stabilize the subgrade soils to ensure the improvement of their physical and mechanical characteristics.
5. It is also recommended to conduct theoretical work by using finite element analysis to determine the bearing ratio of subgrade soils and compared with predicted soils model by using statistical analysis.

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الخلاصـــــــــــــــــة

في تصميم رصف الطرق السريعة والمطارات ، تتأثر قوة الطبقة السفلية في الغالب بسمك طبقات الرصف ، وتعد قابلية التحمل الكلورفوني (CBR) واحدة من الطرق التجريبية البسيطة التي تستخدم عادة لتحديد قوة التربة لطبقات الارض الطبيعية وطبقات الحصى الخابط ووطبقات الرصف السفلية في الطرق السريعة و المطارات. يستخدم اختبار CBR بشكل أساسي لتحديد االسمك المطلوب لطبقات التبليط. يتم إجراء فحص قابلية التحمل الكلورفوني مختبريا او حقليا. يمتاز فحص قابلية التحمل الكلورفوني الحقلي In-situ CBR بقدرة فحص تحمل التربة دون الحاجة إلى دون الحاجة الى اخذ عينات وفحصها مختبريا. ومع ذلك ، نظرًا لأن اختبار CBR شاق ويستغرق وقتًا طويلًا إلى جانب تطوير تقنيات فحص جديدة ، تم ايجاد علاقات لربط قيمة CBR بمتغيرات الديناميكية التي يتم الحصول عليها بواسطة اجهزة فحص حديثة كجهاز فحص الهطول خفيف الوزن (LWD)

في هذا البحث ، تم استخدام نوعين من الفحوصات على ثلاثة أنواع من التربة الفرعية: فحص قابلية التحمل الكلورفوني الموقعي In-situ CBR ، وطريقة الفحص الديناميكي تم تنفيذها باستخدام جهاز فحص الهطول خفيف الوزن LWD لايجاد الخصائص الديناميكية ومعامل المرونة للتربة. تم اختيار ثلاثة مشاريع طرق في مدينة كربلاء لتقييم تربة مختلفة. المواقع هي منطقة الميلاد ، منطقة الفارس ، ومنطقة الرفيع ، حيث صنفت أنواع التربة فيها على أنها A-1-b و A-3 و A-7-6 ، على التوالي.

علاوة على ذلك ، تم إجراء تحليلات إحصائية لتقييم مقاومة تحمل التربة اعتمادًا على المتغيرات الحاصل عليها من اختبار LWD مثل المعامل الديناميكي والانحراف السطحي ، ومحتوى الماء والكثافة الجافة التي تم الحصول عليها من فحص الكثافة الحقلية. تشير نتائج ستة نماذج إحصائية للترب (الحبيبية و الطينية) إلى إمكانية كبيرة لايجاد علاقة بين اختبار CBR و LWD لايجاد مقدار تحمل التربة بالإضافة إلى ذلك ، تشير النتائج إلى أن معامل الديناميكي من اختبار LWD يرتبط بشكل جيد مع نسبة تحمل لمختلف أنواع التربة. علاوة على ذلك ، ترتبط الكثافة الحقلية ارتباطًا كبيرًا بقيمة CBR ، في حين تشير المقارنة بين المتغيرات الديناميكي والخصائص الفيزيائية إلى أن المعامل الديناميكي هو الأكثر أهمية من الكثافة الحقلية

للتحقق من النماذج الإحصائية ، تم اختيار موقعين في مدينة كربلاء: منطقة الطاقة ، وطرق فرعية في حي الإمام علي . تم تصنيف تربة تلك الطرق طبقًا لـتصنيف AASHTO لتكون A-1-b و A-3 ، على التوالي. أظهرت النتائج الأولية القدرة على التنبؤ بمقاومة تحمل التربة بناءً على خصائصها الفيزيائية والديناميكية .

جمهوريـــــة العراق
وزارة التعليم العالي والبحث العلمي
جـــامعة كربـــــلاء
كليـــــة الهندســـــة
قسم الهنسة المدنية

التقييم اللاائتلافي لمقاومة تحمل التُرب المحلية تحت الطرق بإستخدام مقاييس فحص الهطول خفيف الوزن

رسالـــــــــــــة

مقدمة إلى قسم الهندسة المدنية في جامعة كربلاء

وهي جزء من متطلبات الحصول على درجة الماجستير في الهندسة المدنية

(هندسة البنى التحتية)

من قبل

رَوْق محمد عبد علي آل فتح الله

بكالوريوس في الهندسة المدنية – جامعة بابل 2007-2008

بإشراف

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أيلول 2019 محرم 1441