Republic of Iraq
Ministry of Higher Education
and Scientific Research
University of Technology

Field Models on Gypseous Soils Reinforced with Stone Columns Stabilized with Asphalt and Lime

A Thesis
Submitted to the Department of Building and Construction Engineering at University of Technology in Partial Fulfillment of the Requirements for the Degree of Master of Science in Geotechnical Engineering

Submitted By

Nihad Bahaaldeen Salih
(B.Sc. Civil Engineering, 1996)

Supervised by

Prof. Dr. Namir K. S. Al-Saoudi

Prof. Dr. Hussein H. Karim

March - 2003

بسم الله الرحمن الرحيم

وَمَا تَشَاءُونَ إِلَّا أَنْ يَشَاءَ اللَّهُ إِنَّ اللَّهَ كَانَ
عَلِيمًا حَكِيمًا

صدق الله العظيم

سورة الإنسان – آية 30

Certification

We certify that this thesis entitles " ***Field Models on Gypseous Soils Reinforced with Stone Columns Stabilized with Asphalt and Lime*** " was prepared by **Nihad Bahaaldeen Salih** under our supervision at the Building and Construction Engineering Department, in the University of Technology, in partial fulfillment of the requirements for the Degree of Master of Science in Geotechnical Engineering.

Signature:
Name: Prof. Dr. **Namir K. S. Al-Saoudi**
Date: 25/5/2003

Signature: HHKarim
Name: Prof. Dr. **Hussein H. Karim**
Date: 25 / 5 /2003

In view of the available recommendation, I forward this thesis for debate by examining committee.

Signature:
Name: Prof. Dr. **Namir K. S. Al-Saoudi**
Chairman of Highways and Bridges Engineering Branch
Date: 25 / 5 /2003

Committee Certification

We certify that we have read this thesis entitled ***"Field Models on Gypseous Soils Reinforced with Stone Columns Stabilized with Asphalt and Lime"*** and as examining committee examined the student ***Nihad Bahaaldeen Salih*** in its content and in what is connected with it, and that in our opinion it meets the standards of a thesis for the degree of Master of Science in Geotechnical Engineering.

Signature:
Name: Prof. Dr. Namir K. S. Al-Saoudi
Date: 17 / 7 / 2003
Supervisor

Signature:
Name: Prof. Dr. Hussein H. Karim
Date: 17/ 7/ 2003
Supervisor

Signature: T. M. Toma
Name: Asst. Prof. Dr. Tahsin M. Toma
Date: 17/ 7/ 2003
Member

Signature:
Name: Asst. Prof. Dr. Kowkab H. Al-Raw
Date: 20/ 7/ 2003
Member

Signature: Y. Sh
Name: Prof. Dr. Yousif J. Al-Shakarchi
Date: 21 / 7/ 2003
Chairman

Signature:
Name: Prof. Dr. Namir K. S. Al-Saoudi
The Head of Building and Construction Engineering Department
Date: 17 / 7 / 2003

Dedication

To

My Dear Father

My Merciful Mother

My Brothers and Sisters

Whom I love

With my respect

Nihad

Acknowledgment

In the name of **Allah**, the most compassionate, the most merciful.

Praise be to **Allah** and pray and peace be on his prophet **Mohammed** (Allah's blessings and peace be on him) his relatives and companions and all those who follow him.

First of all, thanks for **Allah** who enabled me to achieve this research.

I would like to express my sincere thanks and gratitude to my supervisor **Dr. Namir K. Al-Saoudi** for his guidance, advice and giving generously of his time during the study preparation of the manuscript.

Sincere thanks are due to my Co. supervisor **Dr. Hussein H. Karim** for his valuable aid and comments.

Sincere appreciation to the staff of State Establishment of Geological Survey and Mining, especially geologist **Abdul-Haq Ibrahim**. My special thanks to the staff of Sodium Sulphate Factory, especially the factory director geologist **Ghazi Ateya**.

Thanks are presented to the staff of the central library, University of Technology, and especial thanks to Miss. **Sura Seleam** (Building and Construction Engineering Department Library).

My special thanks to Eng. **Ahmed Salih** for his kindest helping me along my study and research. Special thanks to My friends Eng. **Jasim Mustafa**, Eng. **Ahmed Nihad**, and Eng. **Yasir Khalil**.

Finally, my special thanks to **my family** those who gave me all the supports and love.

Abstract

Stone columns are a well-known technique mainly used to reduce compressibility and improve the bearing capacity of soft saturated soils. It has been considered as the most successful improving method.

The present work investigates the possibility of using stone columns stabilized with 3.75% liquid asphalt and 7.5% lime of the total weight to control the collapsibility of gypseous soils due to wetting. It must be stated here that no pervious works have been carried out considering stone columns as a remedial tool for gypseous soils.

Four Field model footings were performed on a site near the Sodium Sulphate Factory in Al-Dour region. Each model footing is 1.25m by 1.25m, two were placed directly on the ground and the other two placed on ground treated with four stabilized stone columns, (0.3m in diameter and 1.5m in length). 32 and 44.8 kPa stresses were applied on each footing. Flooding of the area surrounding the four footings was carried out for 90 days and continuous monitoring of settlement was recorded.

The field model tests revealed an encouraging reduction in settlement due to the presence of stone columns. About 33% and 50% reduction in settlement were observed at applied stresses of 32 and 44.8 kPa.

Finally, the stone column technique may be considered as a successful technique for controlling the collapsibility of gypseous soils.

List of Contents

List of Tables

List of Figures

List of Plates

List of Notations

Mc-30	Medium Curing Asphalt (Liquid Asphalt)
FAO	Food and Agriculture Organization
NCCL	National Center for Construction Laboratories
G.C.	Gypsum Content, %
c	Cohesion, kPa
ϕ	Angle of Internal Friction, degree
C_r	Swelling Index
C_c	Compression Index
γ_d	Dry Unit Weight, kN/m^3
Gs	Specific Gravity
CBR	California Bearing Ratio
L.L.	Liquid Limit of the Soil
P.L.	Plastic Limit of the Soil
P.I.	Plasticity Index
O.M.C	Optimum Moisture Content, %
Cu	Coefficient of Uniformity (D_{60} / D_{10})
c_u	Undrained Shear Strength, kPa
Ar	Area Replacement Ratio
Ls/Ds	Length of Stone Column to Diameter Ratio
D_{10}	Diameter at which 10% of the Crushed Stone is Finer
D_{30}	Diameter at which 30% of the Crushed Stone is Finer
D_{60}	Diameter at which 60% of the Crushed Stone is Finer
Cc	Coefficient of Curvature (D_{30} / D_{60} * D_{10})
U.C.S	Unconfined Compressive Strength
Cp	Collapse Potential, %
T.D.S	Total Dissolved Solids

W.O.C.	Water of Crystallization
q	Applied Stress, kPa
ν	Poisson's Ratio = 0.35
B	Width of Footing, m
S	Settlement Reading, mm
E	Modulus of Deformation, kPa

Chapter One

1

Introduction

Chapter One
INTRODUCTION

1.1 General

Gypseous soils cover about 31.7% of the total surface area of Iraq (Ismail, 1994). They are distributed in different zones with variable gypsum content but concentrated mainly in Jazirah region, Pleistocene Terraces, and the Mesopotamian Plain (Mohammed, 1993). These soils are present in the basin of Iraq where geologically belong to 1: the Lower Fars formation of the Miocene age, as in Al-Jazirah area and the upper and middle plains of the Euphrates and Tigris rivers, 2: from the Pleistocene age, as in the middle and south of Iraq, Barazanji 1973. Although in some zones they belong to the Upper Fars.

Due to the vast development that took place in Iraq during the last two decades, different types of structures have been built and a number of airfield and highway networks have been constructed. New forms and patterns of failures started to raise and geotechnical and structural engineers have to accept this challenge and provide safety measures and remedies.

Researchers at different institutes carried out a lengthy testing program in attempts to understand the geotechnical properties of gypseous soils and propose solutions for the existing problems. The methodology cumulating from element tests, small-scale model tests in addition to the use of numerical techniques such as finite element. Due to the lack of standardized testing procedures and limited amount of available data led to conflicting conclusions.

Under these circumstances some vague remedial proposals were made. These proposals show the idea of providing some barriers or use of some asphaltic or chemical additives that control and prevent the movement of water from coming underneath the foundations. Some of these techniques which are based on small-scale model tests showed promising results although they have not been applied in the field yet.

The present work is based on monitoring the behavior of four full-scale footings constructed on gypseous soil in the field. Two models were supported on untreated soil and two on soil treated with stone columns.

1.2 Purpose of Study

The main objective of this research is to investigate the feasibility of reinforcing gypseous soils (with high gypsum content) by stone columns stabilized with (cutback asphalt, M-30, and lime). This attempt is made to check weather the stone columns will improve or control the collapse potential of gypseous soil. To arrive to such conclusions, four full-scale model tests were conducted and monitored under different conditions.

1.3 Scope of Study

The skeleton of the thesis is divided into six chapters, brief introduction is presented in chapter one, while chapter two demonstrates the distribution of gypseous soils and some of their properties. Problems and remedies are also discussed in this chapter. Chapter three demonstrates a brief review about the mechanism and uses of stone column technique. Chapter four is devoted to experimental and field works. The details of the site and the location of the field tests in addition to the physical, chemical and mineralogical properties of the soil in the site are outlined. Full description of the different stages of the field test is also presented. Chapter five covers a thorough discussion of the full-scale model tests results and the supplementary laboratory work. Conclusions and recommendations for further work are outlined in chapter six.

Chapter Two

2

Gypseous Soils Problems and Remedies

Chapter Two

GYPSEOUS SOILS PROBLEMS AND REMEDIES

2.1 General

Gypseous soils are abundant in Iraq; they cover about 12% of total surface area (FAO, 1990). These soils created a number of geotechnical problems in the last two decades due to their unpredictable behavior and due to the complexity of the behavior of such soil. The gypsum content varies widely from very low, less than 1% to very high exceeding 80% (Buringh, 1960).

In spite of the vast research carried out at different institutions during the past, there is still a deficiency in understanding the behavior of such problematic soils.

2.2 Formation and Distribution of Gypseous Soils in Iraq

Gypseous soils were first recognized and introduced to soil science by W. Knop in 1871 (quoted by Dokuchaev vol.4, 1896) under the name of sulphate soils, (from soil resources, management and conservation service, FAO land and water development division). They are present in different continents like Europe, Asia, and Africa. Table 2.1 illustrates the percentages of the total areas covered by Gypseous soils in these continents (FAO, 1990).

Iraq is considered as one of the countries that about 12% of its surface area is covered with gypsum. The earlier work concerned with gypsum in Iraq presented by Buringh 1960, where he managed to present the first map considering the distribution of gypsum and its geological formation. Figure 2.1 illustrates the first map of gypseous soils. Buringh's map was based on five zones, primary gypsum, primary gypsum mixed with lime stone, secondary gypsum, gypsiferous alluvium, and non-gypsiferous, mainly lime stone. The map does not cover or show the distribution of gypsum content. Following that, Barazanji 1973, after a thorough investigation presented a more refined map with terms like slightly, moderately, too highly …etc as shown in figure 2.2.

In 1993, the State Establishment of Geological Survey and Mining had provided a more detailed map concerning the formation ages for the over all area of Iraq. It provides an information about the gypseous soils in Iraq as shown in figure 2.3.

The most recent study concerning the formation of gypsum is demonstrated in figure 2.4. This study shows the depth distribution of the Fatha formation which consist mainly gypsum, limestone, and clay.

***Table (2.1) Detailed of Distribution of Gypseous Soils Selected by Countries* (FAO, 1990)**

Continent	Country	Area (Km^2)	(%) of total area country	(%) of area of gypsiferous soils
Africa	Morocco	1114.3	2.5	1.7
	Algeria	7966.3	3.3	12.2
	Tunisia	1439.8	9.3	2.2
	Libya	3956.8	2.2	6.0
	Egypt	382.2	0.4	0.6
	Sudan	785.0	0.3	1.2
	Somalia	10161.2	16.2	15.5
	Ethiopia	1423.4	1.3	2.2
	Mali	2818.3	2.3	4.3
	Mauritania	396.0	0.4	0.6
	Namibia	5327.7	6.5	8.2
South Asia	Syria	3966.6	21.6	6.0
	Jordan	80.5	0.8	0.1
	Saudia Arabia	82.5	0.04	0.1
	Oman	471.6	–	0.7
	Yemen A.R.	2931.0	8.8	4.5
	Kuwait	354.6	–	0.5
	Iraq	4779.2	12.0	7.3
	Iran	4.2	–	–
	Pakistan	9.5	0.01	–
	India	182.0	0.2	0.3
Central Asia	USSR	5074.1	0.04	7.7
	Mongolia	60.9	0.04	0.1
	China	11484.9	1.2	17.5
Europe	Turkey	64.2	0.8	0.1
	Spain	165.5	–	0.3
North America	New Mexico	78.0	–	0.1

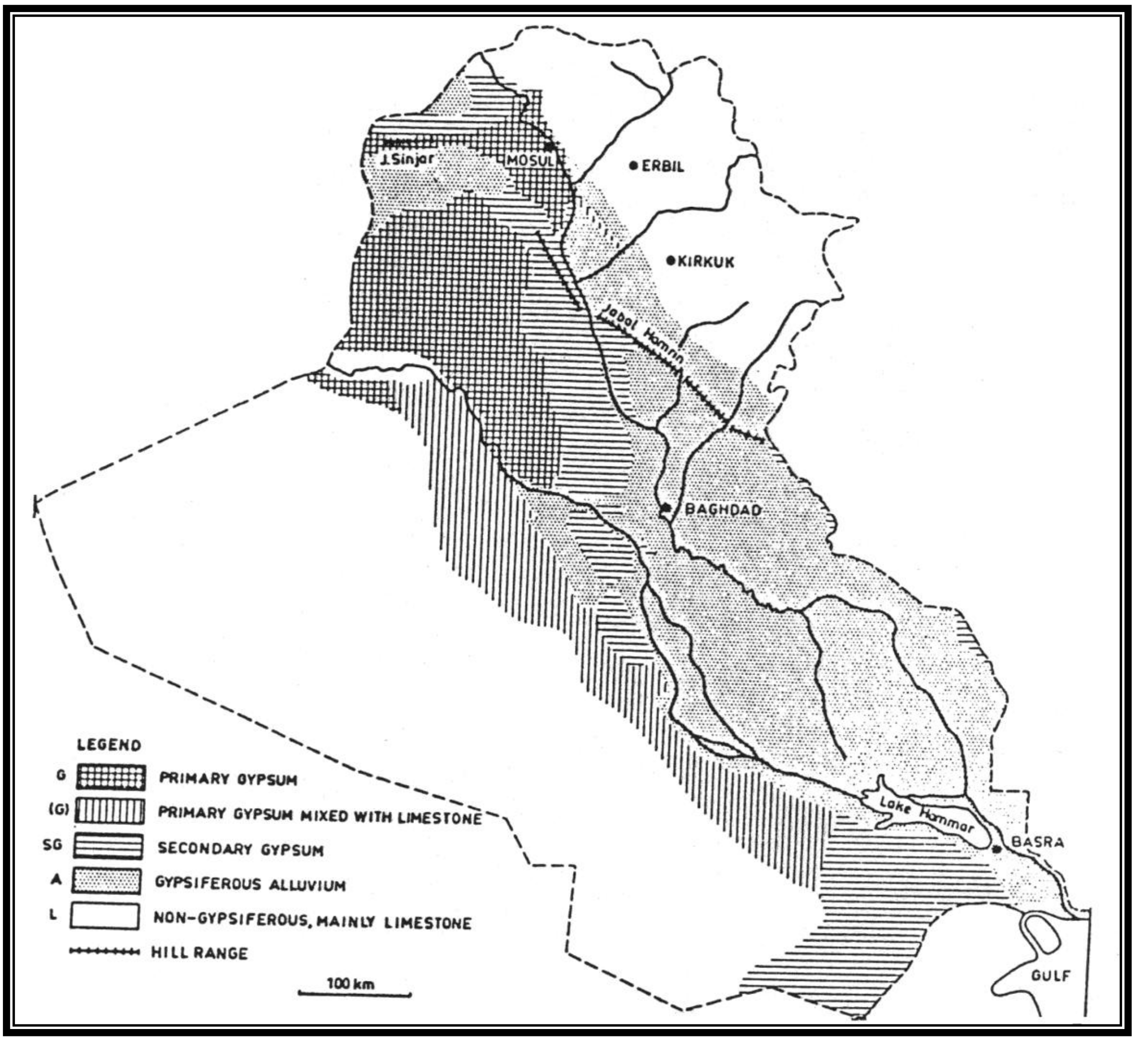


Figure (2.1) Gypsum Map of Iraq, (after Buringh, 1960)

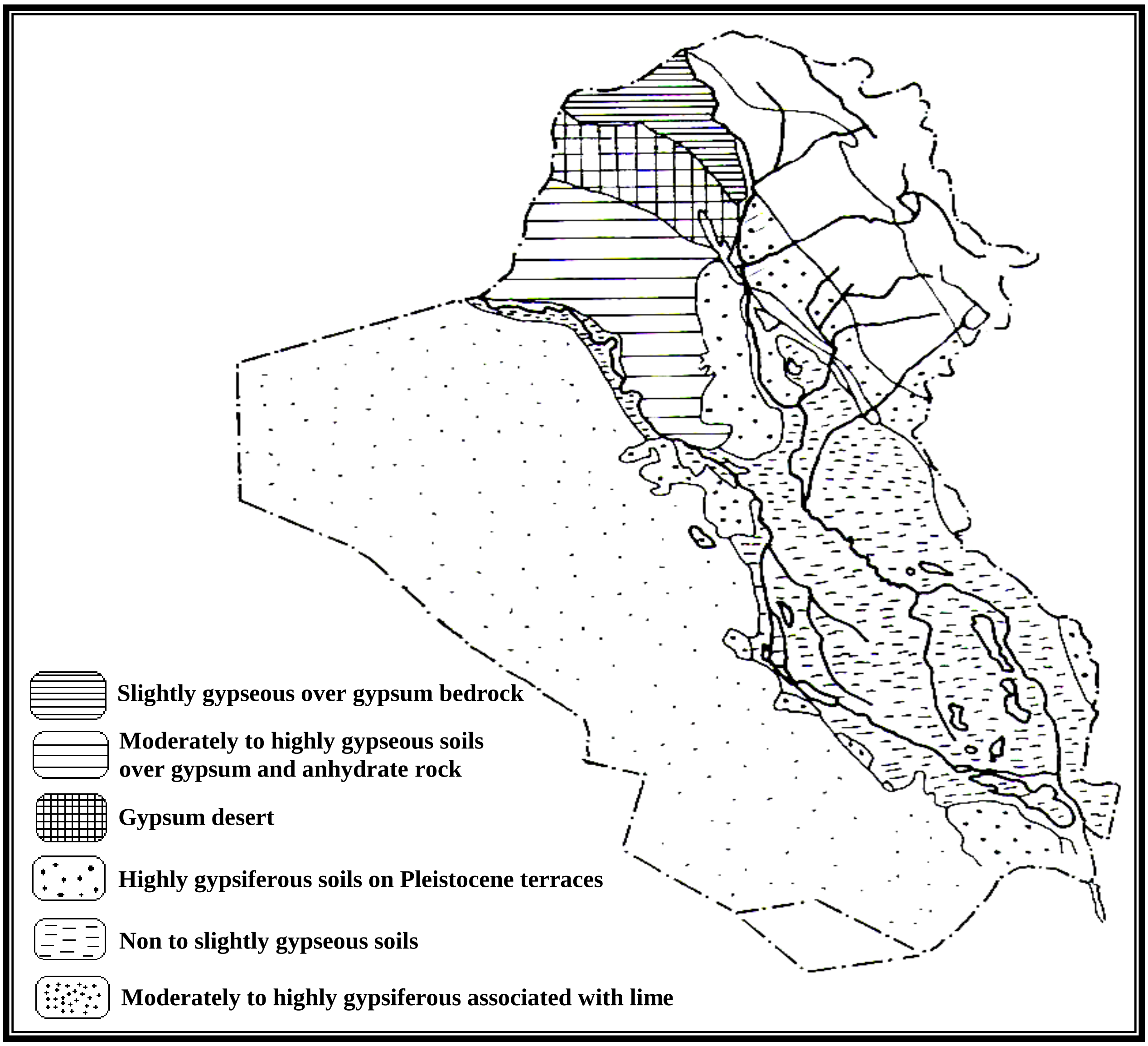


Figure (2.2) Regional Distribution of Gypseous Soils in Iraq, (after Barazanji, 1973)

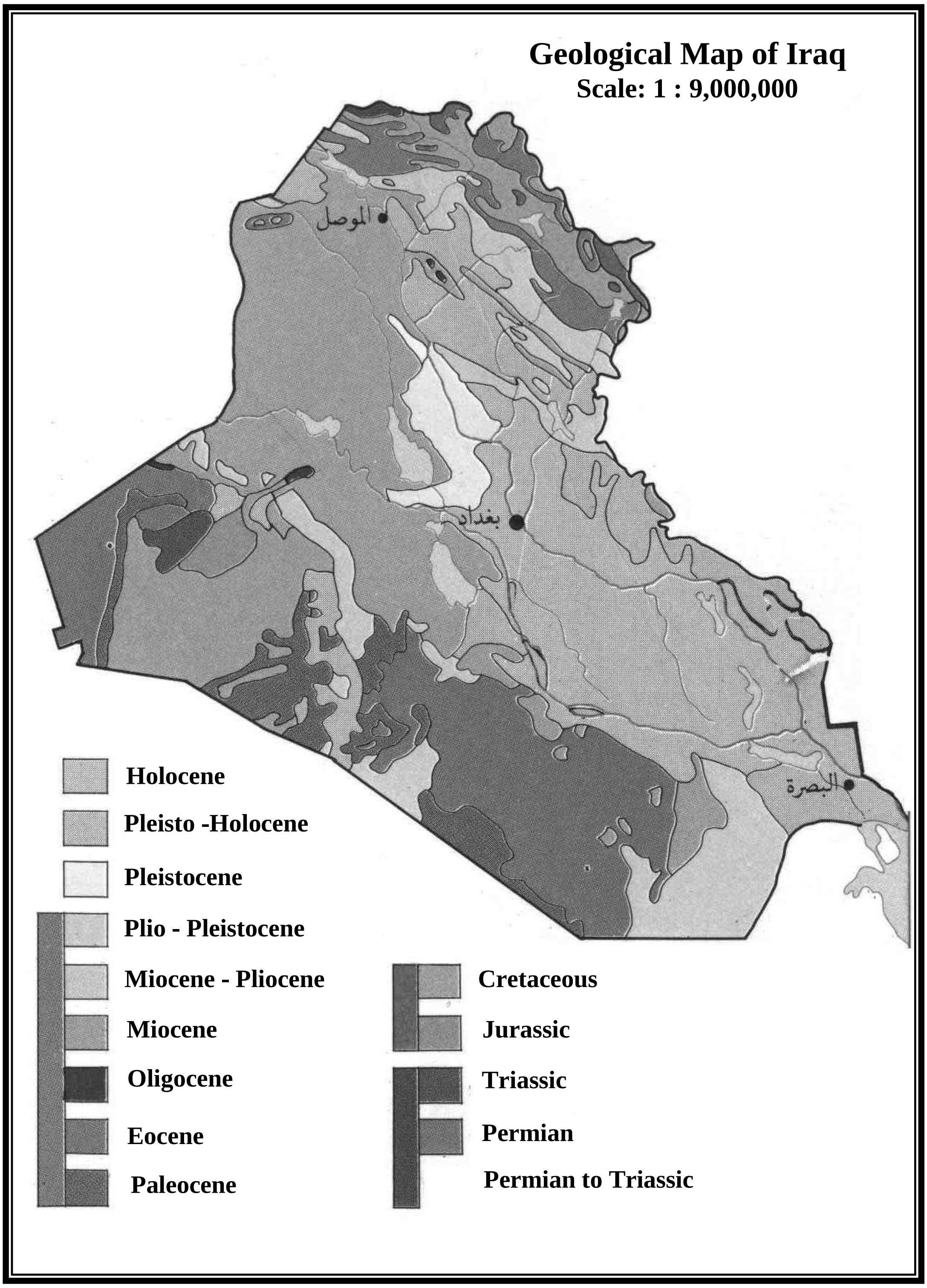


Figure (2.3) Geological Map of Iraq, (State Establishment of Geological Survey and Mining, 1993)

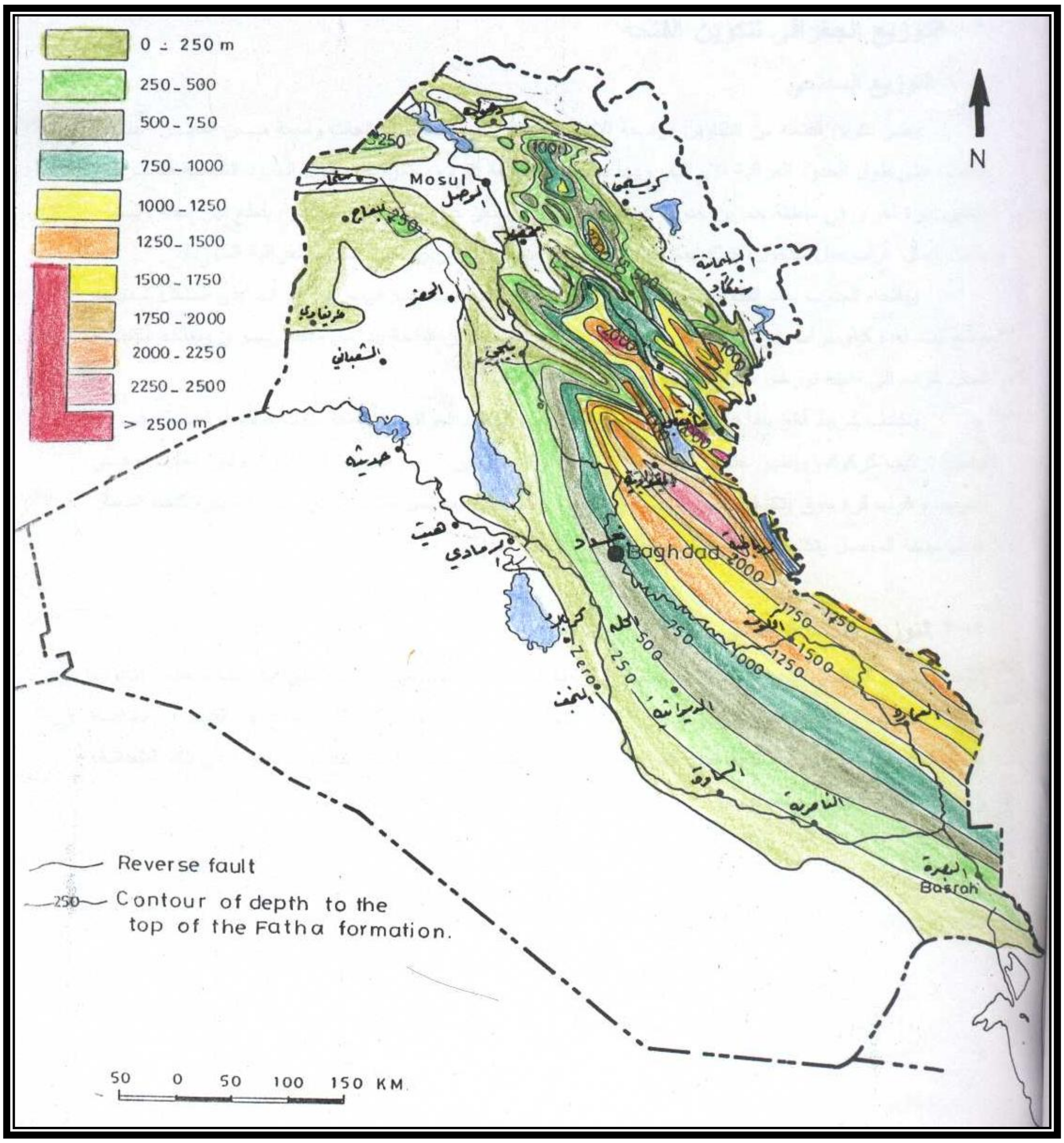


Figure (2.4) Depth Distribution of the Fatha Formation (State Establishment of Geological Survey and Mining, 2002)

2.3 Classification of Gypseous Soils

There is no clear classification system available for the gypseous soils in Iraq. However, Barazanji 1973 classified the gypseous soils according to their gypsum content for agricultural porposes as shown in table 2.2. Where, Nashat 1990 suggested another classification for gypseous soils for (NCCL) as shown in table 2.3.

Table (2.2) Classification of Gypseous Soils (after Barazanji, 1973)

Gypsum Content ($CaSO_4.2H_2O$)%	Classification
0.0-0.3	Non-Gypsiferous
0.3-3	Very Slightly Gypsiferous
3-10	Slightly Gypsiferous
10-25	Moderate Gypsiferous
25-50	Highly Gypsiferous

Table (2.3) Classification of Gypseous Soils For NCCL (after Nashat, 1990)

Gypsum Content ($CaSO_4.2H_2O$)%	Classification
0-10	Slightly
10-25	Moderately
25-50	Highly
> 50	Gypcrete

2.4 Some Geotechnical Properties of Gypseous Soils

It has been stated that during the last two decades, a number of published data have been reported concerning the geotechnical properties of gypseous soils. Most of these data were oriented towards the compressibility, shear strength and collapsibility of gypseous soils.

Due to the lack of detailed specifications and standards the collected data in most cases showed conflicting results. For these reasons the most important parameters and their variation with gypsum content are outlined in table 2.4. The table consists of the properties and their trend with increasing or decreasing gypsum content. The arrows indicate the type of variation.

Table (2.4) Some Geotechnical Properties of Gypseous Soils

No.	Property	Range of G.C. (%)	Variation with G.C.	References
1	Cohesion (c)	0 – 5	↑	Petrukhin and Arakelyan (1985)
		> 15	↓	
		0 –5	↑	Al- Dilaimy (1989)
		5 – 20	↓	
		26 – 80	↑	Seleam (1988)
		↑	↓	Al-Nouri and Al- Qaissy (1990)
2	Friction Component (ϕ)	0 – 20	↑	Petrukhin and Arakelyan (1985)
		> 20	↓	
		↑	↑	Al-Nouri and Al- Qaissy(1990)
		26 – 80	↑	Seleam (1988)
		0 – 5	↑	Al- Dilaimy (1989)
		5 – 20	↓	
3	Swelling Index (Cr)	↑	↓	Al- Qaissy (1989)
		0 – 3	↓	Ramiah (1982)
		3 – 6	↓	

Continued

4	Compression Index (Cc)	0 – 3 3 – 6 26 – 80 0 – 10 10 – 20 ↑	↑ ↓ ↓ ↑ slow rate ↑ rapid rate ↑	Ramiah (1982) Seleam (1988) Al- Dilaimy (1989) Al- Qaissy(1989)
5	γ_d (Max.)	0 – 15 > 15 ↑ ↑ 0 - 5 5 – 20 ↑	↑ ↓ ↓ ↓ ↑ ↓ ↑	Kattab (1986) Subhi (1987) Al-Ani et al. (1988) Al- Dilaimy (1989) Al- Heeti (1990)
6	Specific Gravity (Gs)	↑	↓	Seleam (1988)
7	CBR	0 – 15 > 15	↑ ↓	Kattab (1986)
8	Liquid Limit (L.L.)	↑ ↑ ↑ ↑	↓ ↑ ↓ ↑	Salas et al. (1973) Kattab (1986) Subhi (1987) Al- Ani et al. (1988) Al- Heeti (1990) Al- Dilaimy (1989) Al- Qaissy (1989) Singh and Layla (1979)
9	Plastic Limit (P.L.)	↑ ↑	↓ ↓	Al-Qaissy (1989) Singh and Layla (1979)

Continued

10	Plasticity Index (P.I.)	↑	↓	Salas et al. (1973) Kattab (1986) Subhi (1987) Al- Ani et al. (1988) Al- Heeti (1990)
		↑	—	Al- Qaissy (1989)
			↑	Singh and Layla (1979)
		↑	↓	Al- Dilaimy (1989)

2.5 Problems of Gypseous soils

The problems of encountered with gypseous soils are of different types. Some are related to the changes in their geotechnical properties due to their high sensitivity to environmental conditions such as wetness. Other problems are related to constructional techniques and to the type of the structure and extent of the foundations.

The first category of problems can be illustrated (Al- Mufty, 1997):

1. The non- homogeneity of gypseous soil.
2. Great losses in strength up on wetting.
3. Sudden increase in compressibility upon wetting.
4. Continuation of deformation and collapse upon leaching due to water movement.
5. The existence of cracks due to seasonal changes.
6. The existence of holes due to local dissolution of gypsum.
7. Delayed compression continues due to solution precipitation creep even in apparently dry soil.
8. An abundance of holes caused by rodents and insects.

The above points are related mainly to changes that take place when water from any source become in contact with gypseous soil.

Many of the reported failure events refer to causes of failure of structure founded on gypseous soil to the dissolution of gypsum salts when brought in contact with water. The gradual development of cavities underneath the foundation during the lifetime of the structure will subject the structure to sudden collapse or tilt (Van Alphen and Romero 1971, Mikheev and Petrukhin 1973, Dudley 1970 and Clemence 1981, Razouki et al 1994, Al-Badran 2001).

The time required for the developments of cavities depends on the many factors related to the water percolation, the metastable structure of the soil itself and to its physico-chemical properties.

In Iraq, some failure cases of structures founded on gypseous soils have been recognized.

Excessive settlement took place in dwelling houses and other structures in Qayiara refinery area that is mainly the results of the top soil there, Taha (1979).

Cracks in dwelling houses of Dewania area, and the water seepage in their basements are faced due to their erection on gypseous soils, Al-Khashab (1981).

Samara tourist hotel, by the effect of leaking water from fire hydrant, the leakage caused water seepage and washed the soil under part of the foundation of the hotel, Saaed et al (1989).

Grouting was used in the foundation of saddam dam for filling the cavities under it. This occurred due to the continuous dissolution of gypsum, Nashat (1990).

Cracks appeared on the walls of the Tikrit training center due to long term settlement and loss of strength, NCCL (1992).

Severe damages in irrigation structures of Mendeli irrigation projects, this problem occurred due to leaching of gypseous soils, Jawad (1994).

2.6 Available Remedies for Gypseous Soils

Since the problems with gypseous soils is the drastic changes in its geotechnical properties upon wetting, thus most of the proposed remedies are basically concerned with some sort of treatment or precautions that prevent or control any contact between the soil underneath the foundation and any source of water.

Other remedies prefer the use of chemical additives that are basically mixed with the gypseous soil to provide some chemical bonds, or some chemicals are used to coat the gypsum particales and prevent these from dissolution.

2.6.1 Chemical Treatment

Cement, Lime, and Asphalt are the main chemical additives that have been proposed for the treatment of gypseous soils.

2.6.1.1 Cement Treatment

Kattab 1986, investigated the use of sulphate resisting cement to improve granular gypsified soil. He found the Unconfined Compressive Strength (U.C.S.) increases with increasing cement content immersion of gypseous soils in water reduced the compressive strength, tensile strength and modulus of elasticity of soils stabilized with different percentages of cement. Greater reduction was observed with higher gypsum content.

Al-Hadithy 2001, investigated the possibility of using sulphate resisting cement to control the compressibility of four saline soils brought from Baghdad, Faluja, Kerbala, and Basra. The four soils have been treated with 3%, 5%, 7%, and 10% cement by weight and cured for 7 days, 14 days, and 21 days. It was found that the days curing period is sufficient for the completion of the major reactions between the cement and the saline soils. It was also noticed that the compressibility is improved by the addition of cement and all soils exhibited a decrease in the compression index (C_c) with increasing cement content, and increasing curing period.

2.6.1.2 Lime Treatment

Lime is probably the most widely used additive to improve the geotechnical properties of many soils. It is considered as one of the oldest improvement techniques. The possibility of using lime as an improvement additive with gypseous soils has not been investigated thoroughly. Limited local research work is available.

Al-Obaydi 1992, investigated the suitability of lime to stabilize three types of gypseous soils brought from Al-Jazirah region with (7%, 23%, and 34%, gypsum content). He found that the gypseous soils can be successfully stabilized with 5% lime improves their strength and reduction the permeability. Also he found that the leaching had little effect on strength and compressibility of treated soil as compared with untreated.

Al-Zory 1993, used 5% lime to stabilize natural gypseous soil (43% gypsum content) brought from Al-Jazirah region. The treated soil samples were cured for (2, 7, 14, and 28 days) under (25 ^{o}C) temperature degree. The stabilized soil demonstrated higher resistance to leaching compared to the untreated soil. It was also noticed that samples cured for (14 and 28 days) became practically impermeable after (50 days) of leaching. The cementation caused by the soil-lime reactions is the main factor controlling the leaching effects on the lime stabilized gypseous soils.

2.6.1.3 Petroleum Product Treatment

Various types of petroleum products such as gasoline, kerosene, gas oil, fuel oil, and asphalt have been investigated as improvement agent for gypseous soil. These liquids have viscosity less than water, and do not dissolve in water. These materials will tend to cover the soil particles and prevent them from being in contact with water.

2.6.1.3.1 Kerosene Treatment

Seleam 1988, examined the possibility of using kerosene and gas oil to stabilize these gypseous soils with (40%, 50% gypsum content) brought from Habanya area. The additives decreased the compressibility and permeability by delaying the removal of gypsum. There was no clear influence of the kerosene or gas oil on the strength components.

Al-Aqaby 2001, used kerosene for stabilizing gypseous soils brought from Baiji city with (37%, 58%, and 65% gypsum content). It was found that the cohesion component of the uncontaminated gypseous soil decreased after soaking whether the pore fluid was water or kerosene, while the angle of internal friction is relatively unchanged upon soaking in water and decreased about six degree upon soaking in kerosene.

2.6.1.3.2 Automobile Oil Treatment

Al-Kaisi 1997, used the automobile oil as a remedy for controlling the behavior of gypseous soil. The influence of (4%) automobile oil reduced the hydraulic conductivity by not less than ten folds.

Considerable durability was noticed against the sustained hydraulic gradient.

2.6.1.3.3 Fuel Oil Treatment

Al-Hassany 2001, investigated the possibility of using fuel oil to improve gypseous soil with high percentage of gypsum content. Two types of gypseous soil brought from Al-Therthar region (soil I with 51.6% gypsum content, and soil II with 26.55% gypsum content) were considered. The fuel oil tends to stabilize the soil and prevent water percolation down ward, and hence decreases the permeability, increases the durability, decreases the compressibility by decreasing the void ratio; compression index; swelling index; and the collapsibility.

At attempt was made by Al- Janabi 2002, to improve the behaviors of gypseous soil by remolding it with fuel oil and reed sticks. The sample was prepared at different moisture content with fuel content (0,3,5, and 7%) by weight. Test results indicate that the additional fuel oil decreases the peak strength and stiffness and increases the volumetric strain at failure for the treated remolded samples. The incorporation of the stickes with the gypseous soil and the fuel oil revealed an increase in the angle of internal friction by about 45^{o}. The cohesion showed a slight increase only.

2.6.1.3.4 Bitumen Materials Treatment

Shuber 1999, treating gypseous soils with two types of bitumen materials; (S-125 and R-250 cutbacks bitumen). Better engineering properties were observed with, S-125 cutback bitumen for stabilizing gypseous soil compared with (R-250 Cut-Back) bitumen.

Hassan 2000, examines the use of emulsified asphalt to improve two types of gypseous soils (44.6% gypsum content) brought from Al-

Ramady city, and (52.3% gypsum content) brought from Al-Therthar region. The test results showed that the increasing in binder content decreases liquid limit, plastic limit, specific gravity, maximum dry density, the coefficient of permeability, and the collapse potential. On the other hand, an increase in binder content increases the optimum fluid content.

Al-Morshedy 2001, carried out a laboratory models on two gypseous soils, (69.5% gypsum content) brought from Al-Therthar region and (34.66% gypsum content) brought from Faluja region. He recommended 7% by weight Cut-Back Mc-30) to be mixed with the soil. The collapse settlement was reduced by 50-60% for Al-Therthar and Al-Faluja soil using the spraying technique and by 55-64 % using the mixing technique.

Al-Alawee 2001, tends to examine the use of emulsified asphalt to improve the gypseous soil from Al-Therthar region (72% gypsum content). He recommended that 6% emulsified asphalt gives the best results, and the addition of asphalt to the soil increases in strength to an optimum value and after that it drops down.

2.6.1.4 Other Chemical Additives Treatment

Many chemical additives were carried out in improving engineering properties of gypseous soil and decreasing the effect of water percolation to ensure safety and stability of the structures.

Abood 1993, investigated the treatment of gypseous soil with sodium silicate on gypseous soils with different percentages of gypsum content. He found that the compression index (Cc) of untreated gypseous clayey soil decreased with increasing gypsum content while the rebound coefficient increased with increasing gypsum content. The collapse potential due to soaking and leaching increased with leaching which is attributed to the dissolution and washing out of gypsum.

Al-Abdullah et al 2000, treated gypseous soil brought from Al-Ramady (45% gypsum content) with (2.5, 5, 7, 10%) bentonite. The tests results showed that the addition of 2.5% bentonite gave a decrease in collapse potential (Cp), also the shear strength of soil increase when bentonite was added.

Al-Neami 2000, treated Al-Therthar gypseous soil (44.5% gypsum content) with (2, 4, 6, 8%) kaoline. He found that adding 6%)kaolin to the soil decreased the coefficient of permeability and the collapsibility and amount of leached gypsum.

Dubdub et al 2002, adopted a chemical treatment to improve the properties of gypseous soils utilizing of different percentage of dehydrated calcium chloride ($CaCl_2.2H_2O$). The tests results showed that the shear strength parameters of treated soil were unaffected upon leaching, the treatment of gypseous soils with ($CaCl_2.2H_2O$) gives reasonable reduction in permeability.

2.6.2 Physical Treatment

The physical treatment means that the soil properties improved without chemical additives such as; mechanical improvement and soil reinforcement, dynamic consolidation and soil replacement.

Seleam 1988, studied the shear strength parameters (C, ϕ) of a gypseous sandy soil, and concluded that (C, ϕ) were increased with the increase of the gypsum content due to the cementation action of the gypsum. The shear strength parameters were decreased upon wetting.

Al-Dilaimy 1989, reported that the max. dry density of gypsified clayey soil was increased up to 5% gypsum content, then decreased when the gypsum content exceeded 5%. While the moisture content had an opposite trend to that of the dry density.

Al-Heeti 1990, carried out laboratory tests regarding the compaction properties of gypsified clayey soil prepared by adding (5%, 10%, and 20%) natural gypsum content to the soil. The tests results showed that the maximum dry density increases and the optimum moisture content and decreased as the gypsum content increases.

Al-Ani and Seleam 1993, conducted some tests on compacted gypsified soils. The soil samples compacted at O.M.C are subjected to different amounts of water and left for 24 hours. Following that stored odeometer tests were performed. The results showed that the collapse potential decreased with the increase of initial water content used for wetting the soil.

Al-Khafaji 1997, derived empirical equations which enable to estimate the potential optimum moisture content (O.M.C) and max. dry density (γ_d) for modified proctor compaction from knoledge of the liquid limit (L.L.) and plastic limit (P.L.)

$$\gamma_d = 2.57 - 0.007(L.L. + 2P.L)\ \ kN/m^3$$

$$O.M.C. = 0.6 + 0.07(L.L. + 2P.L.)$$

This equation provides a simple and quick approach to control the field compaction of gypseous soil with 0.5 to 50 percent of gypsum content.

Chapter Three

3

Stone Columns in Gypseous Soils

Chapter Three
STONE COLUMNS IN SOILS

3.1 Introduction

Stone column technique is a well-known tool and widely spread through the world. It has been used successfully for the improvement of the engineering properties of saturated soft soils. Many events have been reported showing the advantages of this technique. During the last two decades many improvements about this technique regarding the construction equipments and materials have been made.

The development currently in process is related mostly towards improving the backfill material by using several additives. This technique has not ever been used with gypseous soils because the gypseous soils differ completely from soft saturated soil.

All techniques discussed in the pervious chapter shed the light on different proposals regarding the improvement of gypseous soils. None of these have discussed the use of traditional (stone only) or modified stone columns because this technique has not ever been used with gypseous soils.

This chapter will brief the main points regarding the stone columns and will point out to the main differences expected between soft saturated soil and gypseous soil.

3.2 Stone Columns Technique

Prior to the investigation regarding the improvement to be achieved stone columns, it is essential to get an idea about the expected modes of failure regarding the presence of stone columns.

Madhave 1982, reported the expected modes of failure of a footing resting on stone columns as shown in figure 3.1. The first mode is bulging, expected in case of long stone column with firm of floating support, figure 3.1a.

The general shear failure is expected for the case of short stone column with rigid base, figure 3.1b. While the third mode of failure was punching failure in the case of short floating columns, figure 3.1c. These modes of failure are basically related to soft saturated soils with low shear strength and high compressibility.

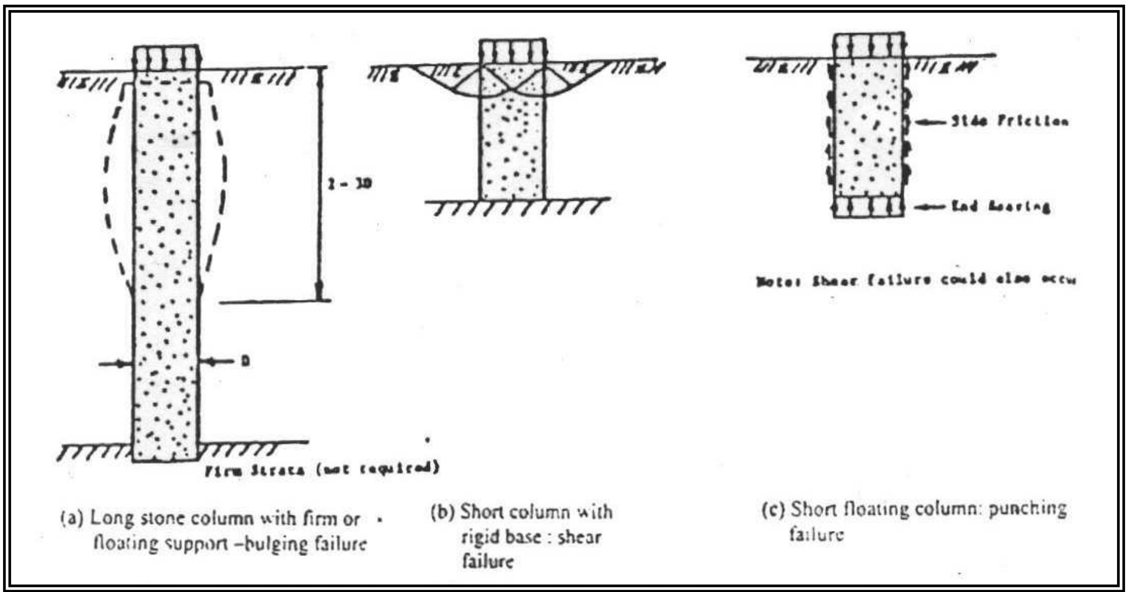


Figure (3.1) Mode of Failure for a Single Stone Column (after Madhave, 1982)

3.3 Basic Design Parameters in Stone Columns Technique

The major parameters related to stone columns are the geometry of the system including (length, diameter, arrangement and type of footing). The other parameters are related to the strength and compressibility of the surrounding soil and to the type of backfill material.

A thorough review of the parameters related to the geometry is summarized by Mahdi, 2002, tables 3.1, 3.2, and 3.3, and figures 3.2, and

3.3, showing that the optimum area replacement ratio (Ar) ranges between 0.2 to 0.3 and optimum length / diameter (Ls/Ds) ranges between 2 to 4.

The arrangement and spacing of stone columns have been investigated by Hussin (2000), showing that the recommended spacing is three times the diameter. This conclusion was also supported by Al-Qayssi (2001), who proposed an optimum spacing between the stone columns ranges from 2.5 to 3 times the diameter.

The parameters related to the surrounding soil and the backfill material are mainly the gradation of the backfill material and the (c_u), undrained shear strength of the surrounding soil. Regard the backfill material, Al-Shekhly (2000), found that the uniformly graded backfill material with average particle size ranging between 11-14 % of the pile diameter is the most recommended size, which provided higher bearing, improvement and higher settlement reduction ratio.

Table 3.1 Summary of laboratory tests (Mahdi, 2002)

Author	Ar	Ls/Ds	comment
Hughes and Withers (1975)	1	4,12	A series of model experiments were run using radiographic techniques to determine the actual behavior of the single stone column in soft clay.
Madhave et al (1979)	0.04, 0.06	0.5, 1	Model tests were performed for estimating bearing capacity and settlement of granular piles reinforced soft clay.
Chales and Watts (1981)	0.02, 0.12, 0.21, 0.25, 0.33	6	Large-scale laboratory tests have been carried out to assess the effectiveness of granular column in reducing the vertical compression of soft soil.
AL-Mosawe et al (1985)	0.178, 0.23, 0.28, 0.35, 1	4.75	A series of model tests were performed to determine the feasibility of using stone columns.
Jihad and Wayne (1986)	1	0.5, 1, 1.5, 2.5, 3.5, 4	Using model tests to investigate the effectiveness of highly angular sand columns.
Juran and Guermazi (1988)	0.04, 0.16		Performed a series of special modified triaxial tests on soil reinforced with stone columns.
Graig and AL-Kahafaji (1997)	0.1, 0.2, 0.3, 0.4	20	Four centrifuge model tests used to investigate the settlement of clay improvement by sand columns.
Rao et al (1997)	0.13, 0.15, 0.2, 0.5	2.5, 5, 10	Conducted a series tests on stone column installed in remolded soft clay with different I_c.
AL-Recaby (1999)	0.09	4.6	Using model tests with backfilling material furtherly improved by additives

Continued

AL-Shakally (2000)	0.88	2, 4, 6	Using model tests to study the effect of grain size of backfill material.
Abdul-Husain (2000)	0.12, 0.14, 0.19 , 0.21	4, 8, 10	Using model tests to study the influence of several parameters on the behavior of group of stone columns such as (Ar, Ls/Ds).
Zakaria (2001)	0.25	4, 6, 8	Performed model tests using several techniques to improve the behavior of granular columns.
AL-Qayssi (2001)	0.03	8	Conducted model tests to improve the behavior of stone columns by using different pattern of reinforcement.

Table 3.2 Summary of field tests (Mahdi, 2002)

Author	Ar	Ls/Ds	Comment
Hughes et al (1975)	0.22	3.5	The load settlement relationship for plate loading of isolated stone column in soft clay was predicated prior to filed tests.
Mckemne et al (1975)	0.09	12.5	Investigation the effectiveness of stone column in reducing the settlement of high embankments built on soft alluvium.
Goughnourand and Bayak (1979)	0.5	6	Field installation, instrumentation and performance of stone column group under a vertical area loading.
Balaam and Poulos (1978)	0.21	6.5	Using stone columns at Hedon site.
Broms (1987)	1 ,0.05	30	Using lime column to stabilized of soft clay in southeast Asia (Bangokok, Singapore, Jakarta.
Bergado et al (1987)	0.01, 0.06, 0.11	8	Full-scale embankment load test on fully penetrating granular piles on the soft Bangkok clay.
Enoki (1987)	0.3, 0.7	6	Stated that th value of n, in the practical design chosen between 3-4 for Ar<0.3 & 1 for Ar> 0.7, based on field data.
Buggy (1994)	0.21	12.8	Conducted a monitoring study on two 35m diameter oil tank Storage tank foundations.
AL-Recaby (1999)	1	6	The tests were carried out in AL-Rahman mosque site.
AL-Qayssi (2001)	0.09, 0.18	8	Performed seven field tests in Saddam mosque site.
Ahmed (1998)	0.1622, 1, 0.6491	8, 8, 4	The tests were carried out in Al-Nahtha area in Al-Rosafa – behind the Head Quarters of Al-Rashid Contracting Company.

Table 3.3 Summary of theoretical approach (Mahdi, 2002)

Author	Ar	Ls/Ds	comment
Balaam and Booker (1977)	0.01, 0.04, 0.25	1, 2, 4, 8	Using finite element method based on the concept of" unit cell" consisting of stone column and the surrounding soil within the column's zone influence.
Schweiger and Pande (1986)	0.1, 0.2, 0.3		Using finite element method with assuming that the influence of the column is uniformly and homogeneously distributed over the reinforced region and the total strain in surrounding soil and columns are same.
Bouassida et al (1995)	0.05, 0.3		A new design method for a foundation on a soil reinforced by columns is described. A lower bound of the bearing capacity is determined within the framework of the yield design.
Priebe (1995)	0.1to 1	0 to 23	Development an equation.
Sabhahil et al (1997)	0.15 to 0.5	8	Generalized method has been proposed to analyze the stability of embank-ment constructed over soft soil reinforced with granular soil.
AL-Mohammadi (1999)	1	4, 6, 8, 10	Using the finite element method (both linear and non- linear analysis were adopted).
Shahu et al (2000)	0.25	10	Analyses of soft clay with stiff crust overlain by granular mat and treated with granular piles. The model test was based on" unit cell" concept and equal strain condition.
AL-Saidi (2000)	0.1, 0.3, 0.7	5, 10, 15, 20, 25	Two finite element programs were used: Axi symmetric condition. Three- dimensional analysis.

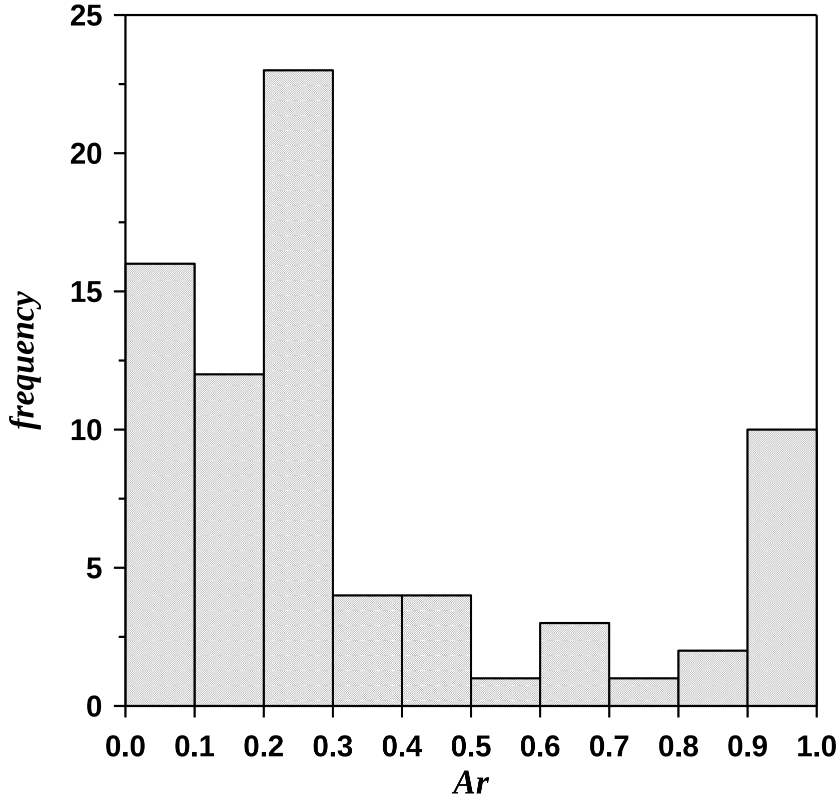


Fig. 3.2 Histogram distribution of area replacement ratio (Mahdi, 2002)

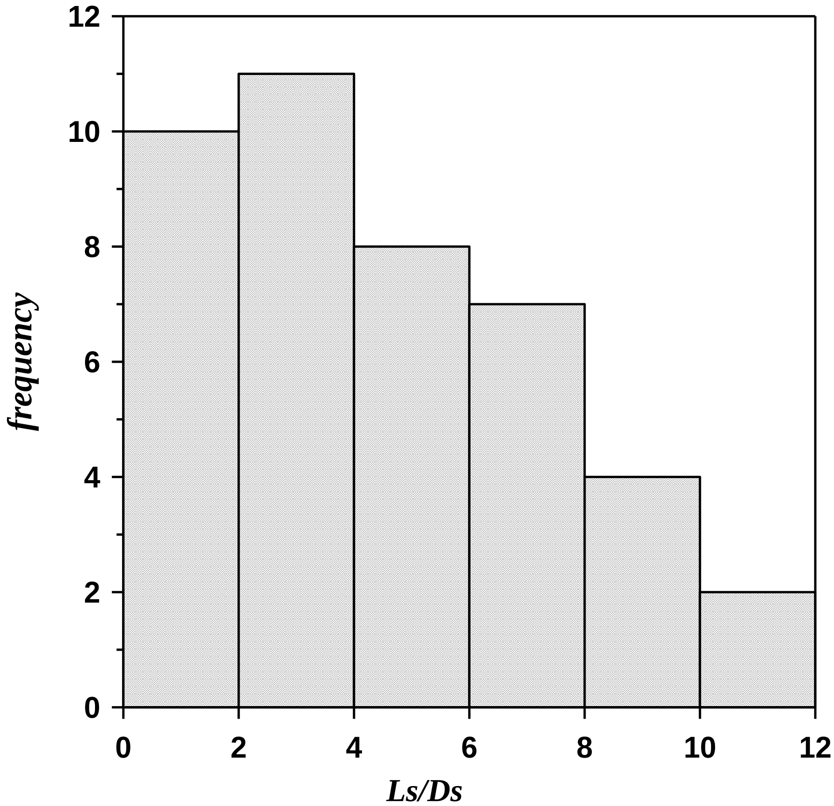


Fig 3.3 Histogram distribution of length to diameter ratio (Mahdi, 2002)

3.4 Construction Technique

In this section, the construction techniques are outlined with some remarks regarding the limitation of each one. It is worth to mention here that these techniques are proposed for soft saturated soils and not for stiff gypseous soils.

Table 3.4 summarized the construction techniques available for stone columns in soft saturated soils.

Table (3.4) Method of Construction of Stone Columns in Soft Clay

Author	Soil Description	(Ls/Ds) ratio	Construction method
Hughes et al (1975)	Silty Clay	(Ls/Ds)=18	Vibro-replacement
McKenna et al (1975)	Silty Clay	Ls=11.33m, Ds=0.9m (Ls/Ds)=12.6	Vibro-replacement
Rathgeb and Kutzner (1975)	- Fine to medium sand with gravels - Clay	- Ls=14m, Ds=0.47m, (Ls/Ds)=30 - Ls=6.5m, Ds=0.5m, (Ls/Ds)=13	Vibro-compction Vibro-replacement
Parson Brinkerhoff (1983)	Silty Clay	Ls=18m, Ds=1.11m (Ls/Ds)=16.2	Vibro-replacement
Buggy et al (1994)	Clay	Ls=13.7m, Ds=1.07m (Ls/Ds)=12.8	Vibro-replacement
Ahmed (1998)	Silty clay with sand and gravel	Ls=4m, Ds=0.5,1m (Ls/Ds)=8,4	Ramming technique
Al-Recaby (1999)	Silty sand underlain by layers of fat clay and silt	Ls=3m, Ds=0.5m (Ls/Ds)=6	Ramming technique
Al-Obaidy (2000)	- Fat clay underlain by layers of silty sand and sandy clay - Silty sand underlain by sand clay	Ls=6,9.44m, Ds=1.5m, (Ls/Ds)=4.6	Boring machine and ramming

Continued

Al-Saoudi et al (2001)	Clay	Ls=4.7m, Ds=0.55m (Ls/Ds)=8.5	Drilling and ramming technique
USA-New York (2002) *	Clay	Ls=4.5,6m, Ds=0.7,m, (Ls/Ds)=6	Vibro-replacement
USA-Maryland (2002) **	Silty clay	Ls=2.4,5.6m, Ds=1.1m, (Ls/Ds)=2,5	Vibro-replacement

* Stone Column Installation Specifications.

** Stone Columns Geotechnics America INC.

According to the construction techniques discussed in table 3.4, it is possible to propose Drilling and Ramming Technique using (Air Rotary Drill) for the construction of stone columns in gypseous soils.

This technique may be used for traditional stone columns as well as for modified stone columns.

Chapter Four

4

Experimental and Field Works

Chapter Four

EXPERIMENTAL AND FIELD WORKS

4.1 General

This chapter includes all the necessary tests performed to investigate the possibility of using the modified stone columns as a technique to improve the collapsibility of gypseous soil. Full-scale field tests were performed for this purpose.

4.2 The Site

The full-scale field tests were performed on a site near the Sodium Sulphate Factory in Al-Dour, located about 29 Km north east of Samara city in Salah-Aldeen Governorate. Figure 4.1 shows the layout of the site. After the first reconnaissance visit, six sample where taken and full chemical analysis was carried out as illustrated in table 4.1.

Based on these results it was decided to select the area exhibited the high gypsum content. This area was prepared for the field tests. Disturbed samples were collected from the top meter below natural ground level for gypsum content determination. This process was carried out by seribing the top 1m with the aid of a poclain and shovel. The soil samples were packed in bags, and transported to the soil mechanics laboratory in the University of Technology. Figure 4.2 shows the grain size distribution of the soil.

The collected soil consists of (12% gravel, 20% sand, and 68% silt). According to the Unified Soil Classification System, the soil was sandy silt of low plasticity classified as (ML).

Three samples were selected from the collected soil and the average gypsum content was found to be 67%. This gives the reason for selecting the site for full scale tests.

Detailed testing program is designed for complete investigation of the physical and chemical properties, in addition to the mineralogical composition of the soil. Figure 4.3 shows a flow chart for the conducted tests.

In addition to the general tests, soil profiles of the site were also determined and special tests before and after flooding were also conducted.

Tables 4.2 and 4.3 show the physical and chemical analysis, while table 4.4 shows the mineralogical composition of the soil.

Table (4.1) Chemical Properties for the Six Field Samples *

Sample No.	PH	CaO (%)	SO3 (%)	W.O.C. (%)	Gypsum Percentage (%)
1	8.3	25.48	20.23	13.58	64.99
2	8.2	25.2	32.37	14.26	69.59
3	9.0	28.7	32.77	14.57	70.45
4	7.7	25.3	35.48	15.85	76.28
5	8.2	26.18	36.04	16.2	77.48
6	7.5	25.9	35.55	16.66	76.43

* Tested by State Establishment of Geological Survey and Mining.

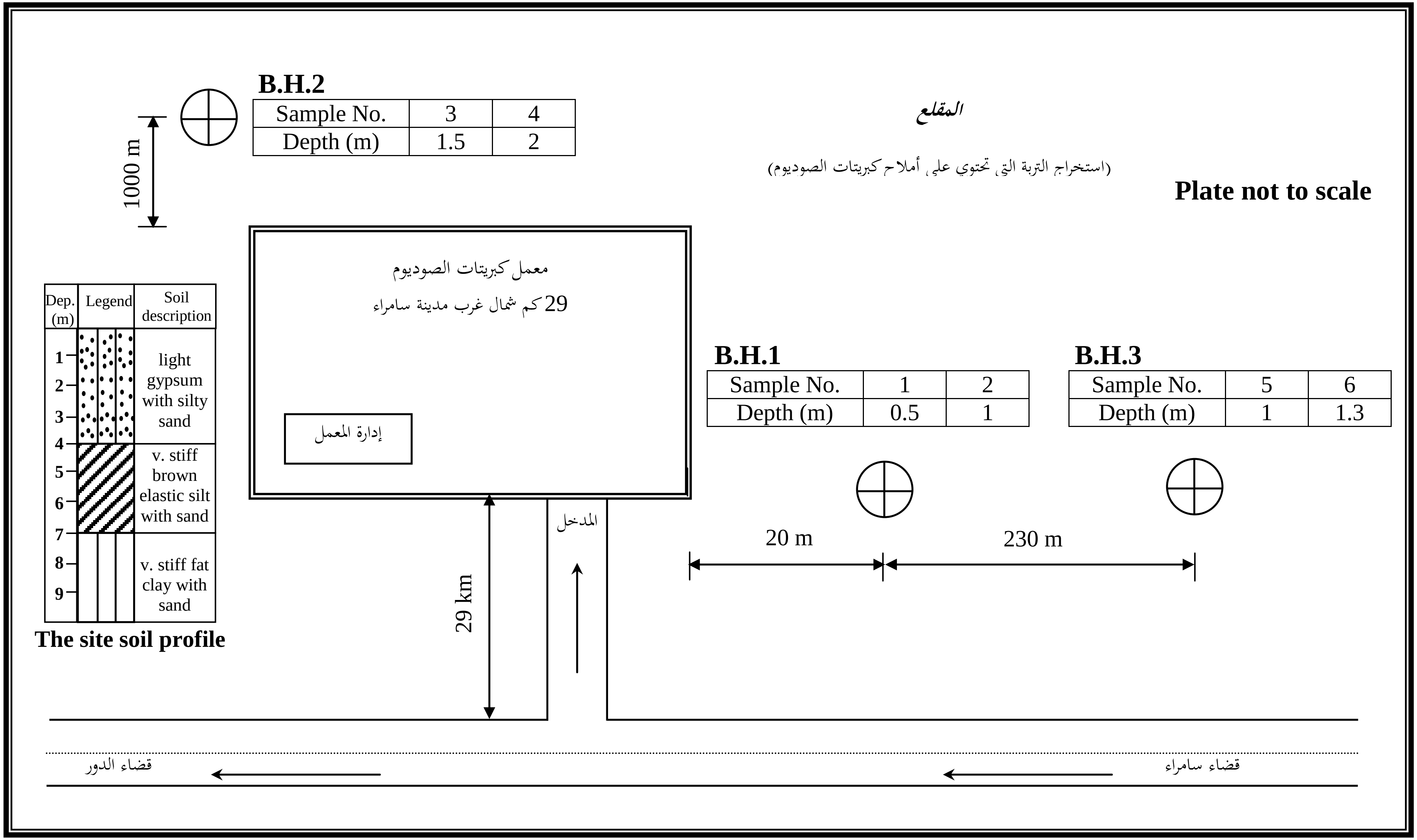


Figure (4.1) Location of the Field Work Site from the Sodium Sulphate Factory

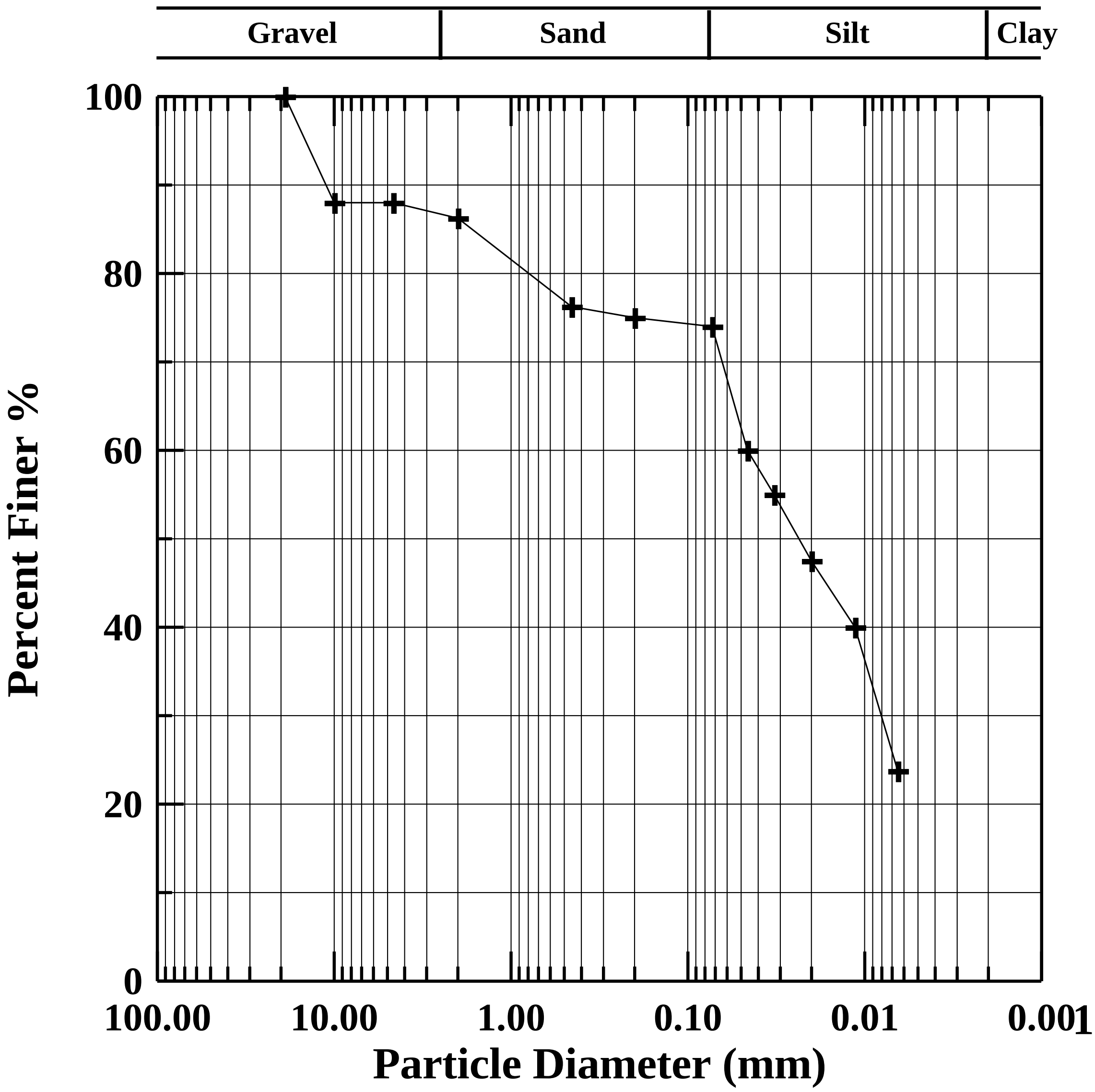


Figure (4.2) Grain Size Distribution of the Soil Used
(Tested by NCCL)

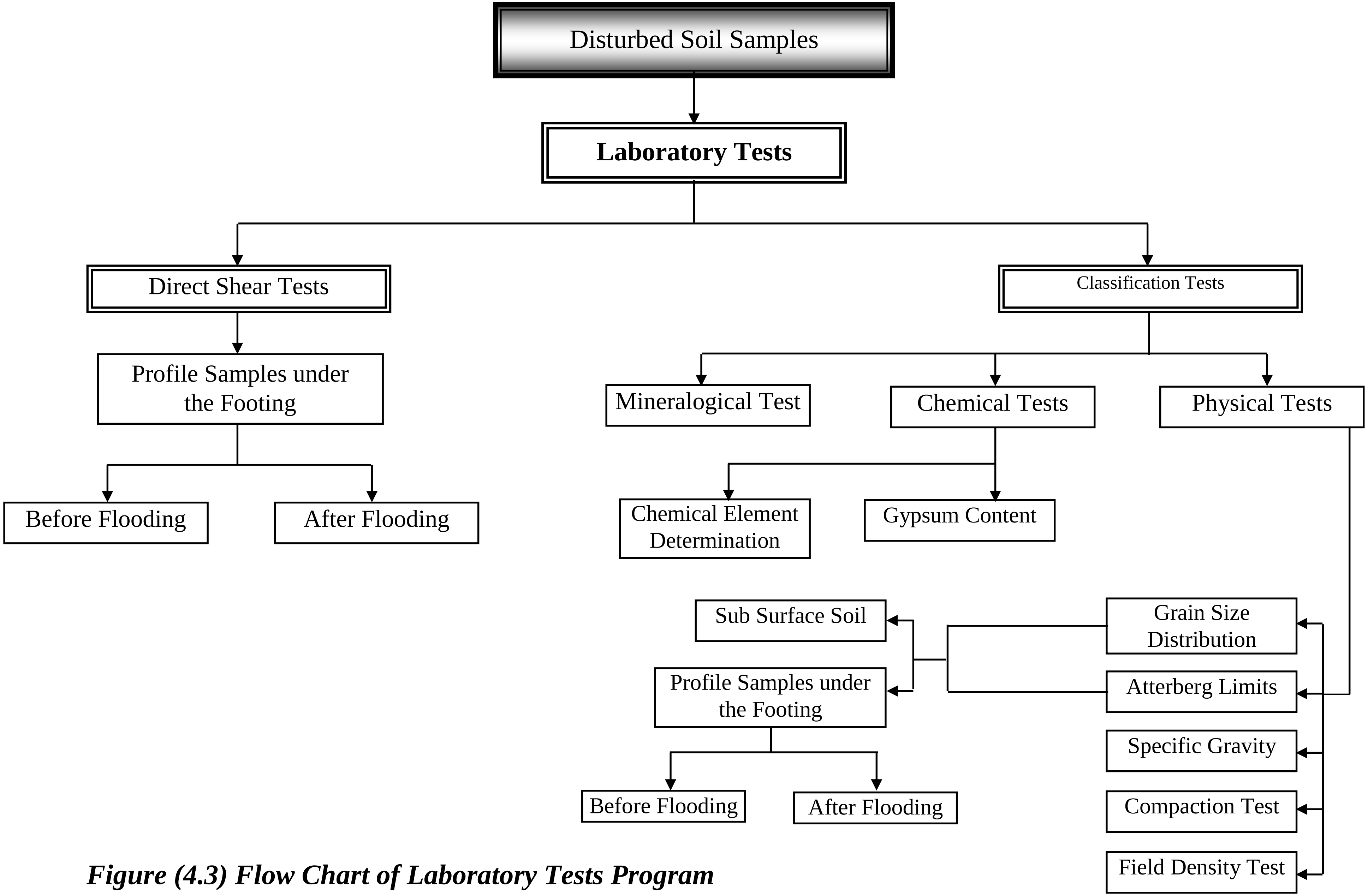


Figure (4.3) Flow Chart of Laboratory Tests Program

Table (4.2) Physical Properties of the Soil

Property	**Soil used**
Liquid Limit (L.L.) (%)	35
Plastic Limit (P.L.) (%)	32
Plasticity Index (P.I.)	3
Specific Gravity (Gs)	2.43
Maximum Dry Unit Weight (Standard) (kN/m^3)	15.17
Maximum Dry Unit Weight (Modified) (kN/m^3)	17.14
Field Unit Weight (kN/m^3)	12.76

Table (4.3) Some Chemical Properties of Subsurface Soil *

Property	**Soil Used**
pH	7.5
CaO (%)	25.9
SO_3 (%)	35.55
T.D.S (%)	62.75
W.O.C (%)	16.66

* Tested by State Establishment of Geological Survey and Mining.

Table (4.4) Mineralogical Composition of the Soil *

Soil Sample	**Description of Content**
Al-Dour Gypseous Soil	Gypsum, Trace Quartz, Very Trace Clay

* Tested by State Establishment of Geological Survey and Mining.

4.3 Field Tests

4.3.1 General

Four field tests were designed and conducted on the site. Two were performed on untreated ground and two on site treated with stone columns. Details of the tests are given below:

4.3.2 The Footing

Four footings square in shape with dimensions 1.25*1.25m, and 0.5m in thickness were constructed. Each footing was reinforced properly and provided with special anchor hooks for handling and transportation. Plate 4.1 shows the reinforcement of the footing.

Special attachments were casted in the footing for fixing the staff rods used for settlement measurements.

Plate (4.1) Reinforcement of the Footing

4.3.3 Additional Materials Used in Field Tests

4.3.3.1 Crushed Stone

Crushed stone materials were obtained from a private factory for ceramic and tiles in Al-Kamalia.

The crushed stone is of yellowish colour with angular shapes ranging in size between (16-32) mm. It is used as a backfill material.

Figure 4.4 shows the particle size distribution for the crushed stone. Some physical properties are also given in table 4.5.

Table (4.5) Some Physical Properties for Crushed Stone Used *

Property	**Stone Used**
Bulk Density (gm/cm^3)	2.64
Specific Gravity (Gs)	2.69
Uniform According to the Unified Soil Classification System	Uniform

***** State Establishment of Geological Survey and Mining performed the tests.

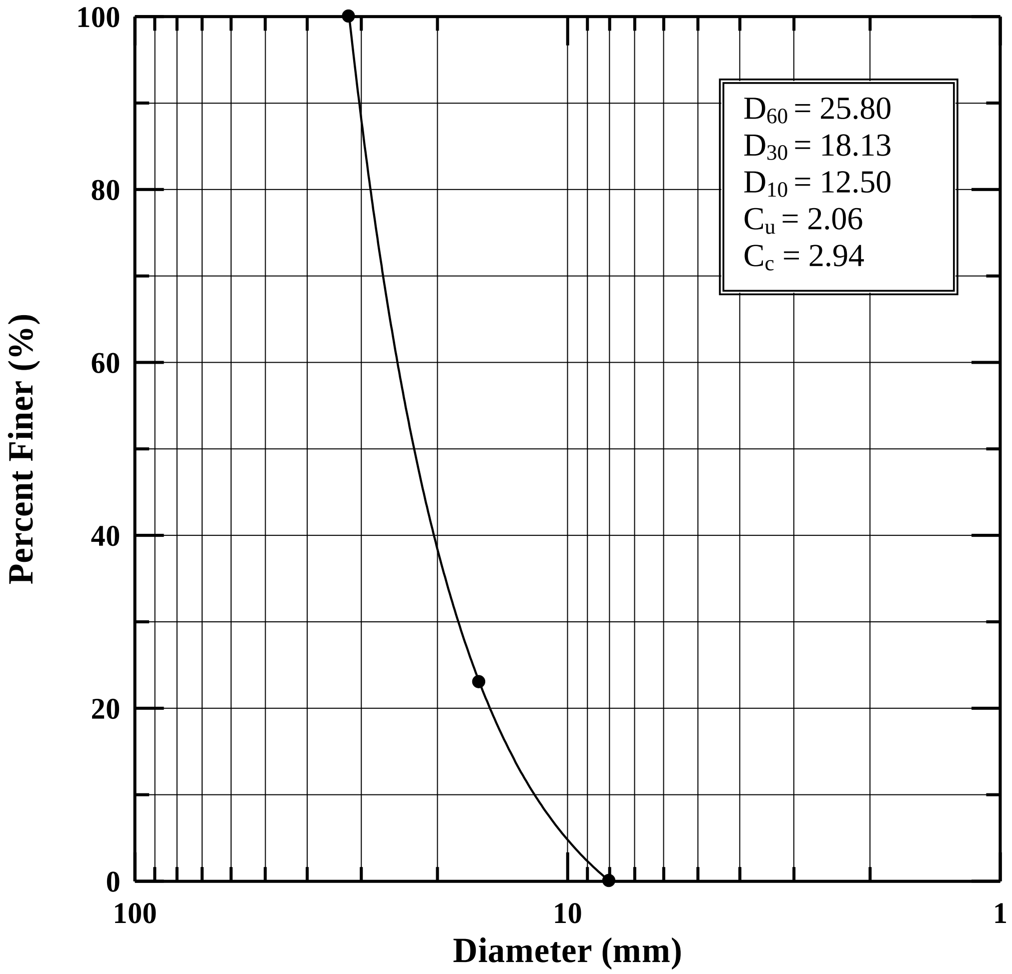


Figure (4.4) Particle Size Distribution for the Crushed Stone Used

(State Establishment of Geological Survey and Mining performed the test)

4.3.3.2 Lime

Quick lime used to be incorporated with the stone particles forming the stone columns. The quick lime was brought from Kerbala lime factory. Table 4.6 shows some properties of quick lime used which were supplied by Kerbala Lime Factory according to the Iraqi specification (807).

Table (4.6) Some Properties of Lime Used

Property	Lime used
Slacking time	(15-18) minute
CaO Activity	(85%) minimum
CO_2	(5%) maximum
MgO	(5%) maximum
Fe_2O_3,Al_2O_3	(5%) maximum

4.3.3.3 Liquid Asphalt

Medium curing (Mc-30) liquid asphalt was used as a binder in the construction of stone columns. This type of liquid asphalt was brought from Al-Daurah Refinery. This type of asphalt provides easy mixing with stone, and ultimately a homogenous mixing is obtained.

Table 4.7 shows the physical properties of the (Mc-30) medium curing asphalt.

4.3.3.4 Water Used

Since no tap water is available in the site and since the available water present in the nearby wells contains high concentration of salts as illustrated in table 4.8. So water from Al-Eshaky Canal located about 35km from the site was used.

Table (4.7) Physical Properties of the (Mc-30) Medium Curing Asphalt

Property	**(Mc-30) Used**
Viscosity Sct at (60)	30-60
Flash Point (coc)	38
Water (%) vol. (max.)	0.2
Distilled test to (360) Distilled (%) vol. Of total distilled: To 225 (max.) To 260 To 315 Residue from distillation to (360)(%) vol. (min.)	 25 40-70 75-93 50
Test on residue from distillation Penetration at 25 (100 gm,5 sec., 0.1 mm) Ductility at 25 (cm)(min.) Solubility in CCl4 (%)w(min.)	 120-250 100 99.5

Table (4.8) Chemical Analysis of Wells Water

Cations	P.P.M	MG.EQ / L	MG.EQ / L (%)	Anions	P.P.M	MG.EQ / L	MG.EQ / L (%)
Na^{++}	1073.64	46.68	50.38	Cl^{-}	1349	38	41.01
K^{+}	5.46	0.14	0.15	$So4^{--}$	2544	53	57.20
Ca^{++}	502.00	25.10	27.09	$Co3^{--}$	3.60	0.12	0.13
Mg^{++}	251.999	20.74	22.38	$Hco3^{-}$	93.94	1.54	1.66
	1833.09	92.66	100.00		3990.54	92.66	100
pH=7.1 , T.D.S at 220^{o}c(MG/L)=6122 Total Hardness (MG/EG/L)=45.84 Carbonate Hardness (MG/EG/L)=1.54 Non Carbonate Hardness (MG/L)=44.3 (Anions + Cations) - (1/2)Hco_3=5776.661							
Formula = $\dfrac{So_4=57.20\ \ Cl=41.01\ \ HCO_3=1.66\ \ CO_3=0.13}{Na=50.38\ \ Mg=22.38\ \ Ca=27.09\ \ K=0.15}$							

***** Tested by the laboratories of the Ministry of Irrigation

4.4 Preparation of the Site

4.4.1 Layout of the Site

The selected site was prepared for conducting the field tests. A depression of 6*15 m was excavated with 0.75-1m depth as shown in plate 4.2. After levelling the ground properly, the location of the four footings was marked. Following that the location of the stone columns was also marked. The distances between the footings are 3.25m center to center in both directions.

4.4.2 Preparation of Stone Columns

Two footings were placed on soil treated with stone columns, and four stone columns under each footing. The stone columns are 0.5m in depth and 0.7m center to center.

Four holes were prepared using boring machine with rock bit size 31cm, plate 4.3 illustrates the boring machine during operation. Plate 4.4 illustrates the four holes after the completion of the drilling process.

Following that, the required amount of crushed stone, lime and asphalt were prepared and mixed thoroughly to achieve homogeneous mix. This stage is shown in plate 4.5. The required amounts of the materials were selected from previous study carried out by (Al-Mosawy, R. H., 2001), 3.75% liquid asphalt and 7.5% lime by weight of the crushed stone.

The material is now ready for filling the holes. Small increments representing about $1/5^{th}$ of the volume of each hole was poured into the holes and compacted in five layers. Compaction was performed by a rammer about 10 kg in weight and base diameter 0.22m.

The stages of boring, filling, ramming, and the final completed stone columns are illustrate in figure 4.5.

After the complete preparation of the stone columns, the site was left for 30 days curing. This period is sufficient for the mixture to reach its final set. Plate 4.6 illustrates the stone columns during the curing period.

Plate (4.2) Selected Site after Preparation

Plate (4.3) The Boring Machine during Operation

Plate (4.4) The Four Holes after the Completion of Drilling Process

Plate (4.5) Material Mixing Stage

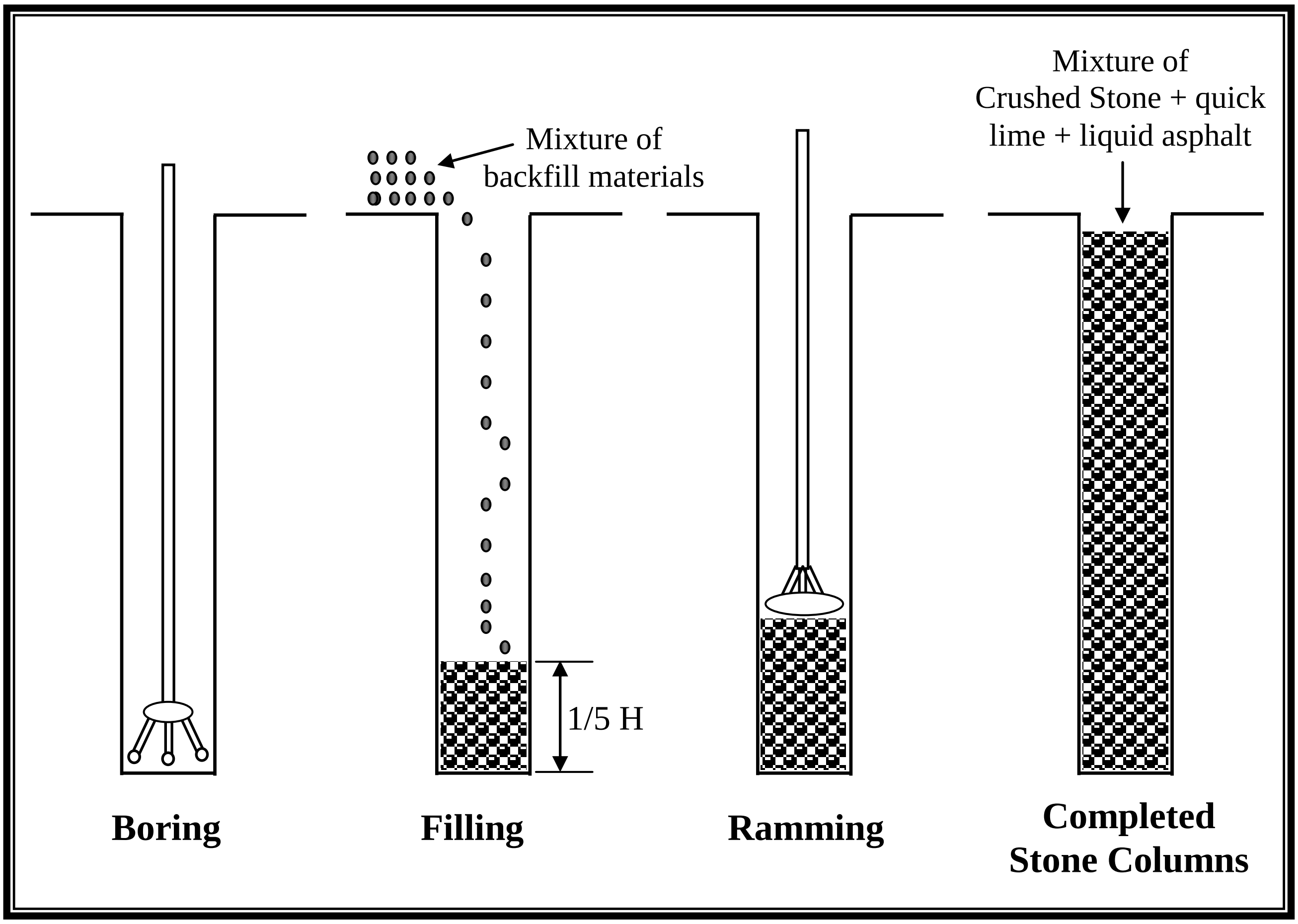


Figure (4.5) Boring, Backfilling and Ramming Technique

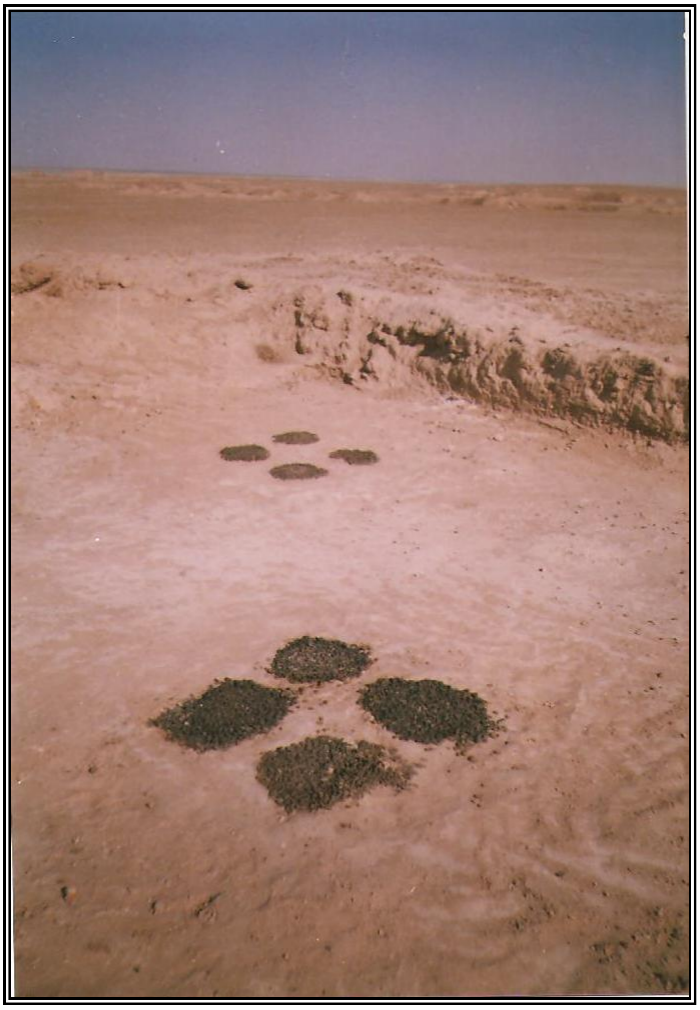


Plate (4.6) The Stone Columns during the Curing Period

4.4.3 Handling and Placing the Footing

At the end of the curing period, the four footings were lifted and placed in position .Two were placed on the untreated zone and two were placed on the treated zone with stone columns, representing an area replacement ratio of 19.32%.

Plate 4.7 shows a typical footing during handling and placing in position. Plate 4.8 shows vertical rods fixed in position for settlement measurements.

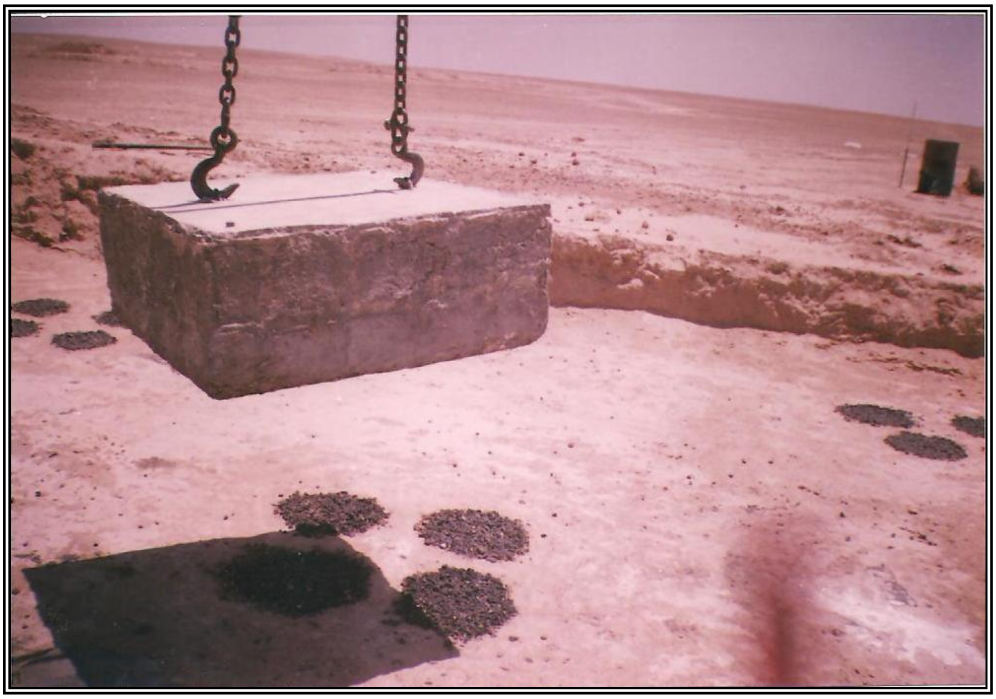


Plate (4.7) A Typical Footing during Handling and Place in Position

Plate (4.8) The Vertical Loads Fixed in Position

4.4.4 Application of Loads

Two load increments were applied on each footing, placed on untreated ground, corresponding to vertical stresses of 32 kPa and 44.8 kPa. The same stresses were applied on footings placed on treated ground.

The loads were applied by large concrete blocks each weights 2.4 ton. Plate 4.9 shows the footings under the action of the first stress increment of 16 kPa. The second load increment was applied, thus corresponding to a total applied stress of 32 kPa. Plate 4.10 illustrates the footing under the action of 32 kPa vertical stress. The applied stress was furtherly increased on two footings (one on treated ground and the other on untreated ground). The maximum applied stress was 44.8 kPa. Plate 4.11 illustrates this state of loading.

During all stages of loading before and during the flooding period, settlement measurements were recorded. The settlements were measured by a level with accuracy 1 mm. Plate 4.12 illustrates the reading during the time of testing.

Plate (4.9) The Footings under the Action of the First Stress Increment of 16 kPa

Plate (4.10) The Footings under the Action of 32 kPa Vertical Stress

Plate (4.11) The Footings under the Action of 44.8 kPa Maximum Applied Stress

Plate (4.12) The Reading during the Time of Testing

4.4.5 The Flooding Stage

After the maximum stresses were applied, the footings reached to an equilibrium state and no further settlements were observed. Following this stage, the flooding stage started. The whole depression was filled with water continuously about 20-30 m^3 daily. This procedure lasted for about 90 days with continuous monitoring of the settlement of the footings. Plate 4.13 illustrates the flooding process.

Plate (4.13) The Flooding Process

4.4.6 Supplementary Tests

At the end of the flooding period ,which lasted for 90 days, the site was left under natural condition. Inspection of the site was made twice a week.

After about one month, samples were extracted at different depths ranging from ground level to 1.5 m. Successive samples were taken every 250 mm. These depths comply with the depth of the stone columns constructed in the site.

Physical properties of the samples were determined in addition to the shear strength parameters. This stage was conducted before and after flooding for comparison purposes.

Chapter Five

5

Presentation and Discussion of Tests

Chapter Five

PRESENTATION AND DISCUSSION OF TESTS

5.1 Introduction

The major part of this work consists of four field tests, two on the untreated ground and two on ground treated with four stone columns.

Field-tests on untreated gypseous soils have been carried out by many researches, Mikheev et al 1977, Petrukhin and Boldyrev 1978, Petrukhin and Al-Perovich 1980. Plate load tests were carried out and revealed information about the behavior of gypseous soil in the dry and wet states. Low compressibility was observed in the dry state but increases gradually by the leaching process. The points drawn from these tests are limited and further field tests are required. The field tests should be properly designed and the output information must be carefully analyzed.

5.2 Field Tests on Untreated Ground

5.2.1 Field Test No.1 (32 kPa Applied Stress)

The field model footing was loaded by two blocks each weights 2.5 ton. When the first block was placed on top of the footing no settlement was observed during the following 24 hour. The second block was then applied giving a total stress of 32 kPa. Even at this stress, no settlement was noticed during the following 48 days. This is expected since the shear strength of the gypseous soil is extremely high compared to the applied stress. Also the gypseous soil are characterized by their very low compressibility when they are in the dry state.

The flooding of water started after two days of the maximum applied stress. 20-30m^3 of water was poured daily in the pit and the settlement was recorded with time. Figure 5.1 illustrate the variation of settlement with time during the 90 days. The figure consists of seven segments, each segment consists of a sharp increase in settlement followed by a rest period. This behavior occurred in spite of the amount of water was fairly constant every day.

The added water was sucked by the soil very rapidly during the first ten days. The continuous inspection of the site showed the development of small sinkhole around the footings which got bigger and bigger with time. Plates 5.1 and 5.2 show the developed sinkhole around the footings.

After 4 days of flooding the recorded settlement increased rapidly and reached 1.5mm, after wards the settlement continued but at a slower rate till the 12^{th} day. Following that a second sharp increase in settlement occurred and leveled off after 8 days. A rest period with very marginal change in settlement was observed and lasted for another eight days and settlement reached 3.6mm. Following this rest period a rapid increase occurred during the four days and the settlement reached 4mm. A gain a rest period was noticed and lasted for eight days. This behavior continued and a fourth sharp increase occurred where settlement reached 4.7mm followed by a rest period, which lasted sixteen days. Another increase in settlement occurred by this time gradually between 57 days and 66 days. Immediately after the 66 days a rapid increase took place during the following eight days and the settlement reached 6.4mm, a rest period for four days followed by a rapid increase where the settlement reached 7mm followed by a rest period till the end of the test. In spite of the regular daily supply of water, 20-30 m^3, it can be seen that the time-settlement relationship consists of seven steps each is basically consists of a sharp increase in settlement followed by a rest period. Consistent repetitive behaviors probably indicate that the destructive collapse of the gypseous soil is not a continuous process and that the development of the collapse is a step increase followed by a rest period.

Figure 5.2 shows the same points indicated by figure 5.1 and the rest periods.

A better evaluation of the influence of flooding water on the degradation of the skeleton of the gypseous soil can be observed in figure 5.3. The figure shows the variation of the $E / (1-\nu^2)$ versus time. As flooding continues, the footing experienced a gradual settlement with rest periods, indicating that the collapse is a gradual process and not sudden phenomena. The modulus of deformation of soil is calculated from the equation shown below:(Bowles, 1996)

$$S= q.B.(1- \nu^2)/E \qquad \ldots\ldots\ldots\ldots\ldots \; 5.1$$

With the following parameters taken as:

$q = 32$ kPa

$B = 1.25$ m

$\nu = 0.35$

Thus, the ratio $E / (1-\nu^2)$ is plotted against time in figure 5.3, the minimum value of E achieved after the 90 days flooding is 5000 kPa. This minimum value is compared with the value determined from the direct shear test at normal stress of 32 kPa performed after the 90 days of flooding. The E value is 1904 kN/m^2. This value is about half that determined from equation 5.1. There is more than a reason for this discrepancy since the two approaches are absolutely different. However the trend shown in figure 5.3 is quite representative for the gradual collapsibility taking place during the addition of water.

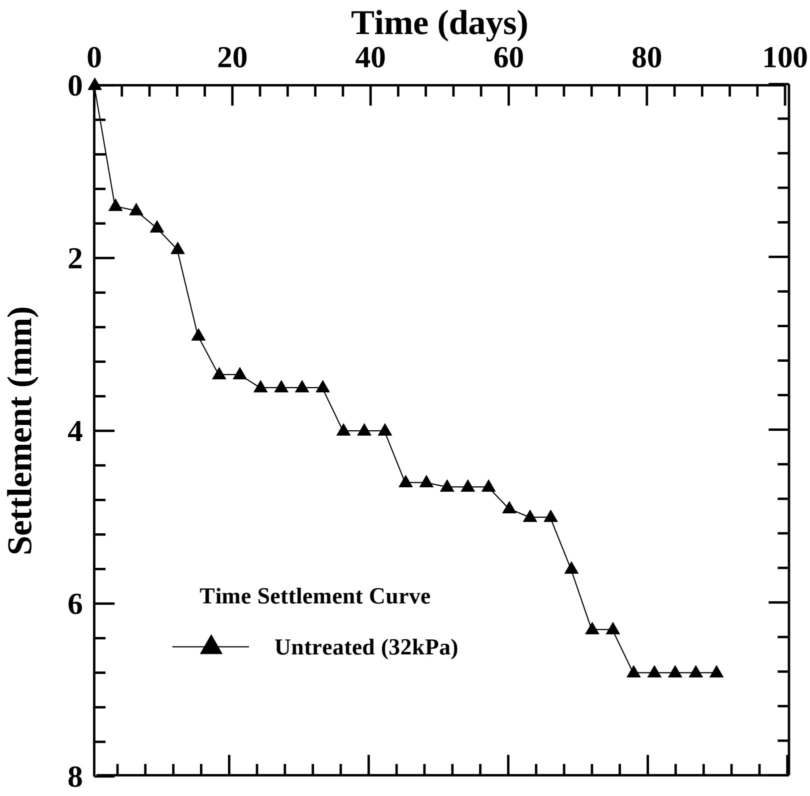


Figure (5.1) Settlement versus Time of Field Test No.1

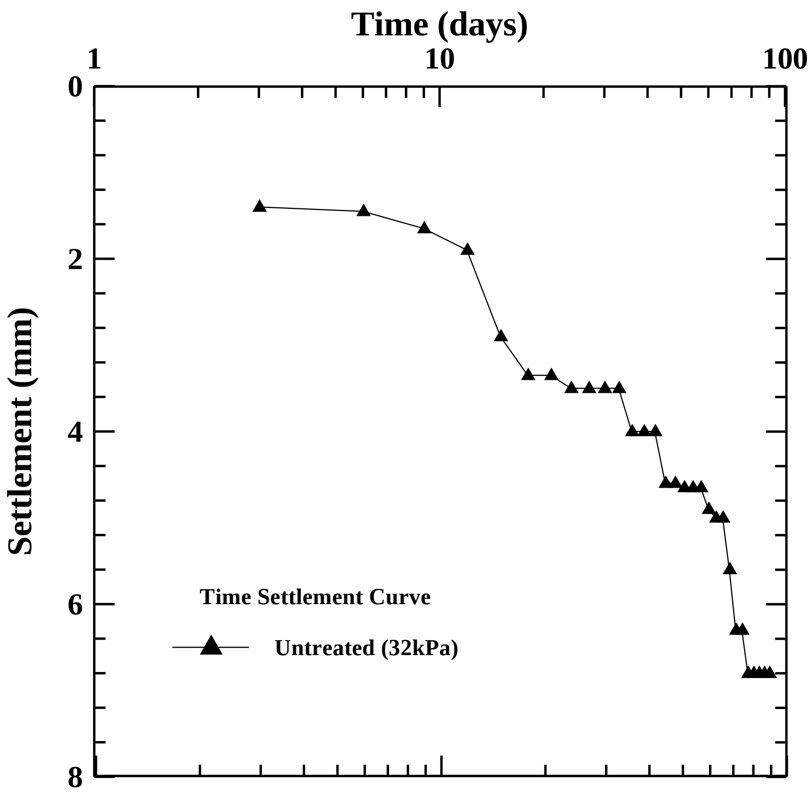


Figure (5.2) Settlement versus Time (Semi log Scale) of Field Test No.1

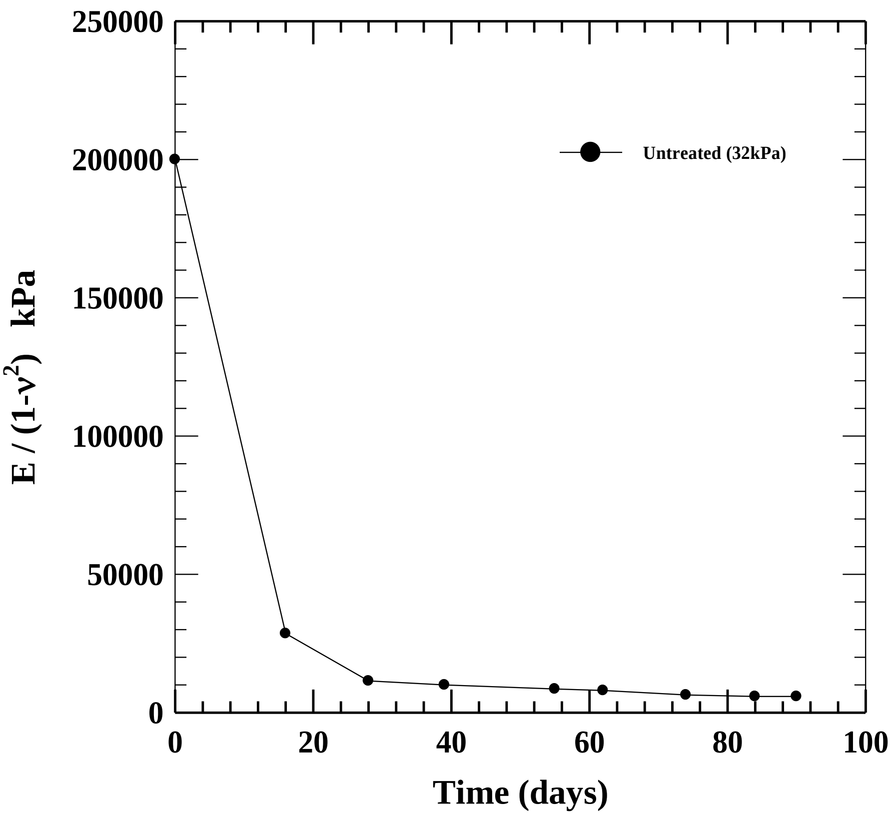


Figure (5.3) E / (1-ν²) versus Time of Field Test No.1

Plate (5.1) Sinkholes around the Footing

Plate (5.2) Sinkholes around the Footing

5.2.2 Field Test No.2 (44.8 kPa Applied Stress)

Similar to field-test No.1, the model footing was loaded incrementally up to 44.8 kPa. The observed settlement during loading was very marginal. Figures 5.4 and 5.5 illustrate the relationship between time and settlement for the footing performed on the untreated ground.

The addition of water followed the same pattern as that of test No.1, since the two footing are placed in the same pit.

The over all behavior of this model test is very similar to that of No.1 (applied stress 32 kPa). Again the time-settlement relationship consists of eight segments each showing a sharp increase followed by a rest period. The developed settlement after each rest period in each segment is higher than that generated by field test No.1.

Figure 5.6 shows the variation of the E / $(1-\nu^2)$ versus time. As soaking continues, the footing experienced a gradual settlement with rest periods, indicating that the collapse is a gradual process and not sudden phenomena.

The minimum value of E achieved after 90 days flooding is 4095 kPa. Again as compared with the E value obtained from the direct shear tests, 2067 kPa, it is found that this value is again about half that determined from the model test.

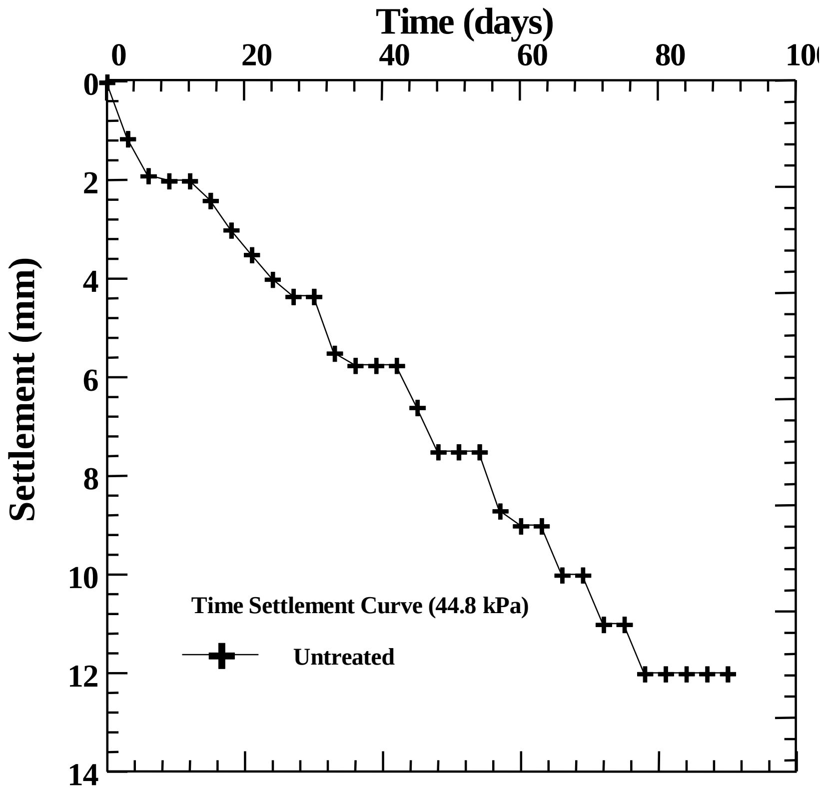


Figure (5.4) Settlement versus Time of Field Test No.2

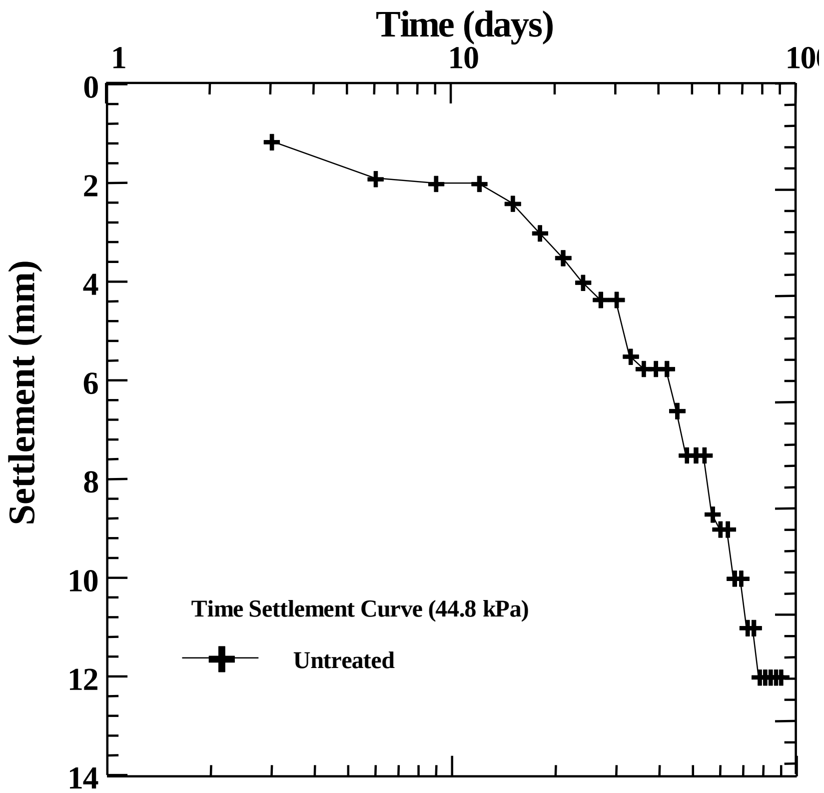


Figure (5.5) Settlement versus Time (Semi log Scale) of Field Test No.2

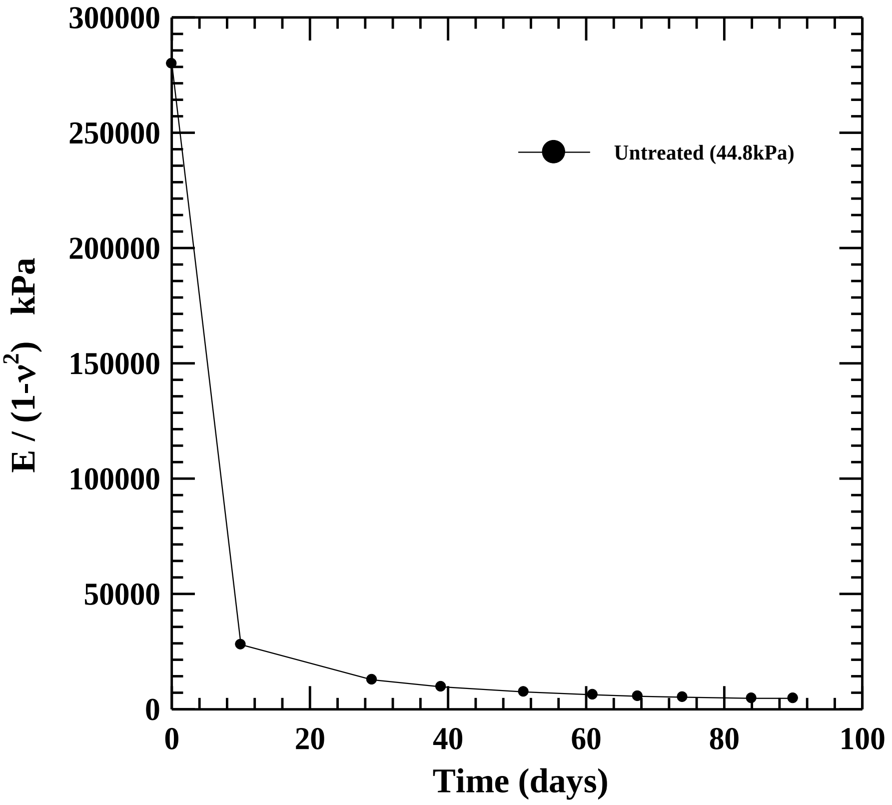


Figure (5.6) E / (1-ν^2) versus Time of Field Test No.2

5.2.3 Effect of Applied Stress on Time-Settlement relationship

Figures 5.7 and 5.8 show the time-settlement relationship for the two-model footing performed on the untreated ground.

The figure demonstrates that the observed settlement for the two tests was very close within the first twenty days of flooding. After that the settlement of test No.2 (P= 44.8 kPa) continued to develop at a higher rate compared with that of test No.1 (P= 32 kPa). After 40 days, the generated settlement were 5.7mm and 4mm for the 44.8 kPa models respectively. The continuous flooding was more effective on the footing with higher applied stress. The step-rest period continued to develop rapidly. After sixty days the observed settlement for the two footing reached 9 mm and 4.8 mm respectively.

The rest period in each segment is longer for the 32 kPa stressed footing indicating that the collapse settlement is occurs gradually and slowly at lower stress but increases rapidly at higher stress. The footing will continue to settle until the void ratio of the supporting soil reached its equilibrium value. The higher the stress, the higher in the collapse potential. Equilibrium State will achieve at lower void ratios.

A better explanation of the effect of applied stress is shown in figure 5.9 expressing the percent increase in settlement due to the applied stress with time. The curve consists of three segments the first is nearby a horizontal line expressing approximately no clear difference in settlement after flooding for twenty days. The second segment illustrates a sharp increase, which lasted for about 70 days. Following that the ratio leveled off at about 75% and remained constant.

This behavior may be explained in the following steps. In the first twenty days of flooding, both footings settled equally indicating that the added water did not reach to a state capable of collapsing irrespective to the amount of stress. The gradual collapse stated to be significant after continuous flooding and the supporting soil exhibited gradual loss in strength until an equilibrium state was achieved after about 70-75 days of flooding. The void at Equilibrium State for the supporting soil is lower for model footing with higher applied stress.

Figure 5.10 illustrates the $E / (1-\nu^2)$ versus time relationships for the two field model tests. The two model tests showed very close values after final degradation of the soil skeleton irrespective to the applied stress. The minimum value of E reached 5000 kPa for the untreated under 32 kPa applied stress, and 4095 kPa for the untreated under 44.8 kPa applied stress.

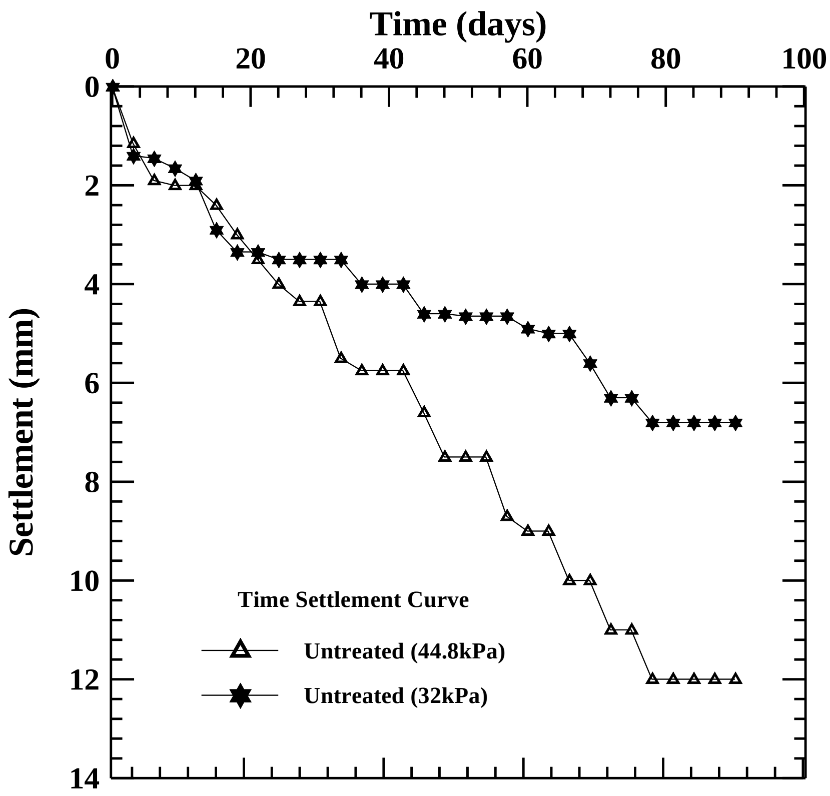


Figure (5.7) Settlement versus Time (Effect of Applied Stress)

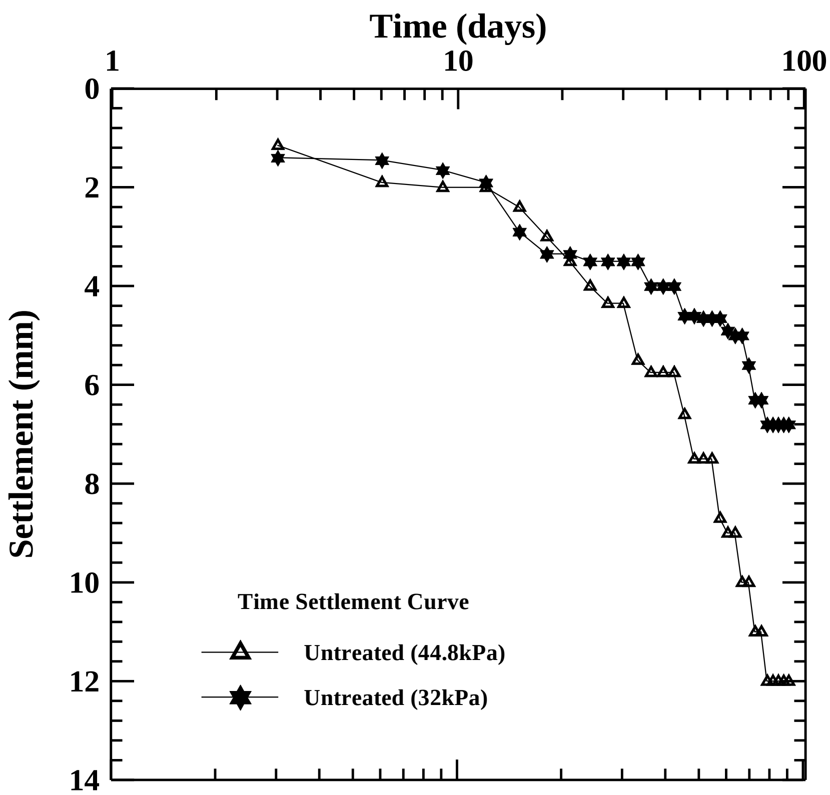


Figure (5.8) Settlement versus Time (Semi log Scale)
(Effect of Applied Stress)

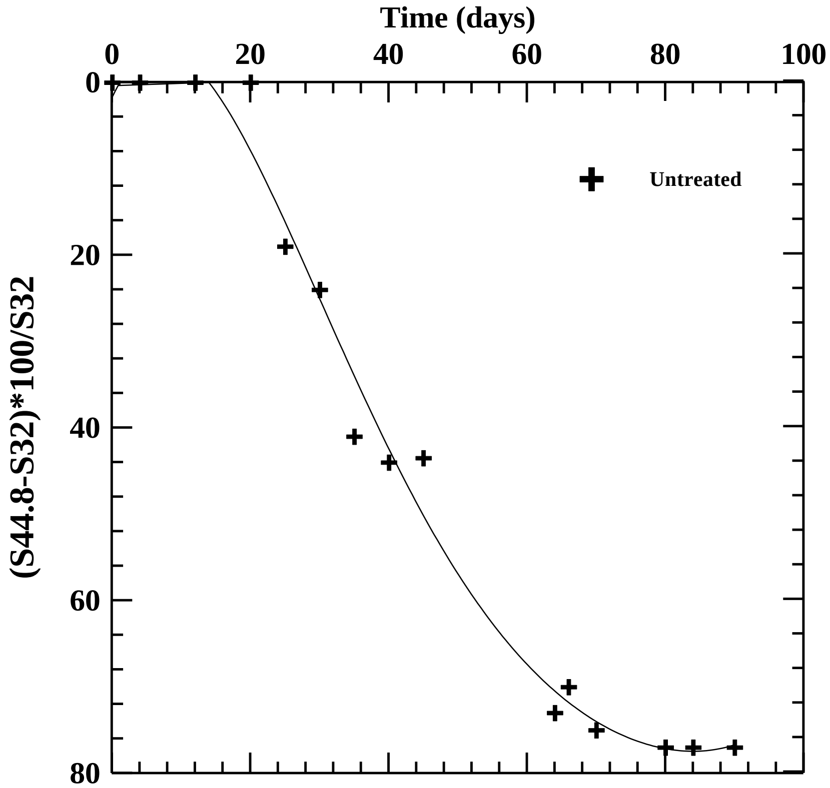


Figure (5.9) Settlement versus Time (Effect of Applied Stress)

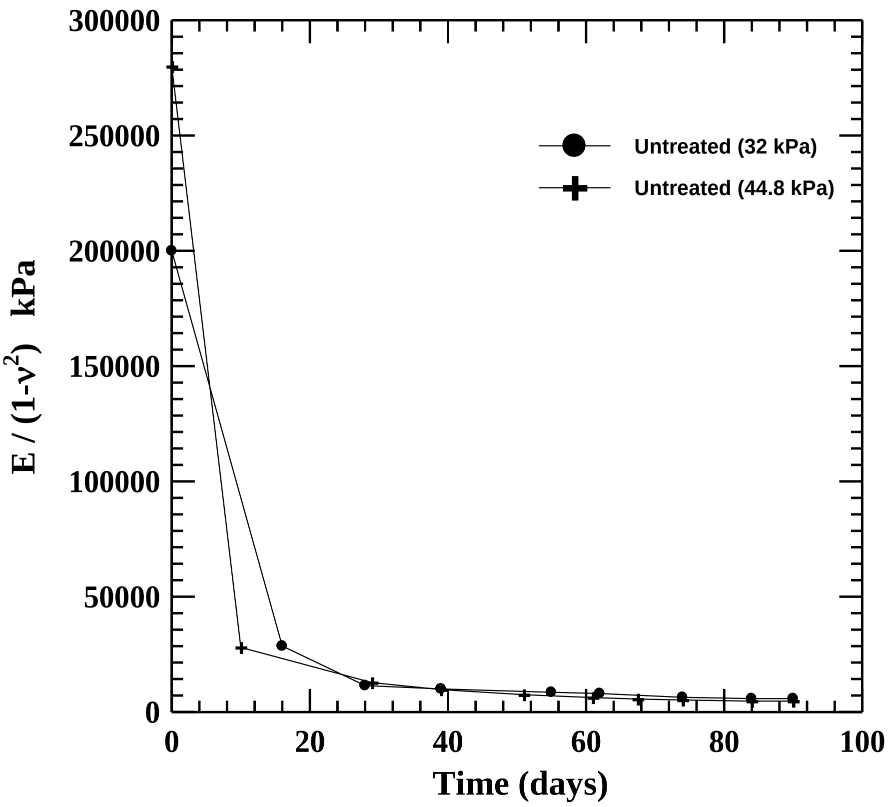


Figure (5.10) **E / (1-ν²)** ***versus Time***
(Effect of Applied Stress)

5.3 Variation of Some Properties of Untreated Gypseous Soil Due to Flooding

To investigate the influence of water on some physical properties of the gypseous soil and on the shear strength parameters, supplementary tests have been carried out.

5.3.1 Variation of Plasticity Index versus Depth

Figure 5.11 shows the variation of P.I. with depth for samples obtained before and after flooding. It can be seen that the P.I. is fairly constant with depth before the addition of water. After the flooding of the site for 90 days, samples corresponding to the same depths were extruded and the P.I. values obtained showed a significant drop at the level close to the N.G.L. As depth increases P.I. starts to increase again and exceeding that prior to flooding at depth 120cm. Such behavior may be explained in terms of the fact that flooding of water gradually suspended the fine particles. As it percolates through the voids, it tends to settle at lower depths giving and increase in clay fraction and hence and increase in P.I.

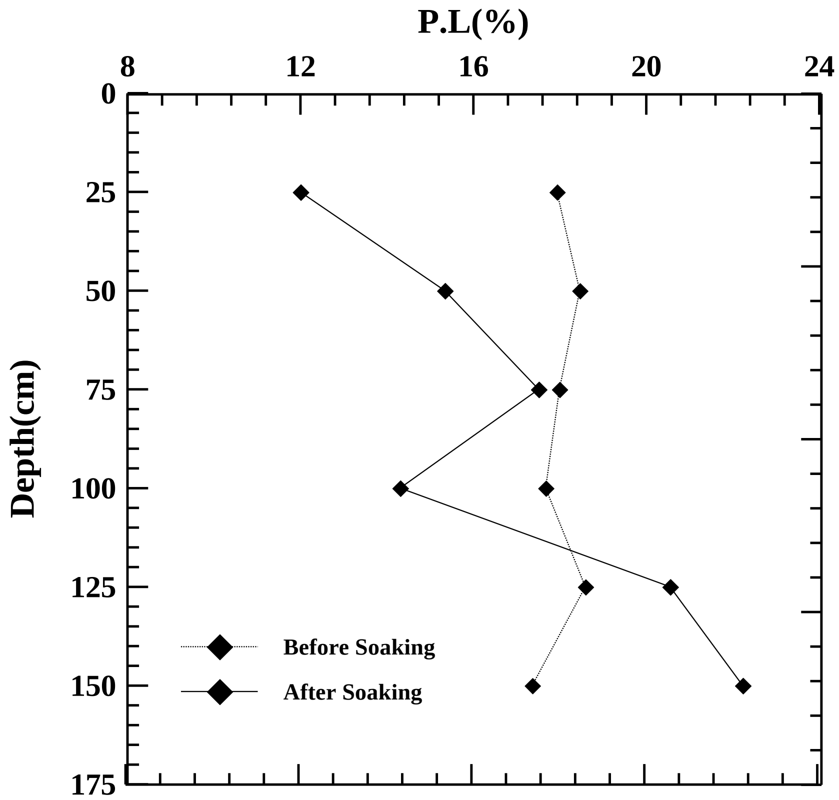


Figure (5.11) Variation of Plasticity Index with Depth

5.3.2 Variation of Gypsum Content (G.C.) versus Depth

The G.C. was determined for samples before and after flooding at selected depths below N.G.L. The G.C. was determined by two methods as indicated in chapter three. Figure 5.12 shows that the two methods exhibited decreasing in gypsum content ranging from 4% to 25% prior to flooding. After 90 days flooding, the two methods exhibited a decreasing trend with depth and the two methods provided close results. To clarify the effect of flooding on G.C., the average value of the two methods before and after flooding is presented with depth as shown in figure 5.13. This clearly shows a substantial decrease in G.C. at different levels below natural ground level. The amount of reduction in gypsum content due to flooding is shown in figure 5.14. The top 0.5m showed a reduction of 12.5% in gypsum content and this layer suffers also from the development of sinkholes. When the depth exceeds 0.5m, the percent reduction in gypsum content increases with increasing depth. Such behavior may be explained in terms of the fact that the flow velocity decreases with depth giving more change for the water to be in contact with the gypsum particles and hence resulting in to a great decrease in gypsum content.

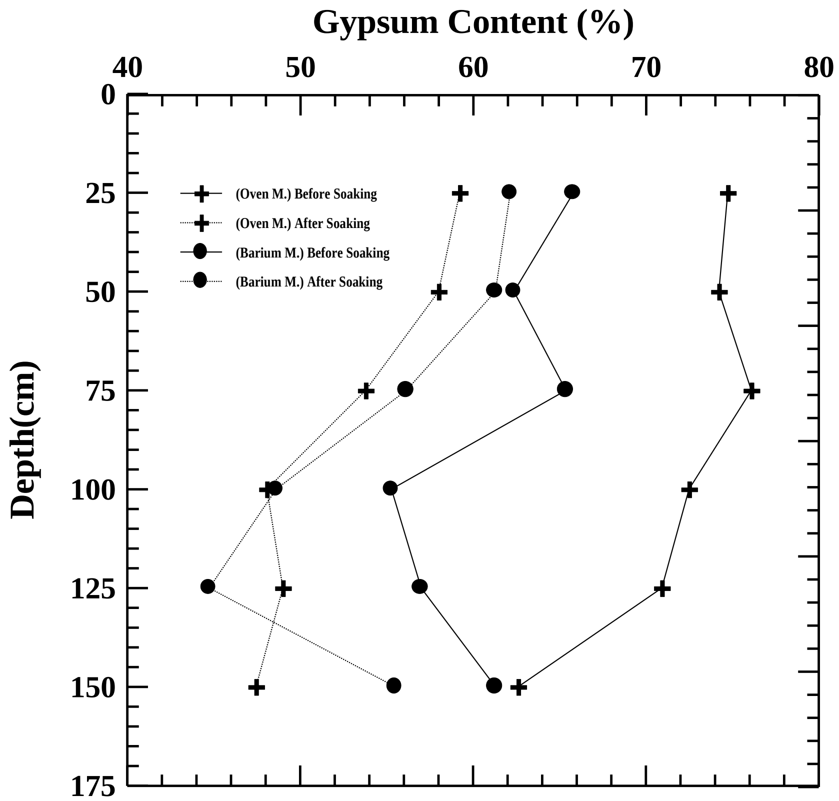


Figure (5.12) Gypsum Content versus Depth by Two Methods

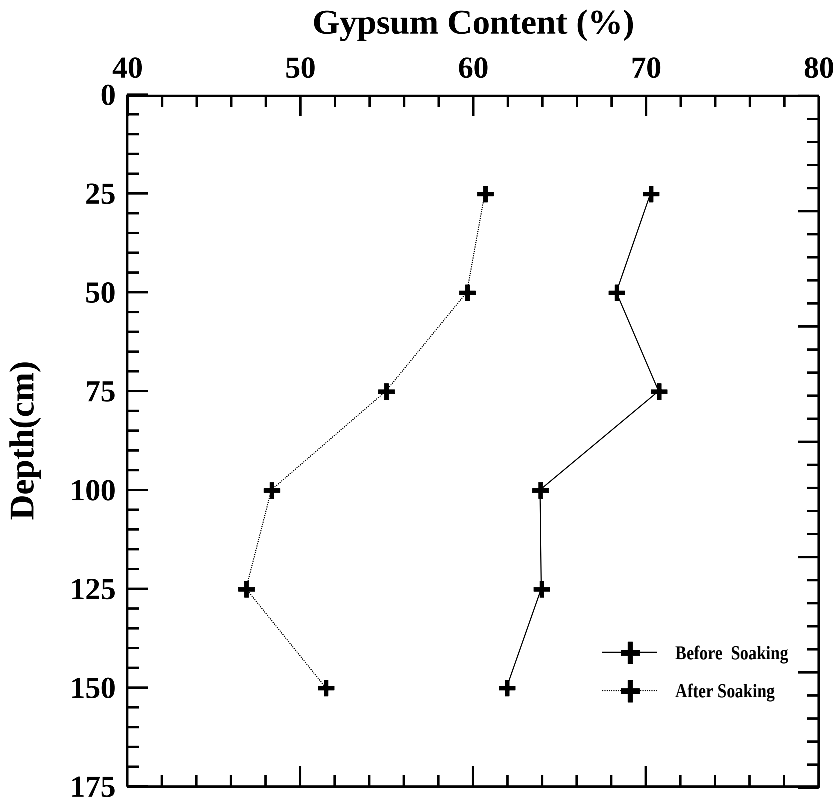


Figure (5.13) Average Gypsum Content versus Depth

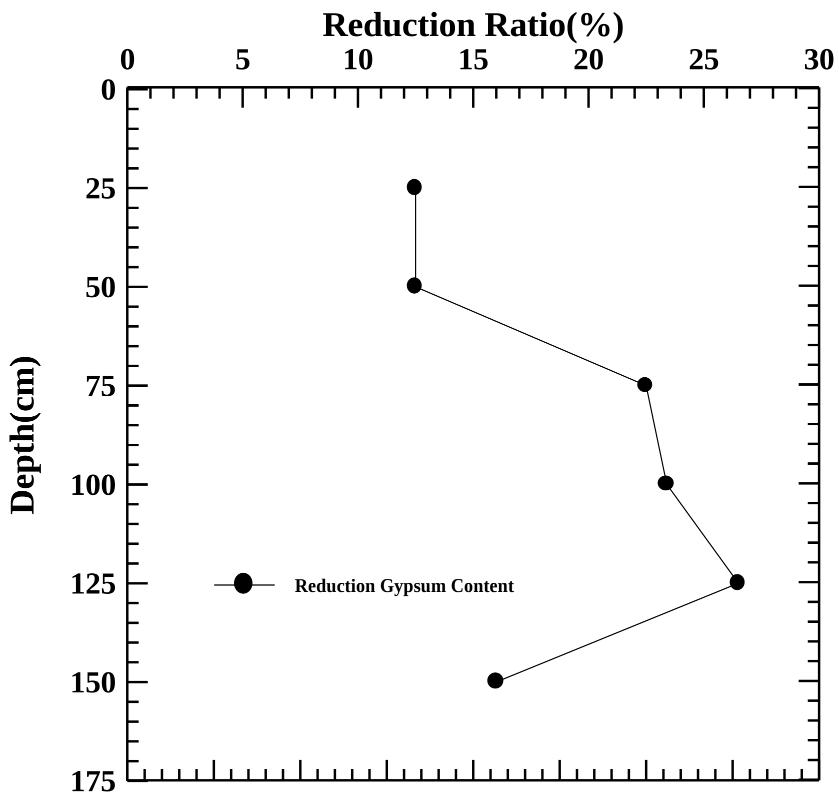


Figure (5.14) Reduction in Gypsum Content with Depth

5.3.3 Effect of Flooding on Shear Strength Parameters

Figures 5.15 and 5.16 show the variation of cohesion intercept and the angle of internal friction ϕ with depth before and after flooding. The cohesion intercept as shown in figure 5.15 exhibited an increasing trend with depth before flooding. After flooding, the general trend of cohesion with depth is a decreasing one. Washing down fine particles which settle at the bottom 1.5m, the average values are 33 kPa and 34 kPa before and after flooding indicating a marginal increase only. On the other hand, the angle of internal friction ϕ is fairly constant with depth prior to flooding, its average value is about 46-47. After flooding the ϕ decreased to about 35^o indicating a percent reduction of 24%.

Table 5.1 shows a summary of some typical shear strength values collected from the pervious literature. The results are rather contradicting and no clear-cut decision can be made about the influence of leaching.

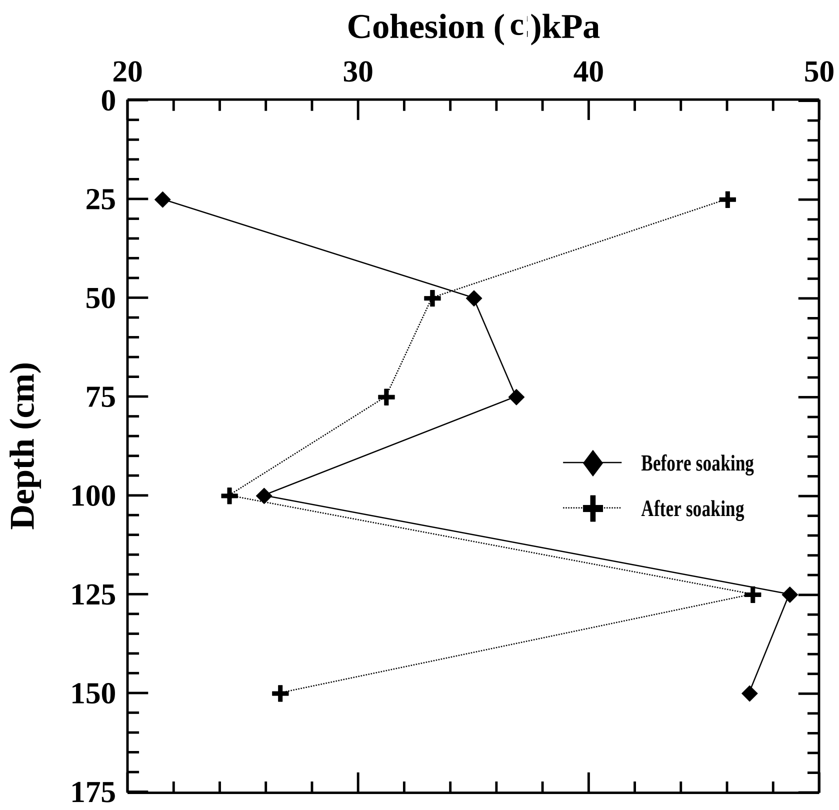


Figure (5.15) Variation of Cohesion with Depth

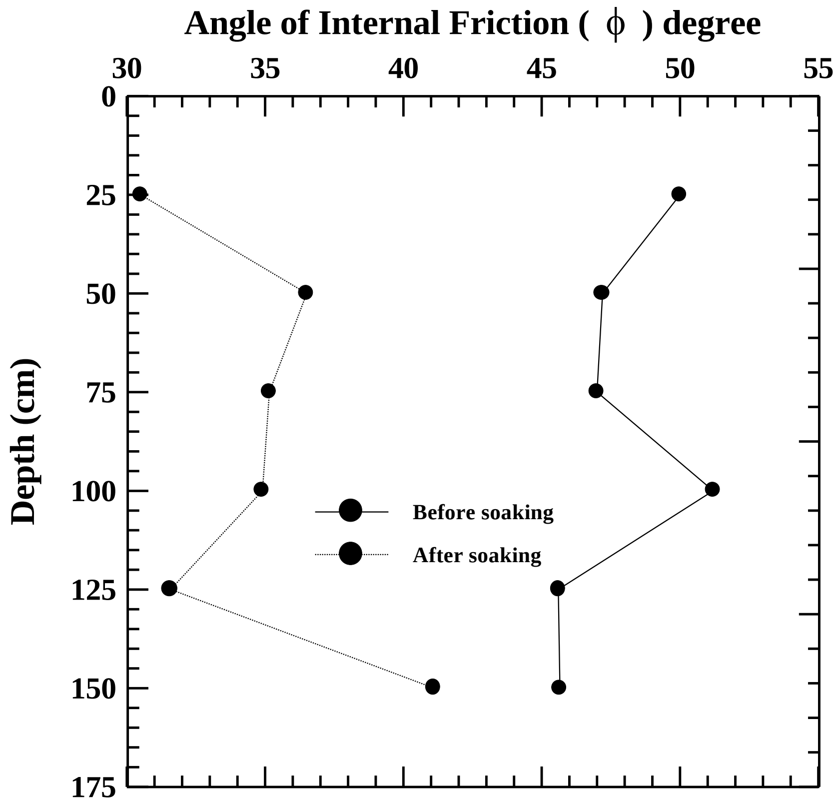


Figure (5.16) Angle of Internal Fiction Variation with Depth

Table (5.1) The Effect of Leaching on Shear Strength Parameters of Different Gypseous Soil (Nashat, 1990)

Soil Designation	Gypsum Content (%)	C (kPa)	ϕ (Deg.)	Type of Test	References
Natural soil	60	5.1	41.2	Triaxial Compression Test, c	Nashat 1990
Leached soil 1	48	zero	43.32		
Leached soil 2	30	9.11	37.08		
Natural soil	3	38	21.65	Triaxial Compression Test, c_u	Al-Layla and Taha 1985
Leached soil	0	20	21.5		
Natural soil	43	50	19		
Leached soil	20	10	18.8		
Natural soil	30.27	2	31.26	Direct Shear (CD)	Al- Khuzaie 1985
Leached soil	10	0	29.03		
Natural soil	41.33	10	30.11		
Leached soil	16	6	28.59		
Natural soil	27.6	140	40	Direct Shear (UU)	Seleam 1988
Leached soil	19.9	33.3	54.5		
Natural soil	20	99	37	-	Mikheev et al 1977
Leached soil	-	9	31		
Natural soil	8	110	30	-	Petrukhin and Arakelyan
Leached soil	0	44	26		
Natural soil	30	30	32		
Leached soil	0	5	30		

5.3.4 Effect of Flooding on Soil Texture

Figure 5.17 illustrates the percentage of each component (% gravel, % sand, % silt, and % clay) before and after flooding. It can be seen from the figure that the % gravel is not affected by flooding. The % sand is decreased from (60% average value before flooding) to about (30% average value after flooding), indicating that there was an appreciable amount of salts interlocked with the sand particles and when they are brought in contact with water, they tend to dissolve and only the solid grain were left.

The percent silts and clay show an appreciable increase due to flooding, this is related to the fact that fine particles are brought into suspension during the flooding process and they migrate between the pores and they settle down causing this increase after flooding.

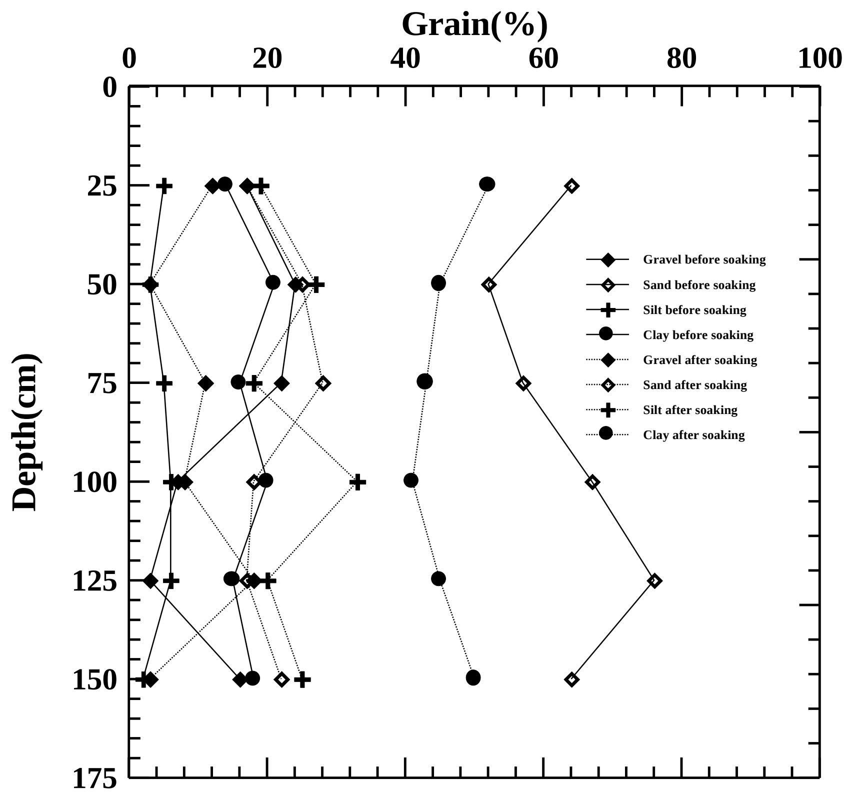


Figure (5.17) Grain Percent versus Depth

5.4 Field Tests on Treated Ground

5.4.1 Field Test No. 3 (32 kPa Applied Stress)

Figures 5.18 and 5.19 illustrate the settlement versus time for a field model test with soil underneath the footing treated with four stone columns under 32 kPa-applied stress. Similar to the untreated ground, the 32 kPa stress did not generate any measurable settlement. When flooding started at a rate of 20-30 m^3 of water per day (no measurable settlement was observed during the first three days), however its effect becomes sensible after six days.

The settlement reached 1mm after 10 days. In spite of the continuous flooding, a rest period occurred after the ten days and lasted for 9 days. Following that a sharp increase in settlement occurred between the 19 days and 22 days, where the settlement reached 1.8mm. Again a rest period was observed which lasted for twenty days (up to 42 days). A third increase was noticed between 42 and 46 days where the settlement reached 2.4mm, followed by a rest period which lasted for four days (from 46-50 day). A fourth increase in settlement following the 50 days lasted for four days where the settlement reached 3mm. A short rest period for four days came after, and a gradual settlement was observed, at 68 days. Again, a rest period, which lasted for about seven days, was noticed. A fifth increment in settlement occurred during the next four days and the settlement reached 4.2mm after 76 days and the footing remained at this settlement for four days. After words the settlement increased and reached 4.8mm. At this time, the footing remained constant and the rest period lasted for ten days, which is the time for the end of the test.

From the above discussion, it can be seen that the stone columns have succeeded in delaying the generated settlement and the accumulated settlement in a step increase followed by a rest period. The seven segments are clearly noticed in the figure.

A better evaluation of the influence of soaking water on the degradation of the skeleton of the treated gypseous soil by stone columns can be observed in figure 5.20. The figure shows the variation of the $E / (1-\nu^2)$ versus time computed at rest periods. As soaking continued, the footing experienced a gradual settlement with rest periods, indicating that the collapse is a gradual process and not a sudden phenomenon. The minimum value of E is 7312.5 kPa achieved after 90 days flooding.

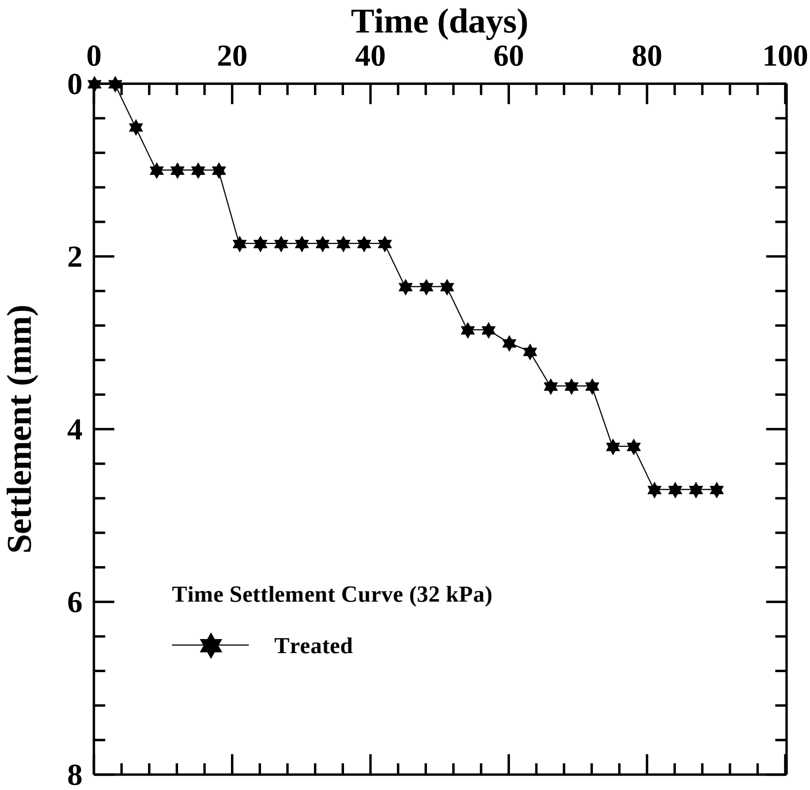


Figure (5.18) Settlement versus Time of Field Test No.3

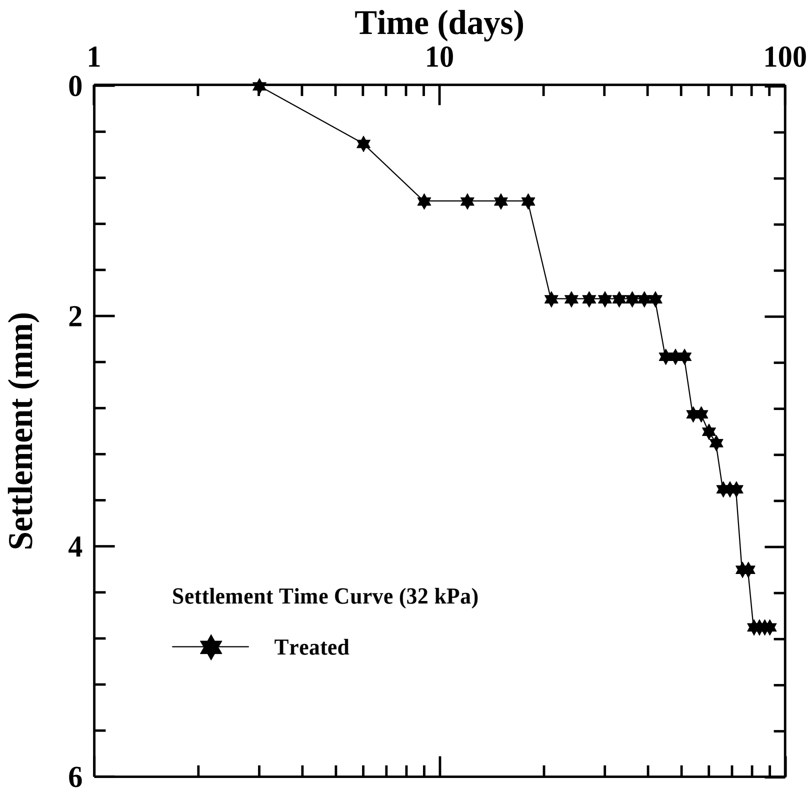


Figure (5.19) Settlement versus Time (Semi log Scale) of Field Test No.3

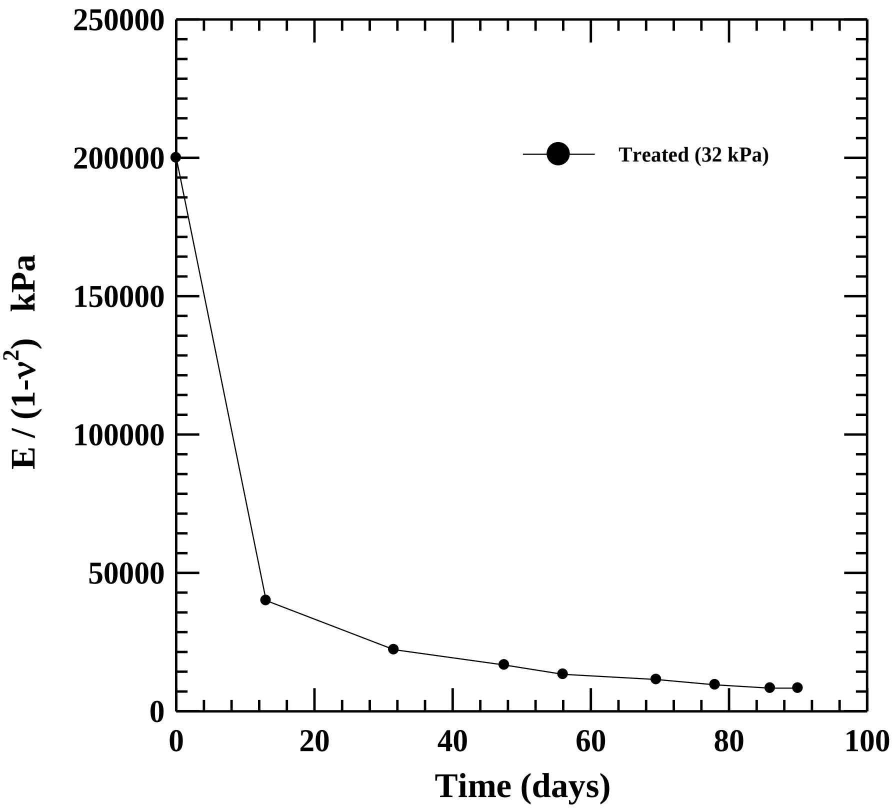


Figure (5.20) E / (1-ν^2) versus Time of Field Test No.3

5.4.2 Field Test No.4 (44.8 kPa Applied Stress)

Figure 5.21 and 5.22 illustrate the settlement versus time for a field model test with underneath soil treated with four stone columns under 44.8 kPa-applied stress. Similar to the pervious test at 32 kPa, the observed settlement during loading was very marginal at the addition of water followed the same pattern as that of field model test at 32 kPa.

The over all behavior of this model test is very close to that of No.3 (applied stress 32 kPa). Again the time-settlement relationship consists of eight segments each showing a sharp increase followed by a rest period. The developed settlement after each rest period in each segment is higher than that generated by field test No.3 (applied stress=32 kPa). When flooding started at a rate of 20-30 m^3 of water per day, no measurable settlement was observed in the first day, however its effect becomes sensible after four days.

The settlement continued and reached 1mm after 6 days. In spite of the continuos flooding. A rest period occurred after the ten days and lasted for 6 days. Following that a sharp increase in settlement occurred between the 12 days and 15 days, where the settlement reached 1.5 mm. Again a rest period was observed which lasted for four days (up to 19 days). A third increase was noticed between 19 days where the settlement reached 2 mm, followed by a rest period, which lasted for 17 days (from 22 days-39 days). A fourth increase in settlement following the 39 days lasted for three days where the settlement reached 2.7 mm. A short rest period for three days came after, and a little settlement was observed, 3.2 mm at 48 days. Again a short rest period which lasted for four days was noticed. A sixth increment in settlement occurred during the following eight days and the settlement reached 4.6 mm after 60 days. Again little settlement was observed between 69 days to 72 days. And the settlement reached 5.1 mm at 72 days. Again a sharp settlement was observed between 75 days and 81 days and the settlement reached 6 mm at 81 days, followed by a rest period lasted for 9 days (up to 90 days), The last day of flooding.

Figure 5.23 shows the variation of the E / (1-ν^2) versus time. The minimum value of E achieved after the 90 days flooding is 8328 kPa.

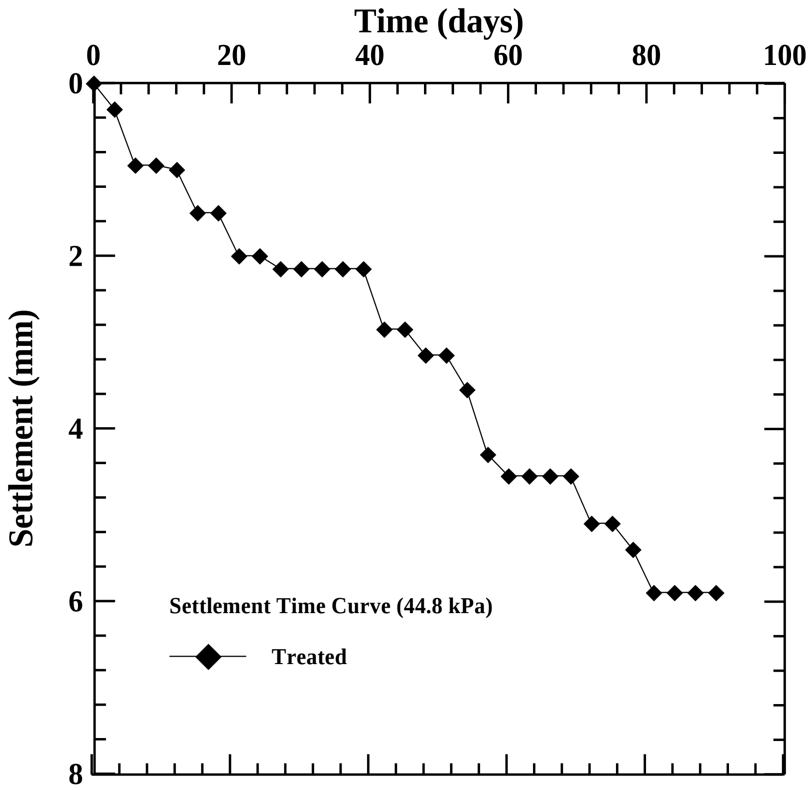


Figure (5.21) Settlement versus Time of Field Test No.4

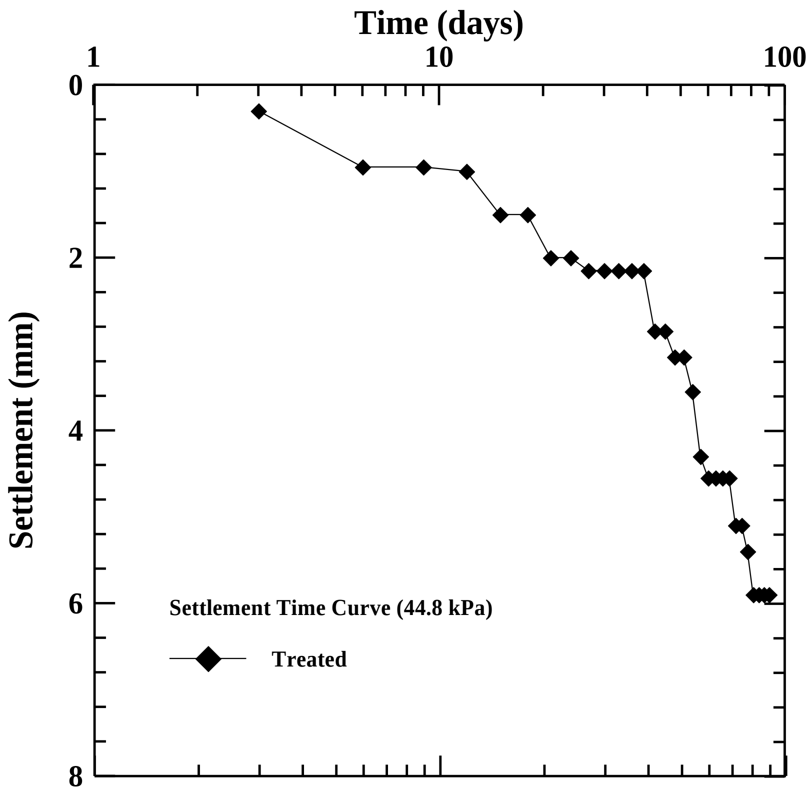


Figure (5.22) Settlement versus Time (Semi log Scale) of Field Test No.4

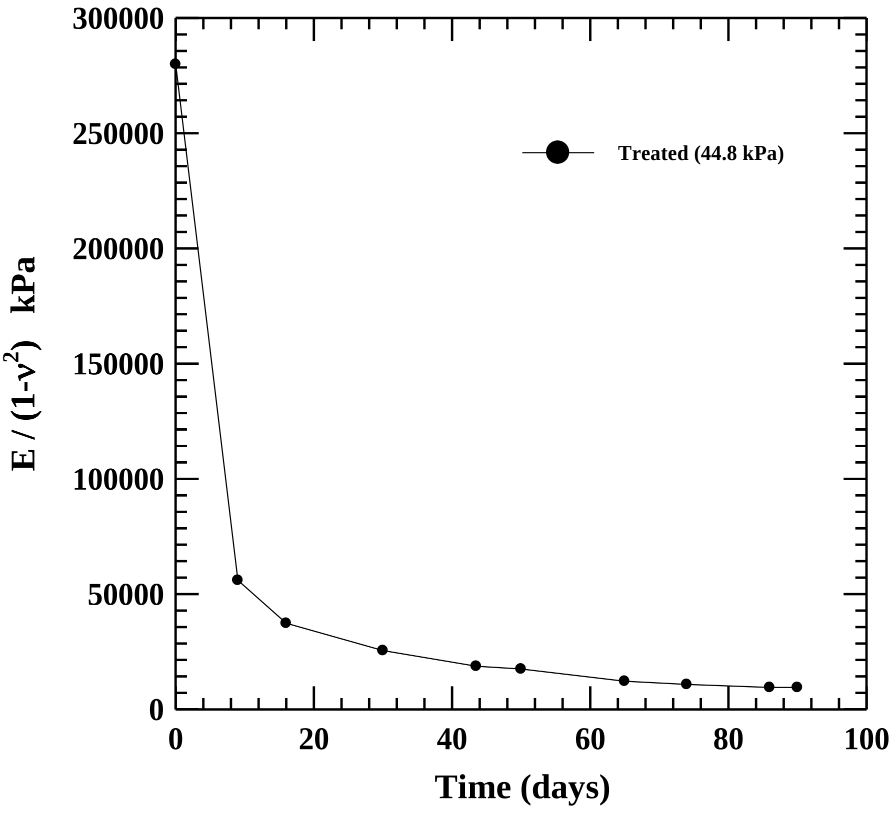


Figure (5.23) E / (1-ν^2) versus Time of Field Test No.4

5.4.3 Effect of Applied Stress on Time-Settlement Relationship

Figures 5.24 and 5.25 show the time-settlement relationship on normal and semi-logarithmic scale, for the two-model footing performed on treated ground.

The figures demonstrate that the observed settlement for the two tests were very close within the first twelve days of flooding. After twelve days the divergence in settlement-time relationship starts to be significant. There is a delayed compression in the footing of lower applied stress. The divergence become bigger and bigger with a continuous flooding. After about 60 days the difference in settlement is clear and reached its maximum value after 40 days, the generated settlements were 1.8 mm and 2.3 mm for the 32 kPa and 44.8 kPa models respectively. The continuous flooding was more effective on the footing with higher applied stress. The step-rest period continued to develop after sixty days, the observed settlement for the two footing reached 2.8 mm and 4.6 mm for the 32 kPa and 44.8 kPa respectively.

The rest period in each segment is longer for the 32 kPa stressed footing indicating that the collapse settlement occurred gradually and slowly at lower stress but increases rapidly at higher stress. After 80 days of flooding the settlement reached 4.6 mm and 5.9 mm for the 32 kPa and 44.8 kPa respectively. The footing will continue to settle until the void ratio of the supporting soil reached its equilibrium value. The higher stress, the higher in the collapse potential. Equilibrium State will achieve at lower void ratios.

A better explanation of the effect of applied stress is shown in figure 5.26, expressing the percent increase in settlement due to the applied stress with time. The curve consists of three segments, the first is a fair horizontal line expressing approximately no clear difference in settlement after flooding for twelve days. The second segment illustrates a sharp increase, which lasted for about 50 days.

Figure 5.27 illustrates the E / (1-ν^2) versus time relationship for the two-field model test. The two model footings were differing at its E (modulus of deformation) in the beginning of flooding stage. The modulus of deformation were 245000 kPa and 175000 kPa for the two treated ground under applied stress 44.8 kPa and 32 kPa respectively. After eight days of flooding, the two models have the same E. E magnitude was reduced with continuous flooding till each model reached equilibrium state. The model under 44.8 kPa-applied stress reaches its minimum value at 8328 kPa, as well as the model under 32 kPa applied stress reach its minimum value at 7312.5 kPa.

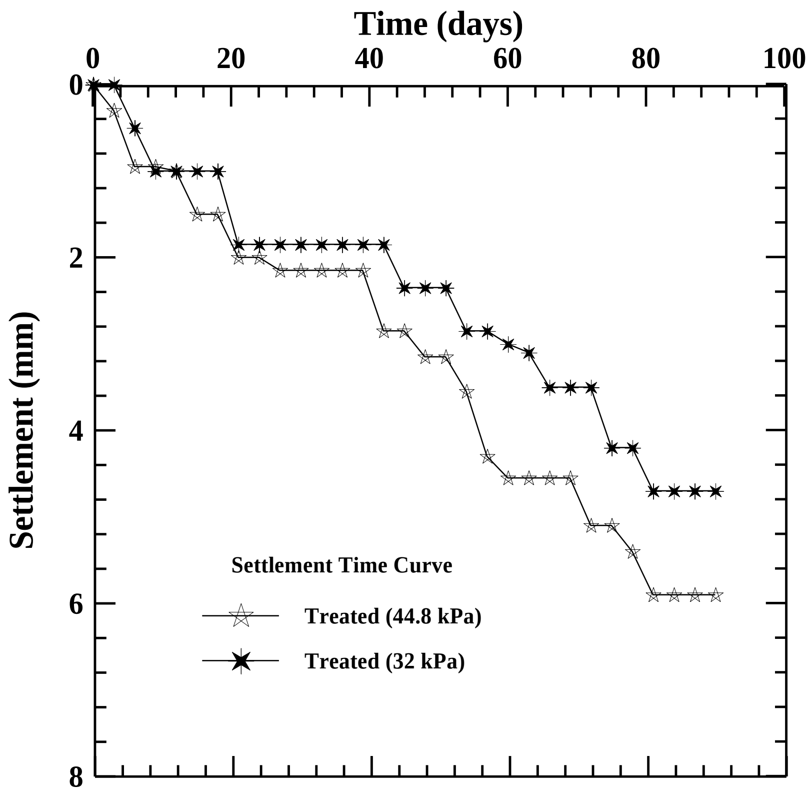


Figure (5.24) Settlement versus Time (Effect of Applied Stress)

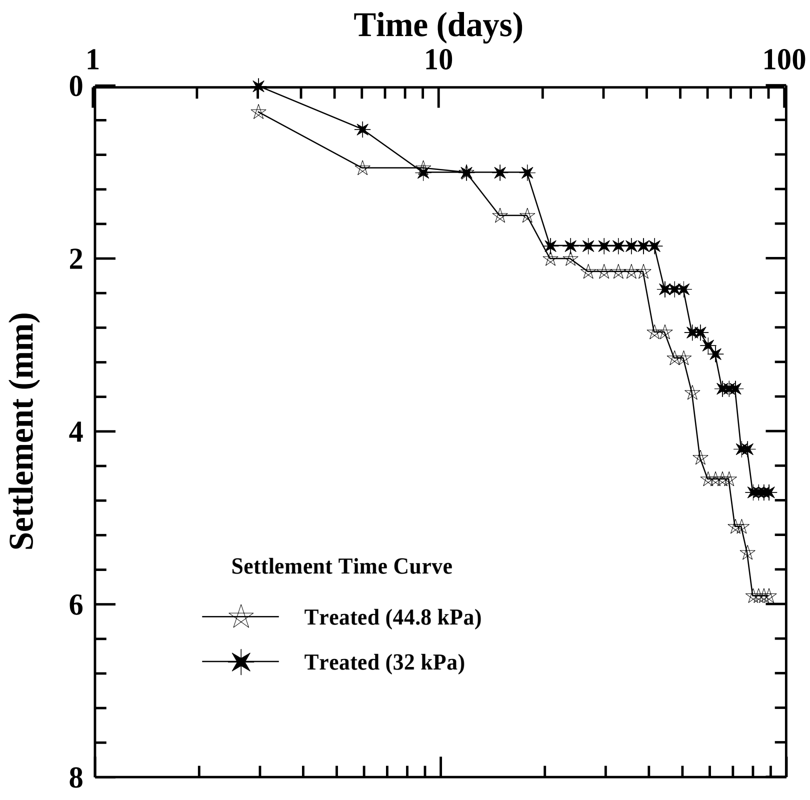


Figure (5.25) Settlement versus Time (Semi log Scale) (Effect of Applied Stress)

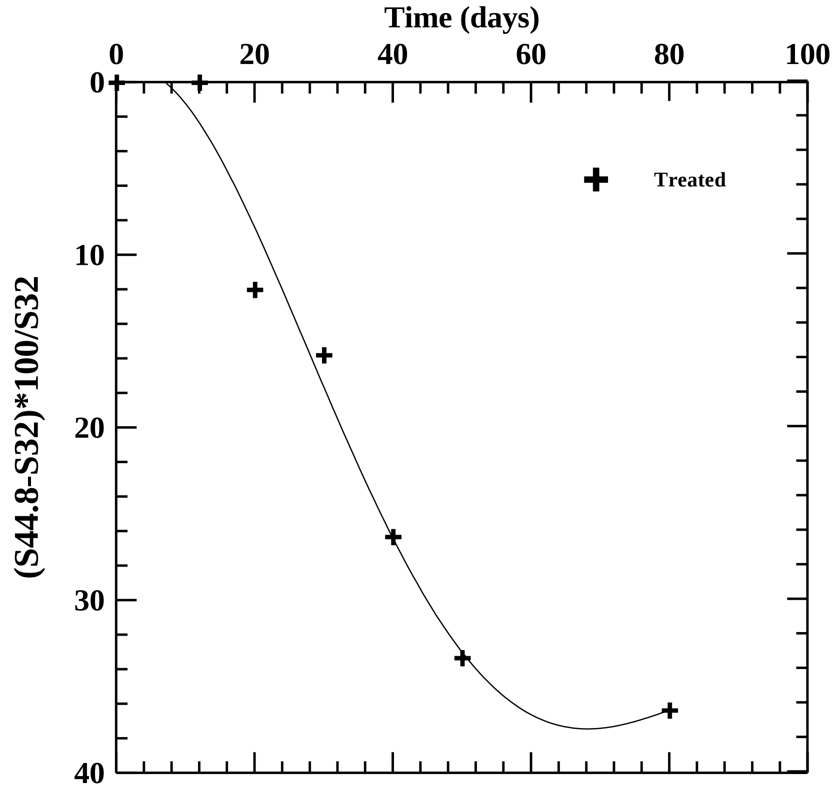


Figure (5.26) Settlement versus Time (Effect of Applied Stress)

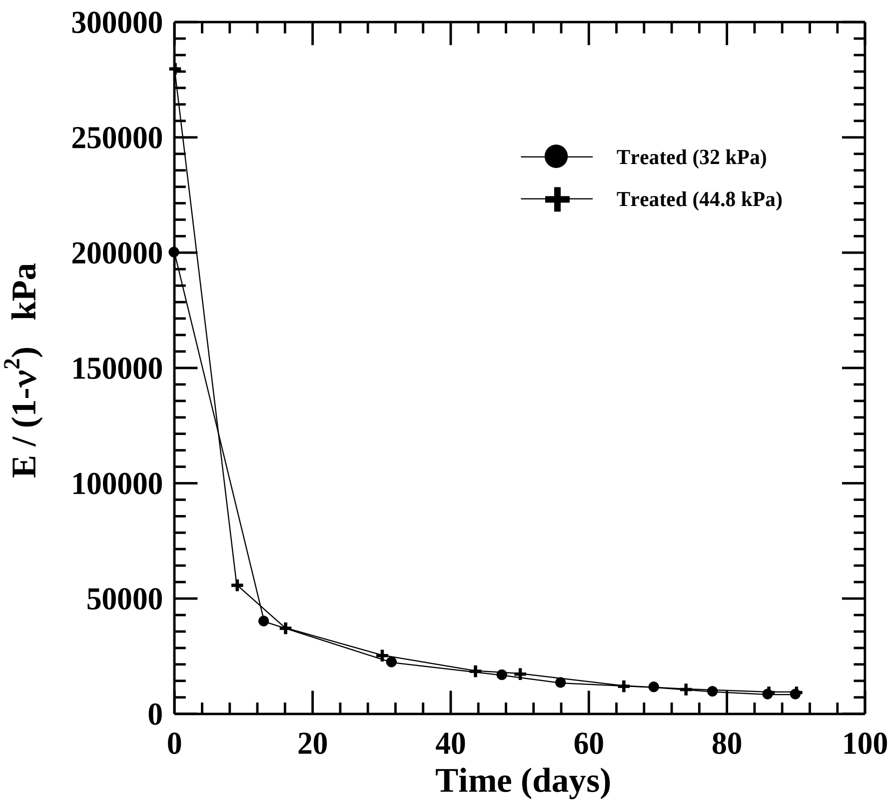


Figure (5.27) $E / (1-\nu^2)$ versus Time
(Effect of Applied Stress)

5.5 Comparison Between Treated and Untreated Field Models at P=32 kPa

Figures 5.28 and 5.29 show the time-settlement relationship for field tests No.1 untreated and field test No.3 treated plotted on normal and semi logarithm scale respectively. It is clear that the presence of stone columns has reduced the amount of settlement in other words, the presence of stone columns underneath the footing caused a delayed compression. The final reduction in settlement is 31 %.

Figure 5.30 illustrate the variation of the E / $(1-\nu^2)$ versus time for the two model tests. The increase in modulus of deformation due to flooding decreased by the presence of stone columns. After continuous flooding for a long period, the modulus of deformation showed about 32% increase for the treated soil as compared to the untreated one.

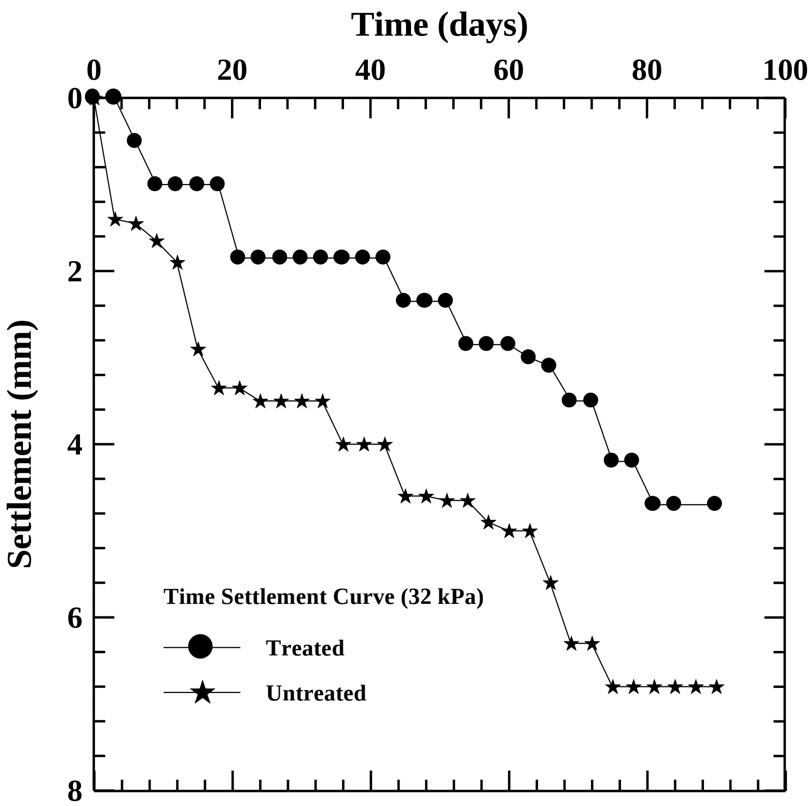


Figure (5.28) Settlement versus Time (Applied Stress = 32 kPa)

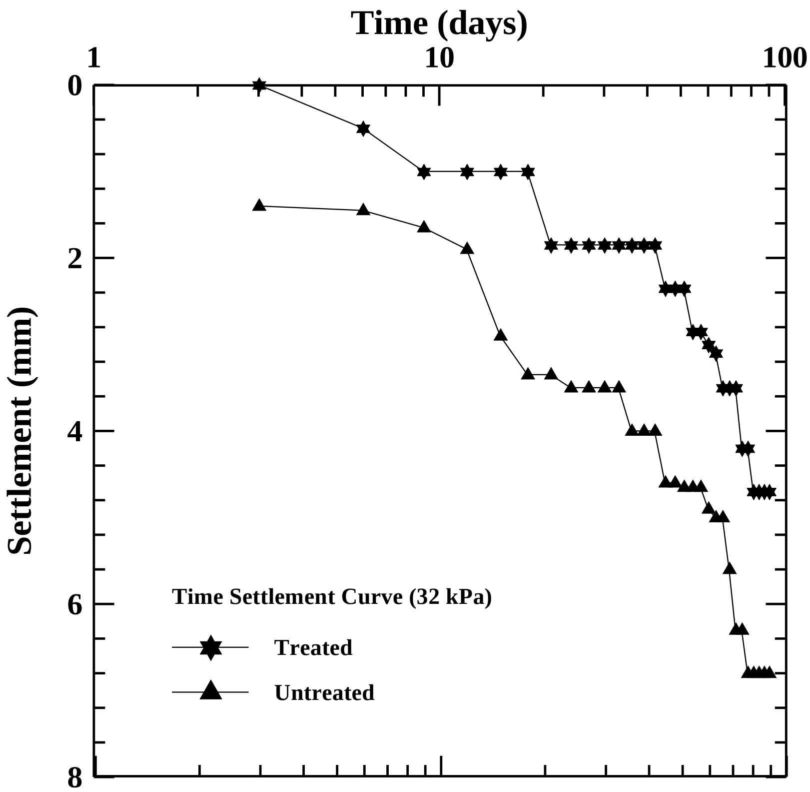


Figure (5.29) Settlement versus Time (Semi log Scale)
(Applied Stress = 32 kPa)

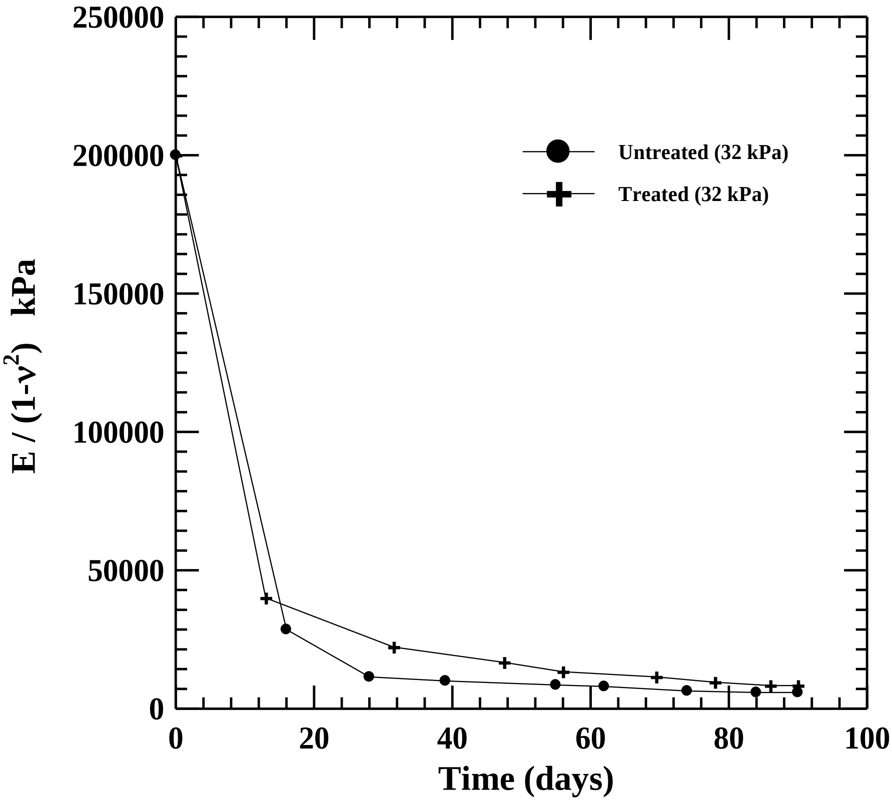


Figure (5.30) E / (1-ν²) versus Time
(Applied Stress = 32 kPa)

5.6 Comparison Between Treated and Untreated Field Models at P = 44.8 kPa

Figures 5.31 and 5.32 show the time-settlement relationship for field models No.2 and No.4. The ground treated with stone columns exhibited an appritiable amount of reduction in settlement especially after 40 days of flooding and further up to 90 days. The final reduction in settlement is about 50%. This improvement in reduction of settlement is very satisfactory and indicates that within the limits of the materials and dimensions of stone columns and backfill material. The technique is considered as a successful one.

Figure 5.33 shows the E / $(1-\nu^2)$ versus time relationship, again the increase in modulus of deformation due to flooding decreased by the presence of stone columns. About 50% of the modulus of deformation only occurred in the treated ground as compared with the untreated soil.

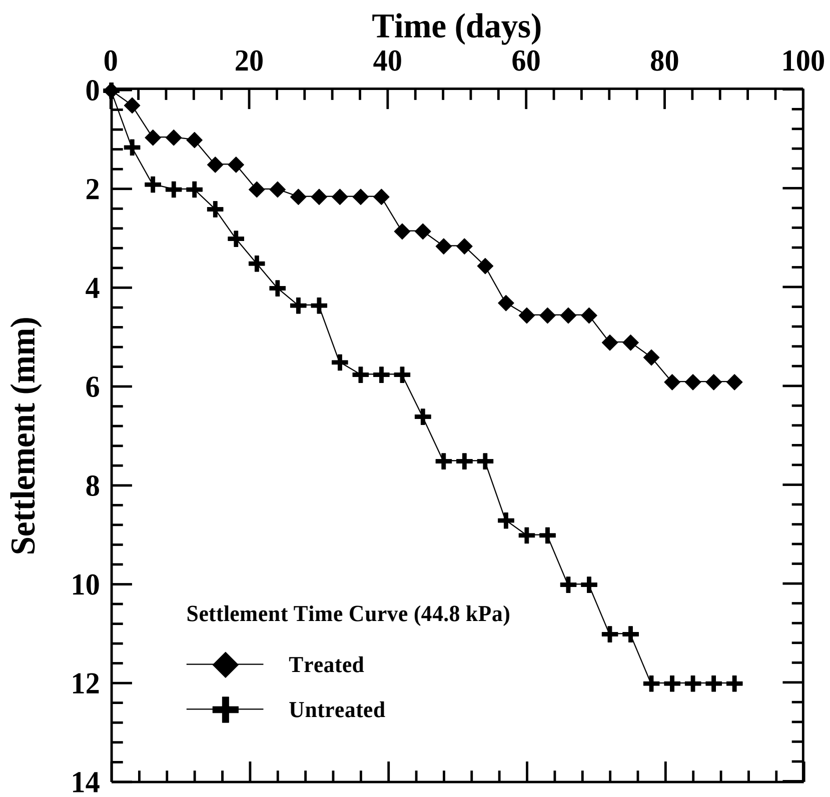


Figure (5.31) Settlement versus Time (Applied Stress = 44.8kPa)

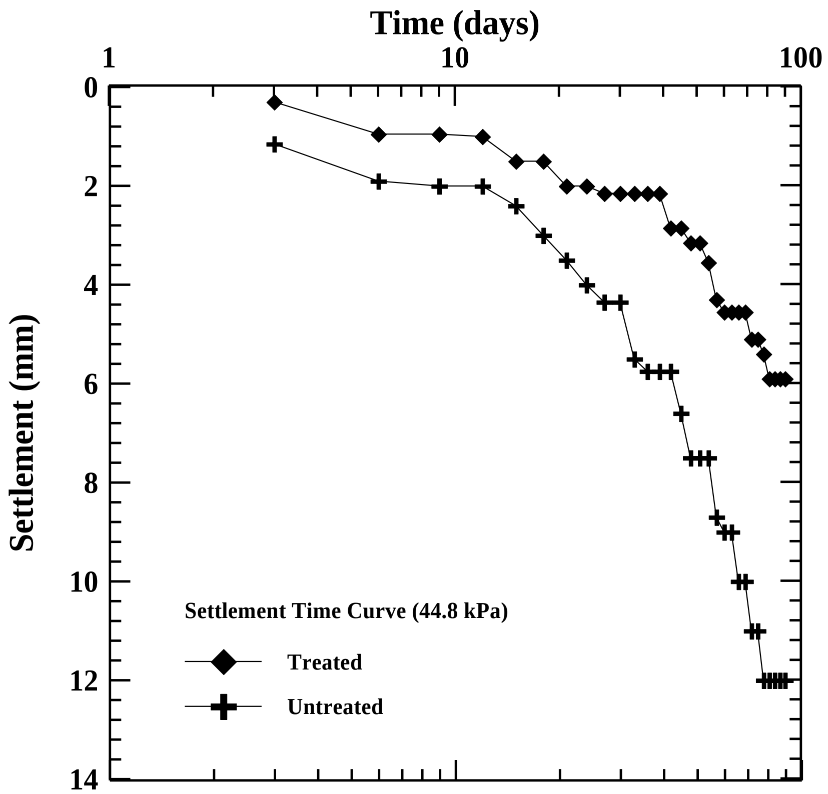


Figure (5.32) Settlement versus Time (Semi log Scale)
(Applied Stress = 44.8kPa)

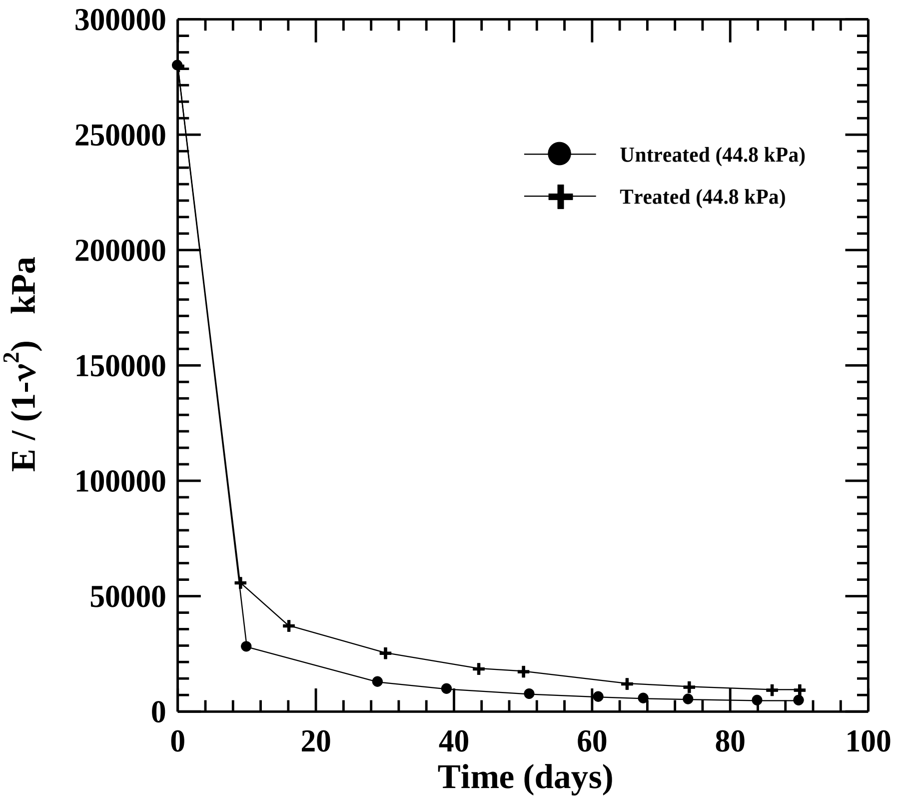


Figure (5.33) E / (1-ν²) versus Time
(Applied Stress = 44.8 kPa)

Chapter Six

1

Conclusions and Recommendations

Chapter Six

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The following points are drawn from the field model tests performed on Al-Dour site. The results are limited to the site conditions.

1. The field model footing for the untreated soil under applied stresses of 32 kPa and 44.8 kPa exhibited a final settlement of 6.8 mm and 12mm after 90 days flooding period respectively. Thus the settlement increased by 43% when the applied stress increased by 29%.
2. The field model footing for the treated soil under applied stress of 32 kPa and 44.8 kPa exhibited a final settlement of 4.7mm and 5.9mm after 90 days flooding period respectively. Thus the settlement increased by 20 % when the applied stress increased by 29 %.
3. $E / (1-\nu^2)$ versus time exhibited a gradual degradation form. For the untreated and treated gypseous soils, at 32 kPa applied stress, E decreased from 175556 kPa to 5000 kPa for untreated soil and from 175556 kPa to 7313 kPa for the treated soil due to 90 days flooding period. Under applied stress 44.8 kPa, the E decreased from 245833 kPa to 4095 kPa for untreated soil and from 245833 kPa to 8328 kPa for the treated soil.
4. The presence of stone columns caused a delayed compression during the flooding period in addition to the reduction of the final settlement. The percent reduction of settlement is 31 % and 51 % at 32 kPa and 44.8 kPa respectively.
5. The increase in E due to the presence of stone columns is 32% and 51% at 32 kPa and 44.8 kPa applied stress respectively.

6. As an overall view based on the collected data, the stone columns are considered a successful technique for controlling the collapsibility of gypseous soils.

7.2 Recommendations

1. The use of different patterns of stone columns (configuration, plug…etc.) in gypseous soil under full-scale field models.
2. Investigating the tendency of using stabilized stone columns with gypseous soils of medium to low gypsum content under full-scale field models.

References

References

- Abood, M.K (1993), “Treatment of Gypseous soil with Sodium Silicate” M.Sc. Thesis, Building and Construction Department, University of Technology.
- Ahmed, A.A. (1997), “Performance of Stone Columns in Soft Soil” Ph.D. Thesis, College of Engineering, University of Baghdad.
- Al-Ani, H.M. and Sherief, J.A. and Mansour, S.L. and Hasso, H.G. (1988), “An Investigation into the Effect of Gypsum on the Properties of Highway Materials” Symposium of Roads, Ministry of Residence, Baghdad.
- Al-Ani, M.M. and Seleam, S.N. (1993), “The Effect of Initial Water Content and Soaking Pressure on the Geotechnical Properties of Gypseous Soil” Al-Muhandis No.116.
- Al-Abdullah, S.F., Dalaly, N.K., Seleam, S.N. (2000), “A Proposed Approach on Improvement of Gypseous Sandy Soil”, Journal of Engineering and Broughting, Al-Mustansirya University, Baghdad.
- Al-Alawee, A.B. (2001), “Treatment of Al-Therthar Gypseous Soil by Emulsified Asphalt Using Model Tests” M.Sc. Building and Construction Department, University of Technology.
- Al-Aqaby, M.D. (2001), “Effect of Kerosene on Properties of Agypseous Soil” M.Sc. Thesis, College of Engineering, University of Baghdad.
- Al-Badran, Y.M. (2001), “Collapse Behavior of Al-Therthar Gypseous Soil” M.Sc. Thesis, College of Engineering, University of Baghdad.
- Al-Dilaimy, F.K. (1989), “Effect of Gypsum Content on Strength and Deformation of Remolded Clayed Soil” M.Sc. Thesis, College of Engineering, University of Salahddin. (In Arabic).

- Al-Heeti, A.A.H. (1990), "The Engineering Properties of Compacted Gypsified Soil" M.Sc. Thesis, Civil Engineering Department, University of Baghdad.
- Al-Hassany, H.Y. (2001), "Gypseous Soil Improvement Using Fuel Oil" M.Sc. Thesis, Building and Construction Department, University of Technology.
- Al-Hadithy, D.K. (2001), "Compressibility of Saline Soils Treated with Cement" M.Sc. Thesis, Building and Construction Engineering, University of Technology.
- Al-Janabi, F.H. (2002), "Assesment of Non-Linear Behavior of Rienforced Gypseous Soil Treated with Fuel Oil" M.Sc. Thesis, Building and Construction Department, University of Technology.
- Al-Khashab, M.N. (1981), "Investigation of Foundation Soil Behavior of Qadisiyah Site-Mosul" M.Sc. Thesis, Civil Engineering Department, University of Mosul.
- Al-Kaisi, A.A. (1997), "Finite Element Analysis of Earth Structures Founded on Gypseous Soils" Ph.D. Thesis, Building and Construction Department, University of Technology.
- Al-Khafaji, A.N. (1997), "Densification of Gypsious Soil by Compaction" Symposium on Ground Improvement Geosystems, London.
- Al-Mufty, A.A. (1997), "Effect of Gypsum Dissolution on Mechanical Behavior of Gypseous Soils" Ph.D. Thesis, Department of Civil Engineering, University of Technology.
- Al-Morshedy, A.D. (2001), "The Use of Cut-Back Mc-30 for Controlling the Collapsibility of Gypseous Soils" M.Sc. Thesis, Building and Construction Department, University of Technology.

- Al-Mosawy, R.H. (2001), "Stone Columns Improved by Asphalt-Cement Materials" M.Sc. Thesis, Building and Construction Department, University of Technology.
- Al-Nori I.A. and Al-Qaissy (1990), "Gypsum Migration and its Effect on Compressibility and Shear Strength" Symposium on Recent Advances in Geotechnical Engineering, 6 July, 1990: pp. 8-14.
- Al-Neami, M.M. (2000), "Effect of Kaolin on Mechanical Properties of Gypseous Soils" M.Sc. Thesis, Civil Engineering Department, Military College.
- Al-Obydi, M.A (1992), "Lime Stabilization of Gypseous Soils" M.Sc. Thesis, Civil Engineering Department College of Engineering, University of Mosul.
- Al-Obaidy, N.K. (2000), "Full-Scale Tests on Stone Piles" M.Sc. Thesis, College of Engineering, University of Baghdad.
- Al-Qaissy, F.F. (1989), "Effect of Gypsum Content and its Migratio on Compressibility and Shear Strength of the Soil" M.Sc. Thesis, Building and Construction Department, University of Technology.
- Al-Qayssi, M.R. (2001), "Unreinforced and Reinforced Behavior of Single and Groups of Granular Piles" Ph.D. Thesis Submitted to the Civil Engineering Faculty of the Military College of Engineering.
- Al-Recaby, M.H. (1999), "Stabilized Stone Columns in Soft Clay with New Reinforcement Pattern" M.Sc. Thesis, Building and Construction Department, University of Technology.
- Al-Shaikhly, A.A. (2000), "Effected of Stone Grain Size on the Behavior of Stone Column" M.Sc. Thesis, Building and Construction Engineering, University of Technology.

- Al-Saoudi, N., and Al-Sheikha, A.H. (2001), “The Collapse Behavior of Gypseous Soils”, Jordanian Geologists Association, The Seventh Jordanian Geological Conference.
- Al-Zory, E.A (1993), “The Effect of Leaching on Lime Stabilized Gypseous Soils” M.Sc. Thesis, Department of Civil Engineering, University of Mosul.
- Barazanji, A.F. (1973), “Gypsiferous Soils of Iraq” Ph.D. Thesis, Dissertation, Ghent University, Belgium.
- Bowles, J. E. (1996), “Foundation Analysis and Design” McGraw-Hill Company Inc. Fifth Edition.
- Buggy, F.H., Martinez and Hussin, D. (1994), “Performance of Oil Storage Tanks on Vibration Improved Hydraulic fill in the Port of Tampa, Florida” Proceedings of Settlement 94 Sponsored by the Geotechnical Engineering Division of the ASCE, Vol.1, 16-18 June, pp.548-560.
- Buringh, P. (1960), “Soils and Soil Conditions in Iraq” 322 Ministry of Agriculture, Baghdad.
- Clemence, S.P. and Finbar, A.O. (1981), “Design Consideration for Collapsible Soils” Journal of Geotechnical Engineering Division, Proc. ASCE, Vol.107, No.GT3, pp.305-317.
- Dubdub, A.J., Al-Abudullah, S.F., and Al-Busoda, B.S. (2000), “Treatment of Gypseous Soils During Loading”
- Dudly, J.H. (1970), “Review of Collapsing Soil” Jor. Soil Mechanics, Foundation Division Proc., ASCE, Vol.96, No.5M.3, and pp.925-947.
- FAO (1990), “Management of Gypsiferous Soils” Food and Agricultural Organization of the United Nations Rome, Internet, http//fao.org/docrep/to323e/ro323e03.htm.

- Hassan, H.A. (2000), “Properties of Gypseous Soils Stabilized with Emulsified Asphalt” M.Sc. Thesis, Department of Civil Engineering, College of Engineering.
- Hughes, J.M.O., Green Wood, D.A. and Withers, N.J. (1975), “A Field Trial of the Reinforcing Effect of Stone Columns in Soft Clay”, Geotechnique, Vol.25, No.1, March, pp.31-44.
- Hussin, A.A. (2000), “The Behavior of Group of Stone Columns in Soft Clay” M.Sc. Thesis, Building and Construction Engineering, University of Technology.
- Ismail, H.N. (1994), “The Use of Gypseous Soils”, Sympsium on the Gypsiferous Soils and its Effects on Structures, NCCL, Baghdad.
- Jawad, K. (1994), “Problems of Irrigation Structures and Contains in Gypseous Soils and their Remedy”, Symposium on Gypseous Soils, NCCL, Baghdad.
- Kattab, S.A. (1986), “Effect of Gypsum on the Strength of Cement Treated Granular Soil and Untreated Soil” M.Sc. Thesis, Department of Civil Engineering, University of Mosul.
- Madhave, M.R. (1982), “Recent Development in the Use and Analysis of Granular Piles” Symposium on Recent Development Technique, Bankok, 29 No.2 Dec., 1982, pp.117-129.
- Mahdi, A.S. (2002), “Basic Design Parameters for Different Stone Column Configurations” M.Sc. Thesis, Building and Construction Department, University of Technology.
- Meckenne, J.M. and Eyre, W.A. (1975), “Performance of an Embankment Supported by Stone Columns in Soft Ground”, Geotechnique, Vol.25, No.1, March, pp.51-59.

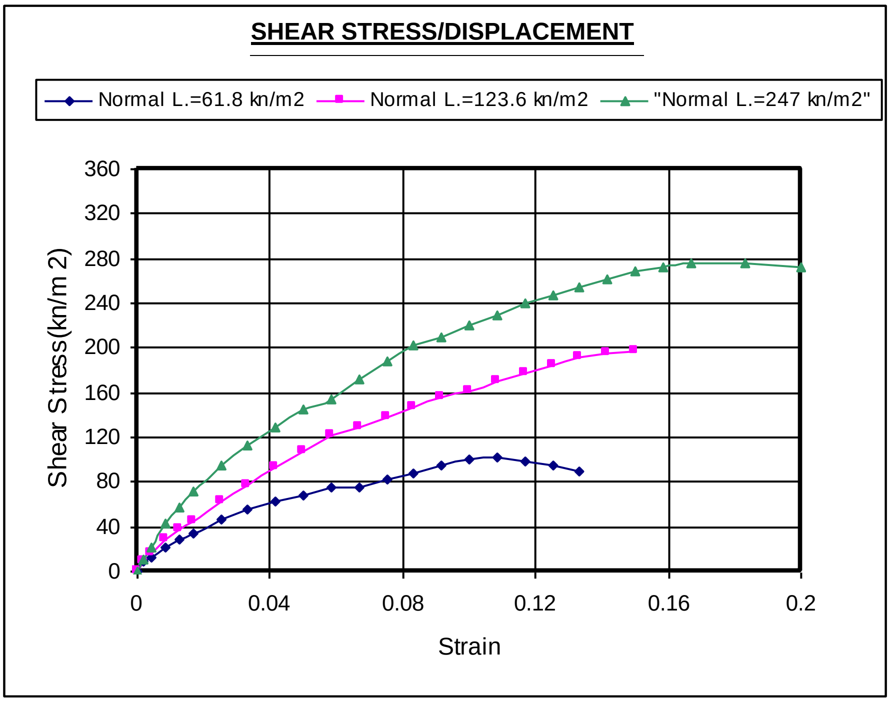
- Mikheev, V. V., Petrukhin, V. P., and Boldirev, G. V. 1997, "Deformability of Gypseous Soils" Proceeding of 9th International Conference on Soil Mechanics and Foundation Engineering, Vol.1, p.p.211-214.
- Mohammed, R. K. (1993), "Effect of Wetting and Drying of Engineering Characteristics of Gypseous Soils" M.Sc. Thesis, Department and Construction Engineering, University of Technology.
- Nashat, I.H. (1990), "Engineering Characteristics of Gypseous Soils of Iraq" Ph.D. Thesis, College of Engineering, University of Baghdad.
- NCCL, (1992), "Tourist Hotel in Tikreet" Symposiums on Gypseous soils, Report No. 144, Baghdad, Iraq.
- Parsons, B. (1983), "Stoned in New Orleans" New Civil Engineering Magazine of the Institution of Civil Engineers, No.529, 24Feb. pp.36-39.
- Petrukhin, V. P., and Boldyrev, G. B. (1978), "Investigation of the Deformability of Gypsified Soils by a Static Load" Soil Mechanics and Foundation Engineering, Vol.15, No.3, p.p.178-182.
- Petrukhin, V. P., and Alperovich, S. V. (1980), "Deformation Mechanism of Cemented Clayey Soils" Soil Mechanics and Foundation Engineering, Vol.17, No.5, p.p.51-57.
- Petrukhin, V.P. and Arakelyan, E.A. (1985), "Strength of Gypsum-Clay and its Variation During the Leaching" Soil Mechanics and Foundation Engineering, Vol.21, No.6, pp.23-25.
- Ramiah, B.K. (1982), "Effect of Chemicals on Compressibility and Strength of Baghdad Silty Clay" Building Research Centre, Baghdad R.D.70/80.

- Rathegeb, E., and Kutzner, C. (1975), “Some Application of the Vibro-Replacement Process” Geotechnique, Vol.25, No.1, March, pp.45-50.
- Razouki, S.S., Al-Omari, R. and Nashat, I.H. (1994), “The Problems of Gypseous Soils in Iraq” 1st Scientific Conference on Soil Mechanics and Foundation Engineering, NCCL, Baghdad.
- Saaed, S.A. and Khorshid, N.N. (1989), “Some Basic Characteristics of the Gypsiferous Soils of Al-Dour” Proceeding of the 5th Scientific Conference, Building Research, Vol.4, Part 2, p.p.76-88, Baghdad. (in Arabic).
- Sales, J.A.J. Justo, Romana, M. and Faraco, c., (1973), “The Collapse of Gypseous Silts and Clays of Low Plasticity in Arid and Semi-Arid Climates” Proc. of 8th International conference of Soil Mechanics and Foundation Engineering, Vol.2, pp.193-199, Moscow.
- Seleam, S.N. (1988), “Geotechnical Characteristics of Contamination with Some Oil Products” M.Sc. Thesis, Building and Construction Department, University of Technology.
- Shubber, A.A. (1990), “Stabilization of Gypseous soil by Bituminous Materials of Highway Use” Building and Construction Department, University of Technology.
- Singh,R. B., and Al-Layla, M. T. (1979), “An Investigation of the Deformability of Mosul City” Proc. of the Asian Regional Conf. on S.M.F.E., Singapore.
- State Establishment of Geological Survey and Mining (Geosurve), (1993), “Geological Map of Iraq” A Study of the Geological Ages for Iraqi Map.

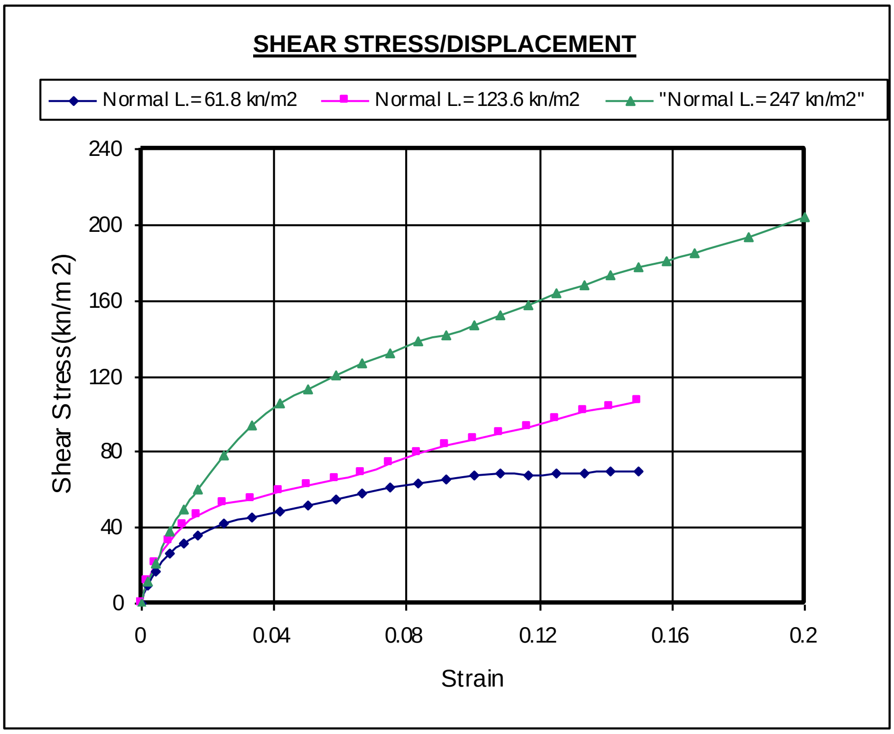
- State Establishment of Geological Survey and Mining (Geosurve), (2002), "Geographic and Thickness Distribution of the Fatha Formation in Iraq" A Geological Study for the Fatha Formation.
- Stone Columns Geotechnics America Inc., (2002), Internet http://tc1/poly.eau/vc.htm//.
- Stone Column Installation Specifications (2002), Geotechnics American Inc, Internet http://tc1/poly.eau/vc.htm//.
- Subhi, H.M. (1987), "The Properties of Salt Contaminated Soils and their Influence on the Performance of Roads in Iraq" Ph.D. Thesis, Queen Mary College, University of London.
- Taha, S.A.W. (1979), "The Effect of Leaching on the Engineering Properties of Qayiqra Soil" M.Sc. Thesis, College of Engineering, University of Mosul.
- Van Alphen, J.G, and Romero, F.D.E (1971), "Gypseous Soils, Notes on their characteristics and Management" International Institution for Land Reclamation and Improvement, Wegeningen, Netherlands.

Appendix A

Appendix A

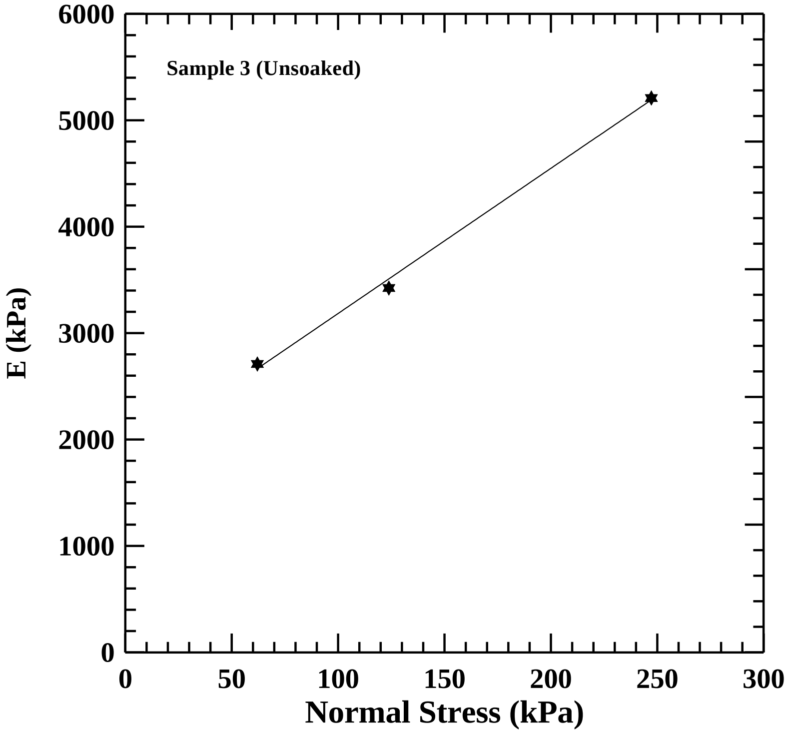


Results of Direct Shear (Sample 3, Before Flooding)

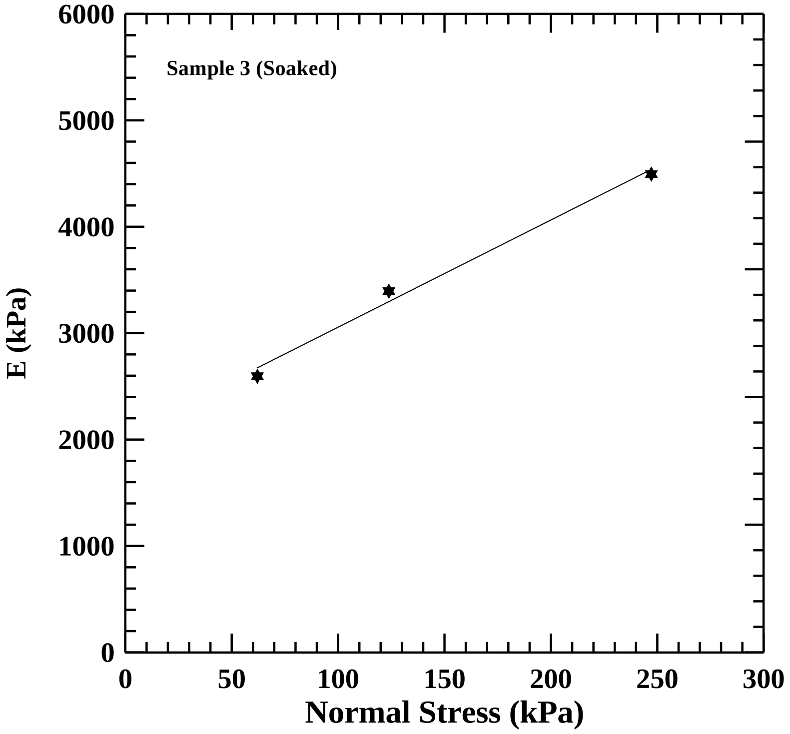


Results of Direct Shear (Sample 3, After Flooding)

Appendix A



Results of Modulus of Elasticity (Sample 3, Before Soaking)



Results of Modulus of Elasticity (Sample 3, After Soaking)

Appendix A

Table (A-1) Results of Modulus of Elasticity, c, ϕ

Test No.	Soil State	c (kPa)	ϕ (Deg.)	E at (32kPa)	E at (44.8kPa)
1	Before Flooding	21.5	50	1673.08	1846.15
	After Flooding	46	30.5	1307.69	1500
2	Before Flooding	35	47.2	1700	2000
	After Flooding	33.2	36.5	733.33	1000
3	Before Flooding	36.84	47	2350	2500
	After Flooding	31.2	35.15	2321.43	2464.29
4	Before Flooding	25.9	51.2	2950	3350
	After Flooding	24.4	34.9	2913.04	3086.96
5	Before Flooding	48.7	45.6	2875	3000
	After Flooding	47.1	31.58	2346.15	2461.54
6	Before Flooding	46.96	45.66	2518.52	2648.15
	After Flooding	26.6	41.1	1800	1890

الخلاصة

إن تقنية الأعمدة الحجرية هي واحدة من الطرائق المعروفة لتحسين أداء الترب الطينية الضعيفة تحت الأسس من حيث زيادة قوة التحمل وتقليل الهبوط.

في هذه الدراسة تم استخدام الأعمدة الركامية المثبتة بالإسفلت والنورة لغرض السيطرة على انهيارية الترب الجبسية بسبب انغمارها بالماء. إن هذه المحاولة هي الأولى من نوعها في استخدام هذه التقنية في الترب الجبسية.

تم عمل أربعة نماذج لأسس مربعة الشكل بأبعاد $1.25m \times 1.25m$ مشيدة على تربة جبسية في موقع قريب من معمل كبريتات الصوديوم في منطقة الدور. اثنان من النماذج الأربعة استند مباشرةً على التربة الطبيعية والآخران استندا على تربة طبيعية معالجة بأربعة أعمدة حجرية. الأعمدة الحجرية كانت بقطر 0.31m وبطول 1.5m. تم تثبيت الحجر باستخدام الإسفلت بنسبة 3.75% مع النورة بنسبة 7.5%.

تم تسليط أحمال بمقدار 32 kPa و 44.8 kPa على كل أساس ومن ثم غمرت المنطقة كلياً بالماء لمدة 90 يوم وبمعدل $20\text{-}30\ m^3$ ماء يومياً.

لقد بينت النماذج المستندة على الأعمدة الحجرية نجاح التقنية مقارنة بالتربة غير المعالجة.

جمهورية العراق
وزارة التعليم العالي والبحث العلمي
الجامعة التكنولوجية

النماذج الحقلية المستندة على ترب جبسية مسلحة بالأعمدة الركامية المثبتة بالاسفلت والنورة

رسالة
مقدمة إلى قسم هندسة البناء والانشاءات في الجامعة التكنولوجية
كجزء من متطلبات نيل درجة الماجستير في علوم الهندسة الجيوتكنيكية

من قبل المهندس
نهاد بهاء الدين صالح
(بكالوريوس هندسة مدنية، 1996)

تحت إشراف
الدكتور نمير خورشيد السعودي
الدكتور حسين حميد كريم

آذار – 2003