

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**STATIC AND CYCLIC PROPERTIES OF EXPANSIVE CLAYS TREATED
WITH LIME AND FLY ASH WITH SPECIAL REFERENCE TO SWELLING
AND RESILIENT MODULI**



Ph.D. THESIS

İnci SÜT ÜNVER

Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Program

DECEMBER 2018

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**STATIC AND CYCLIC PROPERTIES OF EXPANSIVE CLAYS TREATED
WITH LIME AND FLY ASH WITH SPECIAL REFERENCE TO SWELLING
AND RESILIENT MODULI**

Ph.D. THESIS

İnci SÜT ÜNVER
(501102303)

Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Program

Thesis Advisor: Prof. Dr. Musaffa Ayşen LAV
Thesis Co-Advisor: Prof. Dr. Erdal ÇOKÇA

DECEMBER 2018

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**KİREÇ VE UÇUCU KÜL İLE İYİLEŞTİRİLEN ŞİŞEN KİLLERİN
ŞİŞME VE ESNEKLİK MODÜLÜ ODAKLI STATİK VE TEKRARLI YÜKLER
ALTINDAKİ ÖZELLİKLERİ**

DOKTORA TEZİ

**İnci SÜT ÜNVER
(501102303)**

İnşaat Mühendisliği Anabilim Dalı

Zemin Mekaniği ve Geoteknik Mühendisliği Programı

**Tez Danışmanı: Prof. Dr. Musaffa Ayşen LAV
Eş Danışman: Prof. Dr. Erdal ÇOKÇA**

ARALIK 2018

İnci SÜT ÜNVER, a Ph.D. student of İTUGraduate School of ScienceEngineering andTechnology student ID: 501102303, successfully defended the thesis/dissertation entitled “STATIC AND CYCLIC PROPERTIES OF EXPANSIVE CLAYS TREATED WITH LIME AND FLY ASH WITH SPECIAL REFERENCE TO SWELLING AND RESILIENT MODULI”, which she prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor : **Prof. Dr. Musaffa Ayşen LAV**
İstanbul Technical University

Co-advisor : **Prof. Dr. Erdal ÇOKÇA**
Middle East Technical University

Jury Members : **Prof. Dr. Aykut ŞENOL**
İstanbul Technical University

.....
Prof. Dr. Reha ARTAN
İstanbul Technical University

.....
Assoc. Prof. Dr. İsmail Hakkı AKSOY
İstanbul Technical University

.....
Prof. Dr. Gökhan BAYKAL
Boğaziçi University

.....
Prof. Dr. İlknur BOZBEY
İstanbul University

Date of Submission : 24 October 2018

Date of Defense : 03 December 2018





Dedicated to my mother and father,



FOREWORD

I would like to extend my sincere and special thanks to my advisor, Prof. Dr. Musaffa Ayşen Lav and co-advisor Prof. Dr. Erdal Çokça for their guidance, encouragement and valuable suggestions throughout this research. Their continuous support and patience have made my research come into this stage. I will always remember their positive approach at all stages of this research.

I want to thank Prof. Dr. Gökhan Baykal, who is a member in my thesis committee, for his constructive attitude and critical suggestions during this study. Thanks are also due to Assoc. Prof. Dr. İsmail Hakkı Aksoy and Prof. Dr. Reha Artan for their recommendations. Prof. Dr. Abdullah Hilmi Lav is also appreciated for his helps.

I also want to thank Prof. Dr. Emin Çiftçi for specimen preparation for SEM analysis. Special thanks are extended to Soil Mechanics Laboratory staff of Middle East Technical University, especially Mr. Ali Bal and Mr. Ulaş Nacar for their helpful attitudes throughout the laboratory experiments.

I want to extend my thanks to Mr. Sina Kızıroğlu, Head of Department of Technical Research and soil laboratory staff of General Directorate of Highways for the resilient modulus tests.

I also want to thank my colleagues at Atılım University, Mr. Burak Şahin, Mr. Tunç Gökdemir and Mr. Cumhuri Özbey for their help and encouragement during this study.

These thanks are also due to my friends at İstanbul Technical University, Mrs. Emre Köroğlu, Miss. Hilal Erdem, Mrs. Esra Tatlıoğlu, Mr. Dara Hajebi, and Mr. Ahmet Kanan for their fellowship.

I want to extend my special thanks to my former advisor Dr. Ing. Ünsal Soygür from Gazi University for his guidance in my professional and personal development.

I want to thank my husband Mengüç Ünver also for his endless support and guidance in my life and patience and encouragement during this study. Special thanks are also due to his valuable technical suggestions as being my colleague in geotechnical field.

Finally, I wish to express my special gratitudes to my family members; my father, Yüksel Süt, my mother, Emine Süt, and my sister, Dr. Çağıl Süt Göker for their affection, concern and constant support all through my life. Dr. Cenker Göker is also appreciated for his academic advices.

December 2018

İnci SÜT ÜNVER
(Civil Engineer, M.S.)

TABLE OF CONTENTS

	<u>Page</u>
FOREWORD	ix
TABLE OF CONTENTS	xi
ABBREVIATIONS	xv
SYMBOLS	xvii
LIST OF TABLES	xix
LIST OF FIGURES	xxi
SUMMARY	xxx
ÖZET	xxxv
1. INTRODUCTION	1
2. LITERATURE SURVEY	5
2.1 General	5
2.2 Soil Mineralogy	7
2.2.1 Cohesionless soils	8
2.2.2 Clay minerals	8
2.2.2.1 XRD and Rietveld tests	11
2.2.3 Swell mechanism in clays	12
2.2.3.1 Factors affecting swell potential	14
2.2.4 Classification of expansive soils	16
2.3 Chemical Stabilization	20
2.3.1 Lime stabilization	20
2.3.1.1 Lime-soil reactions	22
2.3.2 Fly ash stabilization	24
2.3.2.1 Fly ash-soil reactions	25
2.4 Cyclic Properties of Treated Soils.....	26
2.4.1 Resilient modulus test	26
2.4.1.1 Resilient modulus of lime-treated soils	28
2.5 Past Studies on Expansive Soils.....	29
3. EXPERIMENTAL PROGRAM	39
3.1 Obtaining Soil Samples from Different Sites.....	39
3.1.1 Esenboğa site.....	41
3.1.2 Bilkent site	44
3.2 Experimental Schedule.....	46
3.3 Untreated (Natural) Soil Properties.....	49
3.3.1 Mineralogy of Esenboğa Clay	50
3.3.1.1 XRD and Rietveld tests	51
3.3.1.2 XRF tests	52
3.3.2 Mineralogy of Bilkent Clay	53
3.3.2.1 XRD and Rietveld tests	53
3.3.2.2 XRF tests	55
3.4 Properties of Chemical Agents	56
3.4.1 Lime	56
3.4.1.1 XRF test	56

3.4.2 Fly ash	57
3.4.2.1 XRF test	57
3.5 Preparation of Samples for Tests	58
3.5.1 Unconfined compression test	60
3.5.2 Oedometer test	66
3.5.3 Resilient modulus test	69
3.5.4 Methylene blue test	73
3.5.5 SEM (Scanning Electron Microscope) analyses.....	74
3.6 Definitions and Method Statements for Tests	77
3.6.1 Index tests	77
3.6.2 Unconfined compression tests	80
3.6.3 Oedometer tests	82
3.6.3.1 Free swell test	82
3.6.3.2 Swell pressure test	83
3.6.3.3 Consolidation test	84
3.6.4 Resilient modulus test	85
3.6.5 Methylene blue test	87
3.6.6 SEM (Scanning Electron Microscope) analyses.....	90
4. TEST RESULTS	93
4.1 Properties of Esenboğa Clay	93
4.2 Properties of Bilkent Clay	95
4.3 Lime Stabilization	97
4.3.1 Esenboğa Clay	97
4.3.1.1 Effects of stabilization on index parameters	98
4.3.1.2 Effects of stabilization on swell parameters	99
4.3.1.3 Effects of stabilization on strength parameters	104
4.3.1.4 Effects of stabilization on compression parameters	105
4.3.1.5 Effects of curing time on lime-treated Esenboğa Clay	109
4.3.2 Bilkent Clay	122
4.3.2.1 Effects of stabilization on index parameters	123
4.3.2.2 Effects of stabilization on swell parameters	124
4.3.2.3 Effects of stabilization on strength parameters	129
4.3.2.4 Effects of stabilization on compression parameters	130
4.4 Fly Ash Stabilization	134
4.4.1 Esenboğa Clay	135
4.4.1.1 Effects of stabilization on index parameters	135
4.4.1.2 Effects of stabilization on swell parameters	136
4.4.1.3 Effects of stabilization on strength parameters	142
4.4.1.4 Effects of stabilization on compression parameters	143
4.4.2 Bilkent Clay.....	146
4.4.2.1 Effects of stabilization on index parameters	147
4.4.2.2 Effects of stabilization on swell parameters	148
4.4.2.3 Effects of stabilization on strength parameters	153
4.4.2.4 Effects of stabilization on compression parameters	154
4.5 Resilient Modulus Tests	159
4.5.1 Esenboğa Clay	159
4.5.1.1 Effects of time on lime-treated Esenboğa Clay	159
4.5.2 Bilkent Clay	162
4.5.2.1 Effects of time on lime-treated Bilkent Clay	162
4.5.3 Effect of stress state on resilient modulus values	167

4.6 Methylene Blue Test (MBT)	169
4.6.1 MBT for Esenboğa Clay	169
4.6.2 MBT for Bilkent Clay	170
4.7 Tests Conducted after Resilient Modulus Test on RM Test Specimens.....	171
4.7.1 Unconfined compression strength test	171
4.7.1.1 Esenboğa Clay	172
4.7.1.2 Bilkent Clay	175
4.7.2 SEM (Scanning Electron Microscope) analyses.....	179
4.7.2.1 SEM for natural Esenboğa Clay.....	179
4.7.2.2 SEM for lime-treated Esenboğa Clay.....	186
4.7.2.3 SEM for natural Bilkent Clay.....	192
5. DISCUSSION	203
5.1 Comparison of Lime-Treated Samples of Esenboğa and Bilkent Clay.....	203
5.1.1 Index parameters	204
5.1.2 Swell parameters	206
5.1.3 Strength parameters	210
5.1.4 Compression parameters	211
5.2 Comparison of Fly Ash-Treated Samples of Esenboğa and Bilkent Clays....	212
5.2.1 Index parameters	213
5.2.2 Swell parameters	215
5.2.3 Strength parameters	219
5.2.4 Compression parameters	221
5.3 Comparison of Resilient Modulus Test	222
5.3.1 Resilient modulus	222
5.4 Comparison of Methylene Blue Test (MBT)	226
5.5 Comparison of Tests Conducted after Resilient Modulus Test on RM Test Specimens	228
5.5.1 Unconfined compression test	228
5.6 Comparison of SEM (Scanning Electron Microscope) Analyses	231
6. CONCLUSIONS AND RECOMMENDATIONS.....	233
6.1 Recommendations for Future Research	239
REFERENCES.....	241
APPENDICES	249
APPENDIX A	250
APPENDIX B	254
APPENDIX C	264
APPENDIX D	286
CURRICULUM VITAE.....	297



ABBREVIATIONS

AASHTO	: American Association of State Highway and Transportation Officials
App	: Appendix
ASTM	: American Society for Testing and Materials
CAH	: Calcium Aluminate Hydrate
CaO	: Calcium Oxide (Lime)
CBR	: California Bearing Ratio
CEC	: Cation Exchange Capacity
CSH	: Calcium Silicate Hydrate
EDAX	: Energy Dispersive X-Ray Analysis Unit
EI	: Expansion Index
FHWA	: Federal Highway Administration
GGBS	: Ground Granulated Blast Furnace Slag
ICSD	: Inorganic Crystal Structure Database
KGM	: Karayolları Genel Müdürlüğü (General Directorate of Highways)
LL	: Liquid Limit
LS	: Linear Shrinkage
MBT	: Methylene Blue Test
MBV	: Methylene Blue Value
MTA	: General Directorate of Mineral Research and Exploration
M_r	: Mean Resilient Modulus Test
pH	: Potential of Hydrogen
PI	: Plasticity Index
PL	: Plastic Limit
RM	: Resilient Modulus
rpm	: Revolutions per Minute
SAMC	: Standard Absorption Moisture Content
SEM	: Scanning Electron Microscope
SL	: Shrinkage Limit
XRF	: X-Ray Fluorescence
XRD	: X-Ray Diffraction



SYMBOLS

A	: Activity
C_c	: Compression Index
C_e	: Expansion Index
D	: Grain Size
$\Delta\sigma_d$	: Deviatoric Stress
$\Delta\varepsilon_a$	: Resilient Axial Stress
e	: Void Ratio
e₀	: Initial Void Ratio
f'	: Dry Weight of Soil Sample
G_s	: Specific Gravity
k	: Coefficient of Permeability
N	: Hazard Coefficient
q_u	: Unconfined Compressive Strength
q_{u soaked}	: Unconfined Compressive Strength (soaked condition)
q_{u non-soaked}	: Unconfined Compressive Strength (non-soaked condition)
V_{cc}	: Volume of Methylene Blue Solution
W_e	: Weight of Sample at Equilibrium
W_s	: Weight of Oven-dry Sample
w_{opt.}	: Optimum Water Content
γ_{drymax}	: Maximum Dry Density
σ'	: Effective Stress
σ_1	: Major Principal Stress
σ_3	: Minor Principal Stress



LIST OF TABLES

	<u>Page</u>
Table 2.1 : The relationship between plasticity index (PI) and expansion potential (Peck et al., 1974).....	17
Table 2.2 : The relationship between activity (A_c) and expansion classification (Skempton, 1953).	17
Table 2.3 : Expansion potential based on expansion index (EI).....	18
Table 2.4 : Classification standard for expansive soils (CMC, 2004)..	19
Table 2.5 : Expansive soil classification system ** (Snethen et al., 1977).....	19
Table 3.1 : Experimental schedule in this research.....	48
Table 3.2 : Index properties of Esenboğa Clay.....	49
Table 3.3 : Index properties of Bilkent Clay.....	49
Table 3.4 : Soil mineralogy of Esenboğa Clay.....	51
Table 3.5 : Major chemical components of Esenboğa Clay.....	53
Table 3.6 : Soil mineralogy of Bilkent Clay.....	54
Table 3.7 : Major chemical components of Bilkent Clay.....	55
Table 3.8 : Chemical composition of hydrated lime used in treatment... ..	56
Table 3.9 : Chemical composition of Soma fly ash (BU), (FU).....	58
Table 3.10 : Typical values of SSA and CEC.....	90
Table 4.1 : Properties of Esenboğa Clay.....	94
Table 4.2 : Properties of Bilkent Clay.....	96
Table 4.3 : Proctor test results for treated samples with lime for Esenboğa Clay.. ..	99
Table 4.4 : Proctor test results for treated samples with lime for Bilkent Clay..	123
Table 4.5 : Proctor test results for treated samples with fly ash for Esenboğa Clay.. ..	136
Table 4.6 : Proctor test results for treated samples with fly ash for Bilkent Clay... ..	148
Table A.1 : Properties of Esenboğa Clay with lime agent.....	250
Table A.2 : Properties of Esenboğa Clay with fly ash agent.....	251
Table A.3 : Properties of Bilkent Clay with lime agent.....	252
Table A.4 : Properties of Bilkent Clay with fly ash agent.....	253



LIST OF FIGURES

	<u>Page</u>
Figure 2.1 : Heave of concrete slabs adjacent to industrial building.....	5
Figure 2.2 : Cracks on wall of a building due to upward soil pressures.	6
Figure 2.3 : Damage at a ground floor due to expansive soil	6
Figure 2.4 : Asphalt road damage due to swelling soils	7
Figure 2.5 : Silicon tetrahedron and silica tetrahedra arranged in a hexagonal network (Mitchell and Soga, 2005).	9
Figure 2.6 : Octahedral unit and sheet structure of octahedral units (Mitchell and Soga, 2005)..	9
Figure 2.7 : Synthesis pattern for the clay minerals (Mitchell and Soga, 2005).....	10
Figure 2.8 : Surface hydration (After van Olphen, 1977).....	13
Figure 2.9 : (a) Swelling of soil with entrance of water in between mineral layers, (b) Interparticle swelling (After Mitchell, 1993).....	14
Figure 2.10 : Criteria for determining swell potential (Das, 1999).....	20
Figure 2.11 : Laying of lime/fly ash and onsite mixing machine.	21
Figure 2.12 : Roller compaction process.	22
Figure 2.13 : Structural transformation as a result of cation exchange and flocculation-agglomeration.....	24
Figure 2.14 : Treatment process before and after lime stabilization.....	24
Figure 2.15 : Definition of resilient modulus.....	27
Figure 2.16 : A typical pavement design.	28
Figure 3.1 : The location of Ankara in Turkey.	40
Figure 3.2 : The location of Akyurt and Çankaya in Ankara.....	40
Figure 3.3 : Metal tube samplers.....	41
Figure 3.4 : Satellite image of Esenboğa Site.	41
Figure 3.5 : Local view of Esenboğa Site..	42
Figure 3.6 : A photograph of the sampling pit at Esenboğa.	42
Figure 3.7 : Pushing the samplers into the ground at Esenboğa.	43
Figure 3.8 : Sample tubes sealed with wax.....	43
Figure 3.9 : Sample tubes in moisture room..	44
Figure 3.10 : Disturbed soil samples of Esenboğa Clay.	44
Figure 3.11 : Satellite image of Bilkent Site.....	45
Figure 3.12 : Pushing the samplers into the ground at Bilkent.	45
Figure 3.13 : Sealing of undisturbed soil samples with wax	46
Figure 3.14 : Grain size distribution of the Esenboğa Clay.	50
Figure 3.15 : Grain size distribution of the Bilkent Clay.....	50
Figure 3.16 : X-Ray diffractograms of Esenboğa Clay for bulk (whole) sample....	52
Figure 3.17 : XRD patterns for clay fraction in Esenboğa Clay.....	52
Figure 3.18 : X-Ray diffractograms of Bilkent Clay for bulk (whole) sample.....	54
Figure 3.19 : XRD patterns for clay fraction in Bilkent Clay.....	55
Figure 3.20 : Harvard miniature compaction test apparatus... ..	59
Figure 3.21 : Obtaining undisturbed soil samples for unconfined compression test.....	60
Figure 3.22 : Obtaining undisturbed soil samples by hydraulic jack.....	60

Figure 3.23 : Trimming process to get undisturbed soil samples.	61
Figure 3.24 : The undisturbed soils samples after trimming.	61
Figure 3.25 : Teflon mould and rods to get samples for unconfined compression test.	62
Figure 3.26 : Compaction process to get samples for unconfined compression test.	62
Figure 3.27 : Extracting soil samples for unconfined compression test.	63
Figure 3.28 : Soil sample for unconfined compression test.	63
Figure 3.29 : Apparatus used for preparation of the samples for unconfined compression test.	64
Figure 3.30 : The mould with filter paper for unconfined compression test (soaked condition).	64
Figure 3.31 : Filter paper covering soil samples for unconfined compression test (soaked condition).	65
Figure 3.32 : Inundation process of the samples for unconfined compression test in soaked condition.	65
Figure 3.33 : Roughened soil samples for oedometer tests of Esenboğa Clay.	66
Figure 3.34 : Roughened soil samples for oedometer tests of Bilkent Clay.	67
Figure 3.35 : Compaction of soil samples for oedometer tests.	67
Figure 3.36 : Compacted soil sample in oedometer ring.	68
Figure 3.37 : Oedometer ring in glass container.	68
Figure 3.38 : Filter paper on the sample for oedometer tests.	68
Figure 3.39 : Apparatus of resilient modulus test samples.	69
Figure 3.40 : Prepared soil samples of resilient modulus test for different cure periods in the moisture room.	70
Figure 3.41 : Soil sample confined by the membrane for resilient modulus test.	71
Figure 3.42 : Resilient modulus test loading frame.	72
Figure 3.43 : Resilient modulus test equipment.	72
Figure 3.44 : Preparation of methylene blue.	73
Figure 3.45 : Methylene blue test.	74
Figure 3.46 : SEM samples in impregnator.	75
Figure 3.47 : Sanding process of samples	76
Figure 3.48 : Cube SEM samples	76
Figure 3.49 : Vacuumed dessicator	77
Figure 3.50 : Preparation of samples for shrinkage limit test.	79
Figure 3.51 : Preparation of samples for shrinkage limit test.	79
Figure 3.52 : Oven-dried soil sample of linear shrinkage limit test.	80
Figure 3.53 : Oven-dried soil sample of shrinkage limit test.	80
Figure 3.54 : Unconfined compression test loading frame.	81
Figure 3.55 : A failed sample in unconfined compression test.	81
Figure 3.56 : Inundation process of the samples for unconfined compression test (soaked condition).	82
Figure 3.57 : Free swell test.	83
Figure 3.58 : Consolidation test.	84
Figure 3.59 : Oven dried samples of consolidation test.	85
Figure 3.60 : Haversine loading pattern for resilient modulus test.	86
Figure 3.61 : Strains under repeated load in resilient modulus test.	86
Figure 3.62 : Chemical composition of methylene blue ion.	88
Figure 3.63 : Adsorption of methylene blue by clay minerals.	89
Figure 3.64 : Flow chart for the methylene blue test	89
Figure 3.65 : Coated specimens on stub.	91

Figure 4.1 : Criteria for determining swell potential (Das, 1999).....	95
Figure 4.2 : Criteria for determining swell potential (Das, 1999).....	97
Figure 4.3 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with lime content.....	98
Figure 4.4 : Criteria for determining swell potential (the marks in the figures are for Esenboğa Clay) (Das, 1999).	100
Figure 4.5 : Variation of swell percentage with lime content.....	101
Figure 4.6 : Variation of net swell percentage with lime content... ..	101
Figure 4.7 : Effect of lime content on swell percentage... ..	102
Figure 4.8 : Variation of swell pressure with lime content.....	103
Figure 4.9 : Variation of net swell pressure with lime content... ..	104
Figure 4.10 : Effect of lime content on swell pressure... ..	104
Figure 4.11 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with lime content.....	105
Figure 4.12 : e - σ' relationship for natural and lime-treated soil samples for Esenboğa Clay.....	107
Figure 4.13 : e - $\log \sigma'$ relationship for natural and lime-treated soil samples for Esenboğa Clay.. ..	108
Figure 4.14 : Compression index (C_c) values with increasing lime content for Esenboğa Clay... ..	109
Figure 4.15 : Expansion index (C_e) values with increasing lime content for Esenboğa Clay.....	109
Figure 4.16 : Variation of swell percentage with lime content without cure.....	110
Figure 4.17 : Variation of swell percentage with lime content for 7 days cure period... ..	111
Figure 4.18 : Variation of swell percentage with lime content for 28 days cure period... ..	111
Figure 4.19 : Variation of swell percentage with lime content for 56 days cure period... ..	112
Figure 4.20 : Variation of swell percentage with lime content for 7, 28 and 56 days cure periods.....	112
Figure 4.21 : Variation of swell pressure with lime content without cure.....	113
Figure 4.22 : Variation of swell percentage with lime content for 7 days cure period... ..	114
Figure 4.23 : Variation of swell percentage with lime content for 28 days cure period... ..	114
Figure 4.24 : Variation of swell percentage with lime content for 56 days cure period.	115
Figure 4.25 : Variation of swell pressure with lime content for 7, 28 and 56 days cure periods.....	115
Figure 4.26 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content without cure... ..	117
Figure 4.27 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 7 days cure period... ..	117
Figure 4.28 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 28 days cure period... ..	118
Figure 4.29 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 56 days cure period... ..	118
Figure 4.30 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 7, 28 and 56 days cure periods... ..	119

Figure 4.31 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content without cure.....	120
Figure 4.32 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 7 days cure period.....	120
Figure 4.33 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 28 days cure period.....	121
Figure 4.34 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 56 days cure period.....	121
Figure 4.35 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 7, 28 and 56 days cure periods.....	122
Figure 4.36 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with lime content.....	123
Figure 4.37 : Criteria for determining swell potential (the marks in the figures are for Bilkent Clay) (Das, 1999).....	125
Figure 4.38 : Variation of swell percentage with lime content.....	126
Figure 4.39 : Variation of net swell percentage with lime content.....	126
Figure 4.40 : Effect of lime content on swell percentage.....	127
Figure 4.41 : Variation of swell pressure with lime content.....	128
Figure 4.42 : Variation of net swell pressure with lime content.....	129
Figure 4.43 : Effect of lime content on swell pressure.....	129
Figure 4.44 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with lime content.....	130
Figure 4.45 : e - σ' relationship for natural and lime-treated soil samples for Bilkent Clay.....	132
Figure 4.46 : e -log σ' relationship for natural and lime-treated soil samples for Bilkent Clay.....	133
Figure 4.47 : Compression index (C_c) values with increasing lime content for Bilkent Clay.....	134
Figure 4.48 : Expansion index (C_e) values with increasing lime content for Bilkent Clay.....	134
Figure 4.49 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with fly ash content.....	136
Figure 4.50 : Criteria for determining swell potential (the marks in the figures are for Esenboğa Clay) (Das, 1999).....	137
Figure 4.51 : Variation of swell percentage with fly ash content.....	138
Figure 4.52 : Variation of net swell percentage with fly ash content.....	139
Figure 4.53 : Effect of fly ash content on swell percentage.....	140
Figure 4.54 : Variation of swell pressure with fly ash content.....	141
Figure 4.55 : Variation of net swell pressure with fly ash content.....	141
Figure 4.56 : Effect of fly ash content on swell pressure.....	142
Figure 4.57 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with fly ash content.....	142
Figure 4.58 : e - σ' relationship for natural and fly ash-treated soil samples for Esenboğa Clay.....	144
Figure 4.59 : e -log σ' relationship for natural and fly ash-treated soil samples for Esenboğa Clay.....	145
Figure 4.60 : Compression index (C_c) values with increasing fly ash content for Esenboğa Clay.....	146
Figure 4.61 : Expansion index (C_e) values with increasing fly ash content for Esenboğa Clay.....	146

Figure 4.62 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with fly ash content.....	147
Figure 4.63 : Criteria for determining swell potential (the marks in the figures are for Bilkent Clay) (Das, 1999).....	149
Figure 4.64 : Variation of swell percentage with fly ash content....	150
Figure 4.65 : Variation of net swell percentage with fly ash content....	151
Figure 4.66 : Effect of fly ash content on swell percentage.....	151
Figure 4.67 : Variation of swell pressure with fly ash content....	152
Figure 4.68 : Variation of net swell pressure with fly ash content....	153
Figure 4.69 : Effect of fly ash content on swell pressure.....	153
Figure 4.70 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with fly ash content....	154
Figure 4.71 : e - σ' relationship for natural and fly ash-treated soil samples for Bilkent Clay.....	156
Figure 4.72 : e - $\log \sigma'$ relationship for natural and fly ash-treated soil samples for Bilkent Clay.....	157
Figure 4.73 : Compression index (C_c) values with increasing fly ash content for Bilkent Clay	158
Figure 4.74 : Expansion index (C_e) values with increasing fly ash content for Bilkent Clay.....	158
Figure 4.75 : Mean resilient modulus (M_r) values with lime content without cure for Esenboğa Clay....	159
Figure 4.76 : Mean resilient modulus (M_r) values with lime content for 7 days cure period for Esenboğa Clay....	160
Figure 4.77 : Mean resilient modulus (M_r) values with lime content for 28 days cure period for Esenboğa Clay....	160
Figure 4.78 : Mean resilient modulus (M_r) values with lime content for 56 days cure period for Esenboğa Clay....	161
Figure 4.79 : Mean resilient modulus (M_r) values with lime content for 90 days cure period for Esenboğa Clay....	161
Figure 4.80 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.....	162
Figure 4.81 : Mean resilient modulus (M_r) values with lime content without cure for Bilkent Clay.....	163
Figure 4.82 : Mean resilient modulus (M_r) values with lime content for 7 days cure period for Bilkent Clay.....	164
Figure 4.83 : Mean resilient modulus (M_r) values with lime content for 28 days cure period for Bilkent Clay....	165
Figure 4.84 : Mean resilient modulus (M_r) values with lime content for 56 days cure period for Bilkent Clay....	165
Figure 4.85 : Mean resilient modulus (M_r) values with lime content for 90 days cure period for Bilkent Clay....	166
Figure 4.86 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Bilkent Clay....	167
Figure 4.87 : Resilient modulus (M_r) versus deviator stress with 14 kPa, 28 kPa and 42 kPa confining stresses (Natural Esenboğa Clay).....	168
Figure 4.88 : Resilient modulus (M_r) values versus deviator stress with 14 kPa, 28 kPa and 42 kPa confining stresses (Esenboğa Clay with 7% lime content).....	168

Figure 4.89 : Variation of Methylene Blue Value (MBV) with lime content for Esenboğa Clay.....	169
Figure 4.90 : Variation of Activity (A) with lime content for Esenboğa Clay.....	170
Figure 4.91 : Variation of Methylene Blue Value (MBV) with lime content for Bilkent Clay.....	171
Figure 4.92 : Variation of Activity (A) with lime content for Bilkent Clay.....	171
Figure 4.93 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content without cure for Esenboğa Clay.....	172
Figure 4.94 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 7 days cure period for Esenboğa Clay.....	173
Figure 4.95 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 28 days cure period for Esenboğa Clay.....	173
Figure 4.96 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 56 days cure period for Esenboğa Clay.....	174
Figure 4.97 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 90 days cure period for Esenboğa Clay.....	174
Figure 4.98 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.....	175
Figure 4.99 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content without cure for Bilkent Clay.....	176
Figure 4.100 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 7 days cure period for Bilkent Clay.....	177
Figure 4.101 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 28 days cure period for Bilkent Clay.....	177
Figure 4.102 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 56 days cure period for Bilkent Clay.....	178
Figure 4.103 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) with lime content for 90 days cure period for Bilkent Clay.....	178
Figure 4.104 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 7, 28, 56 and 90 days cure periods for Bilkent Clay.....	179
Figure 4.105 : SEM image taken at 350x magnification for natural Esenboğa Clay(1).....	180
Figure 4.106 : SEM image taken at 350x magnification for natural Esenboğa Clay(2).....	181
Figure 4.107 : SEM image taken at 2000x magnification for natural Esenboğa Clay(1).....	181
Figure 4.108 : SEM image taken at 2000x magnification for natural Esenboğa Clay(2).....	182
Figure 4.109 : SEM image taken at 5000x magnification for natural Esenboğa Clay(1).....	182
Figure 4.110 : SEM image taken at 5000x magnification for natural Esenboğa Clay(2).....	183
Figure 4.111 : Spectrum view for natural Esenboğa Clay.....	184
Figure 4.112 : Calcium mapping for natural Esenboğa Clay	185
Figure 4.113 : Chemical composition for natural Esenboğa Clay.....	186
Figure 4.114 : SEM image taken at 350x magnification for Lime-Treated Esenboğa Clay.....	187

Figure 4.115 : SEM image taken at 1000x magnification for Lime-Treated Esenboğa Clay.....	187
Figure 4.116 : SEM image taken at 2000x magnification for Lime-Treated Esenboğa Clay (Right in Figure 4.114).....	188
Figure 4.117 : SEM image taken at 2000x magnification for Lime-Treated Esenboğa Clay (Left in Figure 4.114).....	188
Figure 4.118 : SEM image taken at 5000x magnification for lime-Treated Esenboğa Clay (1).....	189
Figure 4.119 : SEM image taken at 5000x magnification for lime-Treated Esenboğa Clay (2).....	189
Figure 4.120 : Spectrum view for Lime-Treated Esenboğa Clay... ..	190
Figure 4.121 : Calcium mapping for Lime-Treated Esenboğa Clay	191
Figure 4.122 : Chemical composition for Lime-Treated Esenboğa Clay... ..	192
Figure 4.123 : SEM image taken at 350x magnification for natural Bilkent Clay(1).....	193
Figure 4.124 : SEM image taken at 350x magnification for natural Bilkent Clay(2).....	193
Figure 4.125 : SEM image taken at 2000x magnification for natural Bilkent Clay(1).....	194
Figure 4.126 : SEM image taken at 2000x magnification for natural Bilkent Clay(2).....	194
Figure 4.127 : SEM image taken at 5000x magnification for natural Bilkent Clay(1).....	195
Figure 4.128 : SEM image taken at 5000x magnification for natural Bilkent Clay(2).....	195
Figure 4.129 : First spectrum view for natural Bilkent Clay... ..	196
Figure 4.130 : Second spectrum view for natural Bilkent Clay.....	197
Figure 4.131 : First calcium mapping for Bilkent Clay	198
Figure 4.132 : Second calcium mapping for Bilkent Clay.....	199
Figure 4.133 : First chemical mapping for Bilkent Clay... ..	200
Figure 4.134 : Second chemical composition for Bilkent Clay... ..	201
Figure 5.1 : Comparison of liquid limits (LL) with lime content.... ..	204
Figure 5.2 : Comparison of plasticity indices (PI) with lime content.....	205
Figure 5.3 : Comparison of shrinkage limits (SL) with lime content.... ..	205
Figure 5.4 : Comparison of swell percentages with lime content.....	207
Figure 5.5 : Comparison of net swell percentages with lime content.....	207
Figure 5.6 : Comparison of effect of lime on swell percentages.... ..	208
Figure 5.7 : Comparison of swell pressures with lime content.....	208
Figure 5.8 : Comparison of net swell pressures with lime content.... ..	209
Figure 5.9 : Comparison of effect of lime on swell pressures.... ..	209
Figure 5.10 : Comparison of unconfined compressive strengths (q_u non-soaked) with lime content.... ..	211
Figure 5.11 : Comparison of unconfined compressive strengths for soaked condition (q_u soaked) with lime content.... ..	211
Figure 5.12 : Comparison of liquid limits (LL) with fly ash content.....	213
Figure 5.13 : Comparison of plasticity indices (PI) with fly ash content.... ..	214
Figure 5.14 : Comparison of shrinkage limits (SL) with fly ash content.....	214
Figure 5.15 : Comparison of swell percentages with fly ash content.... ..	216
Figure 5.16 : Comparison of net swell percentages with fly ash content.... ..	216
Figure 5.17 : Comparison of effect of fly ash content on swell percentages.....	217

Figure 5.18 : Comparison of swell pressures with fly ash content....	218
Figure 5.19 : Comparison of net swell pressures with fly ash content....	219
Figure 5.20 : Comparison of effect of fly ash content on swell pressures....	219
Figure 5.21 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) with fly ash content....	220
Figure 5.22 : Comparison of unconfined compressive strengths of soaked samples ($q_{u \text{ soaked}}$) with fly ash content....	221
Figure 5.23 : Comparison of mean resilient moduli (M_r) values with lime content without cure....	222
Figure 5.24 : Comparison of mean resilient moduli (M_r) values with lime content for 7 days cure period....	223
Figure 5.25 : Comparison of mean resilient moduli (M_r) values with lime content for 28 days cure period....	223
Figure 5.26 : Comparison of mean resilient moduli (M_r) values with lime content for 56 days cure period....	224
Figure 5.27 : Comparison of mean resilient moduli (M_r) values with lime content for 90 days cure period....	224
Figure 5.28 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay....	225
Figure 5.29 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Bilkent Clay....	226
Figure 5.30 : Comparison of methylene blue value (MBV) with lime content....	227
Figure 5.31 : Comparison of activity (A) with lime content....	228
Figure 5.32 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content without cure....	229
Figure 5.33 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 7 days cure period....	229
Figure 5.34 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 28 days cure period....	230
Figure 5.35 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 56 days cure period....	230
Figure 5.36 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 90 days cure period....	231
Figure B.1 : XRD analysis results for Esenboğa Clay (1)....	254
Figure B.2 : XRD analysis results for Esenboğa Clay (2)....	255
Figure B.3 : XRD analysis results for Esenboğa Clay (3)....	256
Figure B.4 : X-Ray diffractograms of Esenboğa Clay for bulk sample....	257
Figure B.5 : XRD patterns for clay fraction in Esenboğa Clay (1)....	257
Figure B.6 : XRD patterns for clay fraction in Esenboğa Clay (2)....	257
Figure B.7 : XRD patterns for clay fraction in Esenboğa Clay (3)....	258
Figure B.8 : XRD patterns for clay fraction in Esenboğa Clay (4)....	258
Figure B.9 : XRD analysis results for Bilkent Clay (1)....	259
Figure B.10 : XRD analysis results for Bilkent Clay (2)....	260
Figure B.11 : XRD analysis results for Bilkent Clay (3)....	261
Figure B.12 : X-Ray diffractograms of Bilkent Clay for bulk sample....	262
Figure B.13 : XRD patterns for clay fraction in Bilkent Clay (1)....	262
Figure B.14 : XRD patterns for clay fraction in Bilkent Clay (2)....	262
Figure B.15 : XRD patterns for clay fraction in Bilkent Clay (3)....	263
Figure B.16 : XRD patterns for clay fraction in Bilkent Clay (4)....	263
Figure C.1 : XRF analysis results....	264

Figure C.2 : XRF analysis results for Esenboğa Clay (1).....	265
Figure C.3 : XRF analysis results for Esenboğa Clay (2).....	266
Figure C.4 : XRF analysis results for Esenboğa Clay (3).....	267
Figure C.5 : XRF analysis results for Esenboğa Clay (4).....	268
Figure C.6 : XRF analysis results for Bilkent Clay (1)...	269
Figure C.7 : XRF analysis results for Bilkent Clay (2)...	270
Figure C.8 : XRF analysis results for Bilkent Clay (3)...	271
Figure C.9 : XRF analysis results for lime (1).....	272
Figure C.10 : XRF analysis results for lime (2).....	273
Figure C.11 : XRF analysis results for lime (3).....	274
Figure C.12 : XRF analysis results for fly ash (1).....	275
Figure C.13 : XRF analysis results for fly ash (2).....	276
Figure C.14 : XRF analysis results for fly ash (3).....	277
Figure C.15 : XRF analysis results for fly ash (4).....	278
Figure C.16 : XRF analysis results for fly ash (5).....	279
Figure C.17 : XRF analysis results for fly ash (6).....	280
Figure C.18 : XRF analysis results for fly ash (7).....	281
Figure C.19 : XRF analysis results for fly ash (8).....	282
Figure C.20 : XRF analysis results for fly ash (9).....	283
Figure C.21 : XRF analysis results for fly ash (10).....	284
Figure C.22 : XRF analysis results for fly ash (11).....	285
Figure D.1 : e- σ' relationship for natural and lime-treated soil samples without cure for Esenboğa Clay.....	286
Figure D.2 : e-log σ' relationship for natural and lime-treated soil samples without cure for Esenboğa Clay.....	287
Figure D.3 : e- σ' relationship for natural and lime-treated soil samples for 7 days cure period for Esenboğa Clay..	288
Figure D.4 : e-log σ' relationship for natural and lime-treated soil samples for 7 days cure period for Esenboğa Clay..	289
Figure D.5 : e- σ' relationship for natural and lime-treated soil samples for 28 days cure period for Esenboğa Clay..	290
Figure D.6 : e-log σ' relationship for natural and lime-treated soil samples for 28 days cure period for Esenboğa Clay..	291
Figure D.7 : e- σ' relationship for natural and lime-treated soil samples for 56 days cure period for Esenboğa Clay..	292
Figure D.8 : e-log σ' relationship for natural and lime-treated soil samples for 56 days cure period for Esenboğa Clay..	293
Figure D.9 : e- σ' relationship for natural and lime-treated soil samples for 90 days cure period for Esenboğa Clay..	294
Figure D.10 : e-log σ' relationship for natural and lime-treated soil samples for 90 days cure period for Esenboğa Clay..	295



STATIC AND CYCLIC PROPERTIES OF EXPANSIVE CLAYS TREATED WITH LIME AND FLY ASH WITH SPECIAL REFERENCE TO SWELLING AND RESILIENT MODULI

SUMMARY

Soils with high plasticity and high swell potential have great volume changes due to water content fluctuations in an unstable manner. As a result of this expansion, the pavements and/or lightweight structures, which are settled on that kind of subsoils, are damaged. Expansive soils can be improved by adding chemical stabilizers such as lime, fly ash, micro cement, etc.

The resilient modulus value, on the other hand, provides a basic relation between the stress and strain of the material in order to perform the structural analysis of the layered pavement systems under cyclic conditions.

Lime and fly ash are the most popular agents used in chemical stabilization method. However, it is not very clear whether these agents are effective enough or not effective at all in very highly or extremely highly plastic clays. Moreover, swell and shrinkage characteristics of the lime and fly ash stabilized soils are not well-known when compared to the strength behavior of the treated soils. Another not well-known aspect of the subject is cyclic loading properties of the native expansive soils and stabilized soil-agent mixtures. It is believed that the resilient modulus properties are needed to be investigated in this context. Finally, the effect of mineralogical composition of the clay soils and chemical components of the agents are believed to be investigated as far as the swelling is concerned. The clays and the lime and fly ash agents used in this research are identified by means of Rietveld, XRD and XRF tests. SEM analyses have also been performed.

In this research, static and cyclic loading properties of the two natural clays, which are extremely highly and very highly plastic expansive soils, are investigated by utilizing lime and fly ash agents for stabilization. Lime has been considered as chemically active additive. Fly ash, on the other hand, is considered as filler material and its experimental results can be accepted as reference values to compare them with the results of lime. Static research has been executed with special reference to swelling behavior. Resilient modulus properties have been examined under variable stress states within the scope of dynamic research.

Two different soil samples, which are taken from different sites of Ankara, have been investigated. They are namely, Esenboğa and Bilkent Clays. Esenboğa Clay is an extremely highly plastic clay (PI=101%). Bilkent Clay is a very highly plastic clay (PI=76%). Both disturbed and undisturbed soil samples have been taken from both sites.

Firstly, reference tests (index etc.) have been carried out on untreated natural soil samples. Then, both clays are mixed with the lime and fly ash agents separately at

different percentages of dry weight of the soil (i.e. 1%, 3%, 5%, 7% and 9% for lime, and 5%, 10%, 15%, 20% and 25% for fly ash).

Methylene blue tests have been performed on untreated and lime-treated Esenboğa and Bilkent Clays for each percentage.

The oedometer tests have been performed to determine the swelling and compressibility characteristics of the soil samples. The unconfined compression tests have been executed for both non-soaked and soaked conditions. These static tests have also been performed for Esenboğa Clay with lime agent at each percentages prepared for 7, 28 and 56-days curing periods.

The Esenboğa and Bilkent Clays, which are mixed with the lime agent at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%), have been tested for the determination of resilient modulus values. Resilient moduli have also been found for soil samples prepared for 7, 28, 56 and 90-days curing periods in each lime percentages.

SEM analyses have been performed for 7% lime-treated Esenboğa Clay and also for natural Esenboğa and Bilkent Clays.

Optimum values have been evaluated for swell potential and compressive strength. The correlations between the agent percentages and index properties, the swell percentage, the swell pressure, time effect, compressive strength and resilient modulus are presented.

As a result of mineralogic analyses of the soil samples, it is understood that the Esenboğa Clay is composed of mainly illite (about 87%). However, it is determined that the amount of smectite minerals are not negligible (about 4%). It is interesting to note, on the other hand, that the Bilkent Clay is completely composed of palygorskite (attapulgite) mineral.

The optimum lime content is about 3% as far as the swelling is concerned in Esenboğa Clay. In a design of sub-base of surficial structures, this optimum lime content can be taken as a reference. Swell tests should be executed with this reference value. In case of not satisfying the design criteria, this optimum lime content should be increased. Laboratory swell percentage tests show that about 8% lime content for Esenboğa Clay may just be sufficient for design criteria according to Highway Standards in Türkiye. Laboratory tests show, on the other hand, that the fly ash agent may not be sufficient alone for swelling design criteria in such type of extremely and/or very highly plastic clays. Lime agent effect on swelling behavior, therefore, is much more effective than the effect of fly ash agent. Swell pressures are about 6.5 times greater in fly ash improvement with 20 to 25% amounts than the pressures found in lime improvement with 7 to 9% amounts in Esenboğa Clay.

When the variation of unconfined compressive strength with lime content for non-soaked and soaked conditions is considered without curing, it is noticed that the unconfined compressive strengths are greater in non-soaked condition up to 5% lime content. Starting from 7% however, the strengths are greater in soaked condition due to self-hardening mechanism which can be called also as pozzolanic reactions.

Time effect of lime stabilization in the extremely highly plastic Esenboğa Clay has been investigated in this research. For the curing period of 28 days, 5% lime content is the optimum content at which the swell percentage is decreased to null in Esenboğa Clay. This result is also valid in swell pressure tests.

Soaked and non-soaked samples have been tested to get unconfined compressive strengths in Esenboğa Clay with different curing periods. It has been concluded that the unconfined compressive strengths increase dramatically as compared to non-cured strengths for all cure periods especially for non-soaked samples. 28-day curing period can be accepted as optimum. Unconfined compressive strengths of soaked samples are always lower than the ones in non-soaked condition in all lime contents for curing periods of 7, 28 and 56 days.

The optimum lime and fly ash contents for Bilkent Clay are about 3% and 10% respectively as far as the swelling is concerned. Since lime is more effective than the fly ash, it is highly probable that the design criteria are to be satisfied at reasonable lime contents. Laboratory swell percentage tests show that 9% to 10% lime content for Bilkent Clay may be sufficient in a transportation sub-structure design as far as the swelling criteria are concerned. Laboratory tests show that the fly ash agent may not be sufficient alone for swelling design criteria in such type of very highly plastic clays. The effectiveness of lime agent on swell behavior can also be observed in swell pressure tests in Bilkent Clay. Swell pressures are about 2.7 times greater in fly ash improvement with 20 to 25% amounts than the pressures found in lime improvement with 7% to 9% amounts in Bilkent Clay.

The methylene blue test results obtained are compatible with the results mentioned above. That is to say, the swell pressure and the swell percentages along with the methylene blue (MBV) and activity values decrease with the increase in lime content.

When the variation in the mean resilient modulus values are considered for the extremely highly plastic Esenboğa Clay, 3% lime content seems quite effective for both non-cured and cured samples. 56-days curing period gives maximum mean resilient modulus values in this clay. In Bilkent Clay which is very highly plastic, 3% lime content can be accepted effective at early stages (i.e. at non cured and 7 days cured conditions) as far as the mean resilient modulus values are concerned. However, 7% lime content gives generally maximum increases in resilient modulus values at later cure periods of 28, 56 and 90 days. 90-days curing period gives always maximum resilient modulus value in the tests for Bilkent Clay. It should be noticed here that the Bilkent Clay is composed of 100% palygorskite clay mineral which may influence the results.

The results achieved in this research show that lime agent alone can be utilized quite effectively in the extremely highly and/or very highly plastic expansive clays obtained from two different construction sites in Ankara, especially on the swelling properties. However, this result is not valid for the agent fly ash which is an industrial waste. It is recommended to use fly ash with a calcium-based agent like lime or cement to improve such type of very highly plastic expansive clays in terms of swelling criteria since utilizing fly ash especially is quite advantageous from an economical and environmental points of view.

Keywords: *expansive clay, lime, fly ash, treatment, swell potential, curing time, unconfined compressive strength, resilient modulus*



KİREÇ VE UÇUCU KİL İLE İYİLEŞTİRİLMİŞ ŞİŞEBİLEN KİLLERİN ŞİŞME VE ESNEKLİK MODÜLÜ ODAKLI OLARAK STATİK VE TEKRARLI YÜKLER ALTINDAKİ ÖZELLİKLERİ

ÖZET

Yüksek plastisiteli ve yüksek şişme potansiyeline sahip zeminlerde, su içeriğindeki kararsız dalgalanmalar büyük hacim değişimlerine sebep olmaktadır. Bu hacimsel değişime bağlı şişmeler sonucunda bu tür zeminlere oturan ulaşım amaçlı yol üstyapı kaplamaları ve/veya hafif ağırlıklı yapılar zarar görmektedir. Şişen zeminler, kireç, uçucu kül, mikro çimento ve benzeri katkıları yardımıyla iyileştirilebilirler.

Esneklik modülü değeri, tabakalı yol kaplama sistemlerinin yapısal analizlerinde kullanılmak üzere, dinamik yükler altındaki malzemenin gerilme ve birim deformasyon arasındaki temel ilişkiyi sağlar.

Kireç ve uçucu kül katkıları kimyasal iyileştirme metodu kapsamında kullanılan en popüler katkılardır. Ancak bu katkıların aşırı derecede veya çok yüksek derecede plastik killerde yeterli bir şekilde verimli oldukları veya hiç verimli olamadıkları gibi hususlar çok açık değildir. Ayrıca kireç ve uçucu kül ile iyileştirilmiş zeminlerin şişme ve büzülme açısından özellikleri mukavemet özellikleri kadar iyi bilinmemektedir. Diğer çok bilinmeyen bir konu doğal şişebilen killerin ve değişik katkıları ile iyileştirilmiş zeminlerin tekrarlı yükler altındaki özellikleridir. Bu kapsamda esneklik modülü özelliklerinin araştırılmasının gereğine inanılmaktadır. Son olarak, şişme davranışı göz önüne alındığında kil zeminlerin mineralojik içerik ve katkı maddelerinin kimyasal bileşenlerinin araştırılması gereğine inanılmaktadır. Bu kapsamda, bu çalışmada kullanılan hem kil zeminler hem de kireç ve uçucu kül katkıları Rietveld, XRD ve XRF testleri ile tanımlanmıştır.

Bu çalışmada, aşırı ve çok yüksek plastisiteli iki doğal kil numunenin kireç ve uçucu kül katkıları kullanılarak iyileştirilmesi ile, statik ve tekrarlı yükler altındaki özelliklerinin incelenmesi hedeflenmiştir. Kireç, kimyasal olarak aktif katkı olarak gözönüne alınmıştır. Diğer taraftan uçucu kül, dolgu malzemesi olarak düşünülmekte ve uçucu kül laboratuvar sonuçları, kil ile elde edilen sonuçlar ile bir karşılaştırma yapmak için referans sonuçlar olarak kabul edilebilir. Araştırmanın statik kısmında özellikle şişme davranışı üzerinde odaklanılmıştır. Esneklik modülü deneyleri ise farklı gerilme durumlarında dinamik araştırma amaçlı yürütülmüştür.

Bu çalışmada, Ankara’da iki ayrı sahadan alınan Esenboğa Kili ve Bilkent Kili olarak adlandırılan farklı iki zemin numunesi detaylı olarak incelenmiştir. Esenboğa Kili aşırı derecede yüksek plastisiteli bir kildir ($PI=101$). Bilkent Kili ise çok yüksek derecede plastisiteli bir kildir ($PI=76$). Her iki sahadan da laboratuvar deneyleri için örselenmiş ve örselenmemiş numuneler alınmıştır.

İlk olarak, doğal numuneler üzerinde referans deneyler (indeks v.b.) yapılmıştır. Sonra, her iki kile ayrı ayrı, kuru ağırlıklarının farklı yüzdelğinde katkı maddeleri

eklenmiştir (kireç için: %1, %3, 5%, 7% ve 9%; uçucu kil için: %5, %10, %15, %20 ve %25).

Metilen Mavisi deneyleri, her iki kilin doğal ve her yüzde için kireç ile iyileştirilmiş numunelerin aktivitelerinin belirlenmesi için yapılmıştır.

Ödometre deneyleri ile, doğal ve iyileştirilmiş zeminlerin, şişme ve sıkışma özellikleri belirlenmiştir. Serbest basınç deneyi ise hem suya boğulmamış (batırılmamış) (optimum su içeriğinde) hem de suya boğulmuş (batırılmış) numuneler üzerinde yapılmıştır. Esenboğa Kili için bu statik deneyler her bir kireç yüzdesinde 7, 28 ve 56 günlük kür süreleri için de yürütülmüştür.

Esenboğa ve Bilkent Killeri için, kireç katkısı ile, her bir yüzde (%1, %3, 5%, 7% ve 9%) ve değişik kür süreleri (7, 28, 56 ve 90 gün) için hazırlanan numunelerin esneklik modülü değerleri saptanmıştır.

Esenboğa Kil numunenin (sadece %7 kireç katkısı için) SEM analizleri yapılmıştır. SEM analizleri doğal Esenboğa ve Bilkent killeri için de yürütülmüştür.

İyileştirilen numunelerin indeks, mukavemet ve şişme özellikleri gibi mühendislik özellikleri bulunmuştur. Şişme potansiyeli ve basınç dayanımlarında iyileşmenin görüldüğü optimum değerler bulunmuştur. Katkı yüzdeleri ve indeks özellikleri, şişme yüzdeleri, şişme basınçları, kür süresi etkisi, basınç dayanımı ve esneklik modülü arasındaki korelasyonlar tezde sunulmuştur.

Zemin numunelerinin mineralojik analizi sonucunda, Esenboğa Kilinin büyük oranda illit mineralinden oluştuğu tespit edilmiştir (%87). Ancak, içeriğindeki smektit minerallerinin de ihmal edilemeyecek miktarda olduğu görülmüştür (%4). Öte yandan, Bilkent Kilinin tamamının paligorskit kil mineralinden oluştuğu ortaya çıkmıştır.

Esenboğa Kili için şişme dikkate alındığında, optimum kireç miktarı yaklaşık %3'tür. Yüzeysel yapıların (otayol, demiryolu vb.) alt temel tasarımlarında bu optimum değer referans olarak kabul edilebilir. Bu referans değerle şişme deneyine başlanarak, bu değerlerin tasarım kriterlerini sağlamaması durumunda referans değer artırılabilir. Laboratuvarda yapılan şişme yüzdesi deneyleri, Esenboğa Kili için yaklaşık %8 kireç içeriğinin Türkiye'deki Karayolları Standartlarına göre ancak yeterli olduğunu göstermektedir. Diğer yandan, laboratuvar deneyleri uçucu kül katkısının bu tür aşırı yüksek plastisiteli ya da çok yüksek plastisiteli zeminlerde tek başına yeterli olmayabileceğini göstermektedir. Esenboğa Kili'nde, şişme basınçları %20-%25 uçucu kül katkısı eklenen numunelerde, %7-%9 kireç eklenen numunelere göre yaklaşık 6.5 kat daha fazladır.

Optimum su içeriğinde (suya boğulmamış) ve suya boğulmuş şekilde hazırlanan katkılı numunelerin serbest basınç dayanımları dikkate alındığında ise optimum su içeriğinde hazırlanan numunelerde serbest basınç dayanımları %5 kireç içeriğine kadar suya boğulmuş numunelere göre daha yüksektir. Suya boğulan numunelerin dayanımları özellikle %7'den başlayarak ortamda suyun varlığı ile gerçekleşen sertlik kazanma ya da puzolanik olarak adlandırdığımız reaksiyonlara bağlı olarak optimumda hazırlanan numunelerin dayanımını geçmektedir.

Bu araştırmada kür sürelerinin, farklı oranlarda kireç ile iyileştirilen aşırı yüksek plastisiteli Esenboğa Kili'ne etkisi araştırılmıştır. Esenboğa Kilinde, 28 ve 56 günlük kür sürelerinde şişme yüzdesinin sıfırlara düştüğü optimum kireç yüzdesi %5'tir. Bu durumda %5 kireç miktarında 28 günlük kür süresi izin verilebilir şişme yüzdesinin sağlanabilmesi için oldukça yeterlidir. Benzer sonuç şişme basıncı için de geçerlidir.

Optimum su içeriğinde (suya boğulmamış) ve suya boğulmuş şekilde hazırlanan doğal ve katkılı numunelerde serbest basınç deneyi yapılmıştır. Özellikle optimum su içeriğinde hazırlanan numunelerde serbest basınç dayanımlarında, kürlü numunelerde kürsüz numunelere göre oldukça büyük artışlar dikkat çekmiştir. Kür süresi açısından 28 günlük periyot optimum kabul edilebilir. Farklı kireç içeriklerinde 7, 28 ve 56 gün kür süreleri için hazırlanan tüm numunelerde, suya boğulan numunelerin serbest basınç dayanımlarının optimum su içeriğinde hazırlanan numunelere göre her zaman daha düşük olduğu görülmüştür.

Şişme söz konusu olduğunda, Bilkent Kili için optimum kireç ve uçucu kül içerikleri sırası ile yaklaşık %3 ve %10 olarak bulunmuştur. Kireç uçucu küle nazaran daha etkili olduğundan, uygun miktarlardaki kireç içeriği tasarım kriterlerini sağlamada büyük ihtimalle tek başına yeterli olacaktır. Laboratuvar deneyleri göstermektedir ki; şişme dikkate alındığında Bilkent Kili'nde şişme kriterleri %9-%10 kireç içeriğinde sağlanmaktadır. Öte yandan, laboratuvar deneylerinde, bu tür çok yüksek plastisiteli killerde şişme özellikleri açısından tasarım kriterlerini sağlamada uçucu külün tek başına yeterli bir katkı olmayabileceği saptanmıştır. Kireç katkısının şişme davranışı üzerindeki etkisi şişme basıncı deneylerinde de gözlemlenmiştir. Bilkent Kili'nde elde edilen şişme basınçları %20-%25 uçucu kül katkısı eklenen numunelerde %7-%9 kireç eklenen numunelere göre yaklaşık 2.7 kat daha fazladır.

Metilen mavisi test sonuçları yukarıdaki sonuçlar ile uyumludur. Şöyle ki, kireç miktarının artmasına bağlı olarak şişme basınç ve yüzdeleri azaldığı gibi aynı şekilde metilen mavisi değerleri (MBV) ve aktivite de azalmaktadır.

Ortalama esneklik modülü değerlerine bakıldığında, Esenboğa Kili için, %3 kireç içeriğinin hem kürsüz hem de kürlü numuneler için oldukça verimli olduğu görülmüştür. Bu kil için 56 günlük kür zamanında maksimum ortalama esneklik modülleri elde edilmiştir. Bilkent Kili'nde kürsüz ve 7 gün kürlü numuneler için %3 kireç içeriği verimli kabul edilebilir. Ancak 28, 56 ve 90 kür süreleri için %7 kireç içeriği genelde en yüksek esneklik modülü artışlarını göstermektedir. 90 gün kür süresinde her zaman en yüksek modül değerleri elde edilmiştir. Bilkent Kili'nin %100 palikorskit mineralinden oluşmasının sonuçları etkilediği düşünülmektedir.

Bu araştırmada elde edilen sonuçlar, sönmüş kireç katkısının, Ankara'da iki farklı sahadan alınan aşırı derecede yüksek (Esenboğa Kili) ve çok yüksek (Bilkent Kili) plastisiteli şişen killerin iyileştirilmesinde özellikle şişme özellikleri dikkate alındığında tek başına oldukça etkili olduğunu göstermektedir. Ancak bu sonucun endüstriyel bir atık olan uçucu kül için geçerli olmadığı görülmüştür. Bu sebeptendir ki uçucu külün bu tür aşırı derecede yüksek plastisiteli şişen killerin şişmeye bağlı tasarım kriterlerini sağlayabilmesi için, kireç ve/veya çimento gibi kalsiyum bazlı başka katkılarla beraber kullanılması önerilmektedir. Endüstriyel bir atık olan uçucu külün zemin iyileştirmesinde kullanılarak değerlendirilmesi özellikle ekonomik ve çevresel açılardan oldukça avantajlıdır.

Anahtar kelimeler: şişen killer, kireç, uçucu kül, iyileştirme, şişme potansiyeli, kür süresi, serbest basınç dayanımı, esneklik modülü



1. INTRODUCTION

Expansive soils expand when water is added, and shrink when they dry out (Chen, 1988, Nelson et al., 2015). These uneven changes in soil volume can distress and disturb the light-weight buildings especially constructed on such soil soils resulting in foundation and structural cracking. Expansive soils can also cause distortion, twisting and cracking of highways, railways, pavements, pipeline routes and ground floor slabs. It is now well-known that repair costs in such problems are to be billion dollars annually in many parts of the World.

There are many design alternatives of foundations to cope with the swelling problem of expansive soils. Two general categories can be stated in this context: structural alternatives and soil treatment alternatives. These alternatives range from simple moisture control precautions to special structural designs like stiffened shallow foundations and deep foundation systems. However, it is known that it is virtually impossible to prevent an increase in water content of the foundation soils by using moisture control alternatives (Nelson and Miller, 1992). Structural alternatives, on the other hand, are generally not economical. Thus, soil replacement and chemical treatment methods to eliminate the swelling problem are frequently chosen as a preferable design alternative. The advantage of chemical treatment methods is such that the soil is treated in-situ instead of replacing the large volume of expansive soils by non-expansive soils.

To begin with, literature review shows that there is a lack of comparative studies on microstructure and mineralogy of expansive soils stabilized with various stabilizers. Therefore, it is believed that a research is needed to better understand the microstructure and mineralogy of soils stabilized with locally available additives. Thus, chemical stabilization by lime and fly ash is the research subject of this thesis.

Aim of the study:

The main purpose of this study is to determine the lime and fly ash stabilization effect on very and extremely highly plastic expansive soils with special reference to swell

behavior and resilient modulus properties. It is not very clear whether these agents are effective enough or not effective at all in very highly or extremely highly plastic clays. Moreover, swell and shrinkage characteristics of the lime and fly ash stabilized soils are not well-known when compared to the strength behavior of the treated soils. Another not well-known aspect of the subject is cyclic loading properties of the native expansive soils and stabilized soil-agent mixtures. Resilient modulus properties have been examined within the scope of this research. Finally, the effect of mineralogical composition of the clay soils and chemical components of the agents are believed to be investigated as far as the swelling is concerned. In this context, both treated soils and the lime and fly ash agents used in this research are identified by means of Rietveld, XRD and XRF tests.

Laboratory tests have been performed to examine the index (including methylene blue properties), swelling, strength (for both soaked and non-soaked conditions), compressibility, and the resilient modulus properties of all native soils and soil-agent mixtures with different proportions. Curing effect has also been investigated for different periods (i.e. 7 days, 28 days, 56 days and 90 days).

Finally, SEM analyses have been carried out to support the laboratory findings; thus the treatment mechanism can be explained in a detailed manner.

Scope of the study:

Within the scope of this research, two different soil samples which have been taken from different sites of Ankara, are investigated. They are namely, Esenboğa and Bilkent Clays. Esenboğa Clay is an extremely highly plastic clay (Plasticity index, $PI=101\%$). The clay samples have been taken from clay deposits in the Akyurt district of Ankara, near Esenboğa Airport. The region is also known as bentonite deposits of Ankara.

Bilkent Clay is a very highly plastic clay ($PI=76\%$). Bilkent Clay samples have been taken from clay deposits of the Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway. Both disturbed and undisturbed soil samples have been taken from both sites in order to perform laboratory tests. Hydrated lime and fly ash have been used as chemical stabilizers.

Chapter 2, begins with a literature survey on the explanation of the swell mechanism including clay mineralogy and classification of expansive soils. These are followed by

the explanation of chemical stabilization methods by lime and fly ash. Then, cyclic loading properties of treated soils based on resilient modulus test are explained. Finally, in Chapter 2, past studies given in literature are presented.

In Chapter 3, laboratory experimental schedule and obtaining of clay soil samples from different sites are explained. Also, mineralogy of the clay samples and the properties of chemical agents used in this research are presented. Besides, preparation of the test specimens, description of the test apparatus and testing procedure for index, strength and oedometer tests are given. Moreover, the details for resilient modulus tests and methylene blue tests and SEM analyses are presented.

Chapter 4 involves the test results and explanation of the behavioral trends and the optimum values for improvement.

Chapter 5 deals with the discussion of the corresponding test results. In this part of the thesis, interpretation of the test results with comprehensive comparisons are given.

Finally, general overview of the research and the major conclusions and recommendations for future research are presented in Chapter 6.



2. LITERATURE SURVEY

2.1 General

Expansive soils can experience great changes in volume when their water content varies. These types of soil are widely distributed throughout the world (Huang and Wu, 2007, Seco et al., 2011). Swelling clay minerals make soils expansive. As they get wet, the clay minerals absorb water molecules and expand. The soils can even undergo loss of shear strength. Conversely, as they dry, they shrink leaving large voids in the soil. Particularly light buildings and pavements (highways etc.) are frequently damaged due to expansive soils. Some examples of damages are given in the following figures (From Figure 2.1 to 2.4). The heave of concrete slabs resulting from the presence of expansive soil is shown in Figure 2.1 (URL-2).



Figure 2.1 : Heave of concrete slabs adjacent to industrial building.

The cracks on the wall of a building due to upward soils pressures is shown in Figure 2.2 (Zumrawi et al., 2017).



Figure 2.2 : Cracks on wall of a building due to upward soil pressures.

The damage at a ground floor of a building in the city San Angelo of U.S.A. can be seen in Figure 2.3 (URL-4).



Figure 2.3 : Damage at a ground floor due to expansive soil.

The longitudinal cracks on an asphalt road due to swelling phenomenon is shown in Figure 2.4 (URL-3).



Figure 2.4 : Asphalt road damage due to swelling soils.

Expansive soil problems exist on every continent, with the exception perhaps of Antarctica (Nelson et al., 2015). The countries like United States, Canada, India, Australia and Sudan have very large lands and therefore have many examples of expansive soil phenomena in construction activities in their semi-arid regions especially (Coduto, 2001). The expansive clays in Turkey are frequently observed in Middle Anatolia and some regions of West Anatolia. Ankara, Tokat, Edirne, Kütahya and Eskişehir are some examples of cities where expansive clays are encountered. It is quite common in Gölbaşı, Çankaya and Akyurt districts of Ankara where the damages in light structures due to expansive clays originate.

2.2 Soil Mineralogy

Mineralogy is the primary factor controlling the size, shape, and properties of soil particles. These same factors determine the possible ranges of physical and chemical properties of any given soil; therefore, a priori knowledge of what materials are in a soil provides intuitive insight as to its behavior (Mitchell and Soga, 2005).

Soils can be classified first according to their particle sizes. Gravel, sand, silt and clay soils range from coarse size to fine respectively. The soils having particles sizes more than 5mm are gravels. The particle sizes for sands are between 5mm and 0.074mm. Particles smaller than about 200 mesh sieve size (0.074mm), which is the boundary

between sand and silt sizes, cannot be seen by the naked eye. Clays refer to all constituents of a soil smaller than a particular size, usually 0.002mm (2 μ m) in engineering classifications. Clay term can also refer to a class of minerals in literature. Clay minerals are primarily hydrous aluminum silicates. An important difference between clay and nonclay minerals is that the nonclays are composed of primarily of bulky particles; whereas, the particles of most of the clay minerals are platy, in a few cases they are needle shaped or tubular.

2.2.1 Cohesionless soils

The physical characteristics of cohesionless soils, that is, gravel, sand, and nonplastic silts, are determined primarily by particle size, shape, surface texture, and size distribution. The mineral composition determines hardness, cleavage, and resistance to physical and chemical breakdown (Mitchell and Soga, 2005).

The gravel, sand, and most of the silt fraction in a soil are composed of bulky, nonclay particles which are predominantly rock fragments or mineral grains of the common rock-forming minerals. In igneous rocks, which are the original source material for many soils, the most prevalent minerals are the feldspars (about 60%), the pyroxenes and amphiboles (about 17%), quartz (12%) and micas (4%). In most soils, however, quartz is by far the most abundant mineral. Feldspar and mica also present with small amounts.

2.2.2 Clay minerals

Clay minerals in soils belong to the mineral family termed phyllosilicates, which also contains other layer silicates such as serpentine, pyrophyllite, talc, mica and chlorite. The structures of the common layer silicates are made up of combinations of two simple structural units, the silicon tetrahedron (Figure 2.5) and the aluminium or magnesium octahedron (Figure 2.6).

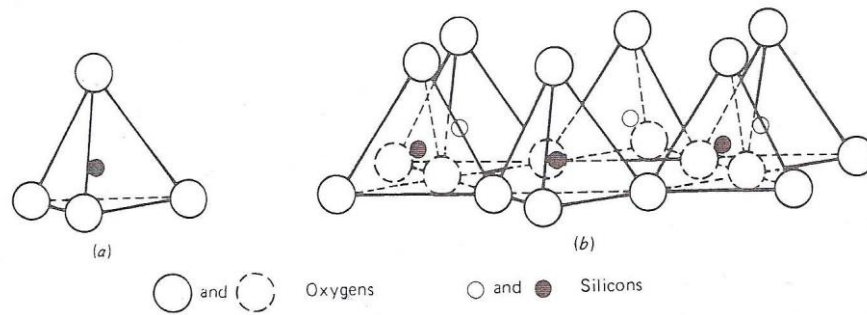


Figure 2.5 : Silicon tetrahedron and silica tetrahedra arranged in a hexagonal network (Mitchell and Soga, 2005).

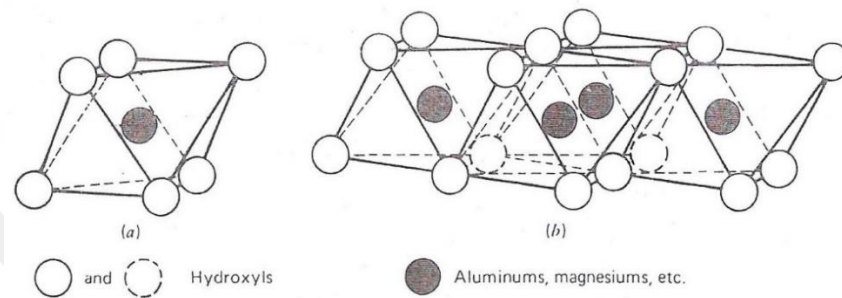


Figure 2.6 : Octahedral unit and sheet structure of octahedral units (Mitchell and Soga, 2005).

There are valency imbalances in both units in the above figures, resulting in net negative charges. The basic units, therefore, do not exist in isolation but combine to form sheet structures. The tetrahedral units combine by the sharing of oxygen ions to form a silica sheet. The octahedral units combine through shared hydroxyl ions to form a gibbsite sheet. The silica sheet retains a net negative charge but the gibbsite sheet is electrically neutral. Silicon and aluminum may be partially replaced by other elements, this being known as isomorphous substitution, resulting in further charge imbalance (Craig, 2004).

The manner in which atoms are assembled into tetrahedral and octahedral units, followed by the formation of sheet and their stacking to form layers that combine to produce the different clay mineral groups is illustrated in Figure 2.7.

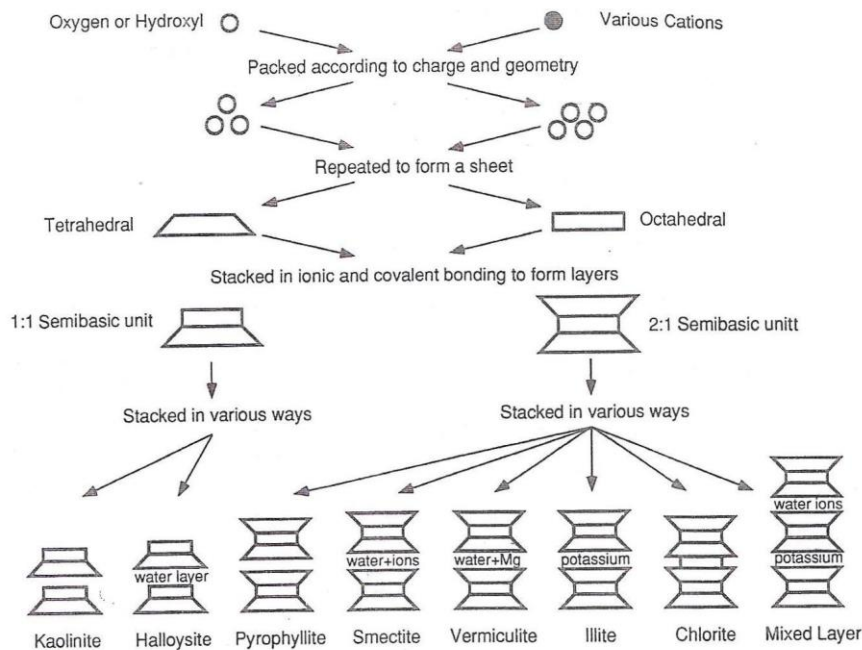


Figure 2.7 : Synthesis pattern for the clay minerals (Mitchell and Soga, 2005).

The basic structures shown in the bottom row of Figure 2.7 comprise the great majority of the clay mineral types that are found in soils. The principal clay minerals in Figure 2.7 are kaolinite, illite and montmorillonite (smectite group).

Kaolinite consists of a structure based on a single sheet of silica combined with a single sheet of gibbsite. There is very limited isomorphous substitution. The combined silica-gibbsite sheets are held together relatively strongly by hydrogen bonding.

Illite has a basic structure consisting of a sheet of gibbsite between and combined with two sheets of silica. In the silica sheet there is partial substitution of silicon by aluminium. The combined sheets are linked together by relatively weak bonding due to non-exchangeable potassium ions held between them.

Montmorillonite has the same basic structure as illite. In the gibbsite sheet there is partial substitution of aluminium by magnesium and iron, and in the silica sheet there is again partial substitution of silicon by aluminium. The space between the combined sheets is occupied by the water molecules and exchangeable cations other than potassium, resulting in a very weak bond.

Considerable swelling of montmorillonite can occur due to additional water being adsorbed between the combined sheets (Craig, 2004).

Illites and especially kaolinites are, on the other hand, are infrequently expansive (Nelson and Miller, 1992). However, especially illite mineral can cause volume changes when particle sizes are extremely fine (less than a few tenths of a micron) (Mitchell and Raad, 1973, Snethen et al., 1977).

Palygorskite (attapulgitite) is another mineral but not as common as other clay minerals. The name specified by the International Nomenclature Committee is palygorskite. However, the name attapulgitite is so well established in trade circles that it continues to be used by many producers and users. While commercial deposits of it is rare, this clay mineral occurs relatively frequently in sediments and soils. The main producers in the World are U.S.A., Senegal, Spain, Australia, India and Turkey (Galan, 1996).

Palygorskite is a fibrous authigenic clay mineral with the chemical composition of $(\text{Mg,Al})_2\text{Si}_4\text{O}_{10}(\text{OH}) \cdot 4(\text{H}_2\text{O})$. Palygorskite is a phyllosilicate in as much as it contains a continuous two-dimensional tetrahedral sheet; however, it differs from other layer silicates in that it lacks continuous octahedral sheets (Galan, 1996).

Besides other expandable clay minerals, such as smectite and illite-smectite which occur primarily within detrital clay throughout the Neogene succession, palygorskite is also believed to have some swelling potential (Mishra et al., 2012).

2.2.2.1 XRD and Rietveld tests

X-Ray diffraction (XRD) patterns are utilized in order to quantify the mineralogy of the soil types. Because of the overlapping of the peaks of the different types and processes of absorption of the components and due to the wide range of particle size distribution, etc., the classical method is not sufficient for XRD pattern analysis.

There has been an improvement in the accuracy of the quantitative results for mixtures of minerals by the use of the Rietveld method for XRD quantitative analysis. This method also enables the utilization of XRD to complex systems as soils and makes it a useful tool for mineralogical soil investigations.

The method developed by Hugo Rietveld in 1969 has the capability to refine crystalline structures by means of neutron diffraction data. This method has progressed in a short time and it is used in various fields such as structural analysis (McCuster et al., 1999), crystalline perfection investigations, reticular parameters measurement, phase

transformations (Bish & Howard, 1988; Pascoal et al., 1999; Ortiz et al., 2000), determination of amorphous content (De la Torre et al., 2001) and quantitative analysis of mineral mixtures (Jones & Bish, 1991; Taylor & Rui, 1992; Bish & Post, 1993; Raudsepp et al., 1999).

The Rietveld method is quite suitable for various applications in studies regarding the soil. It has been used in the determination of clays in soils (Weidler et al., 1998; Bravo et al., 2003) and in coal (Chuande & Ward, 2003), of mixtures of minerals (Bonetto et al., 2003), of montmorillonites quantification in soils (Dermatas & Dadachov, 2003) and in the adjustment among experimental and calculated values of densities of surface soil charges (Taubaso et al., 2004).

2.2.3 Swell mechanism in clays

It is essential to understand microscopic mechanisms underlying clay swelling to control the swelling in many engineering applications. Swelling soils, also known as expansive soils, are ones that swell in volume when subjected to moisture. These swelling soils typically contain clay minerals that attract and absorb water. When water is introduced to expansive soils, the water molecules are pulled into gaps between the soil plates. As more water is absorbed, the plates are forced further apart, leading to an increase in soil pressure or an expansion of soil's volume (Handy, 1995). Therefore hydration and double layer repulsion are principal driving forces in swelling. The clay mineral responsible for most expansive damage is smectite, although illite also has some swell potential. The montmorillonite clay minerals, one of the smectite group, are considered as highly expansive. They show the most profound swelling behavior.

The minerals of the smectite group have a prototype structure similar to that of pyrophyllite, consisting of an octahedral sheet sandwiched between two silica sheets. Bonding between successive layers is by van der Waals forces and by cations that are present to balance charge deficiencies in the structure. These bonds are weak and easily separated by cleavage or adsorption of water or other polar liquids.

In the first stage of hydration of the clay particles, water is adsorbed in successive monolayers on the surfaces and pushes the particles or the layers of montmorillonite clay apart. In this stage, surface hydration energy causes swelling (Figure 2.8). Interaction of water with clay surface reduces the chemical potential of water, thereby

generating a gradient in the chemical potential that causes additional water to flow into the system (Mitchell, 1993).

At plate distances beyond $10\text{\AA}$, surface hydration energy is no longer important, and the electrical double layer repulsion becomes the major repulsive force between plates (Van Olphen, 1977). Exchangeable cations that are held to negatively charged clay surface have a higher concentration than in water. Since cations do not have the ability to move, water enters in between the mineral layers to equalize the concentration difference. Consequently, water exerts an osmotic pressure force to open up the mineral layers as shown in Figure 2.9 (Mitchell, 1993).

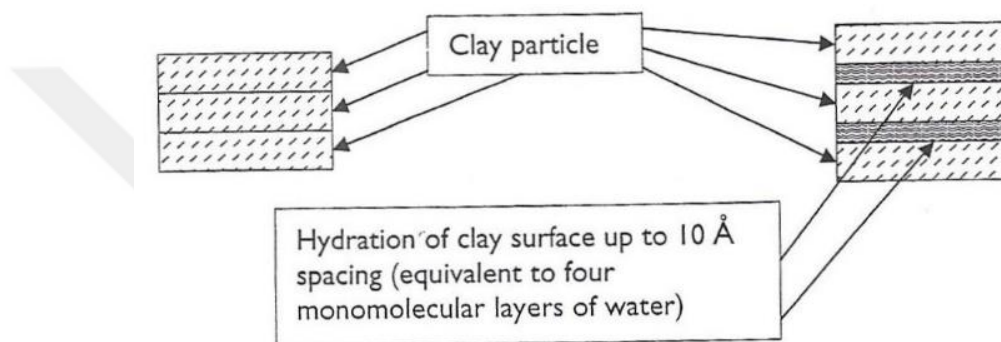


Figure 2.8 : Surface hydration (After van Olphen, 1977).

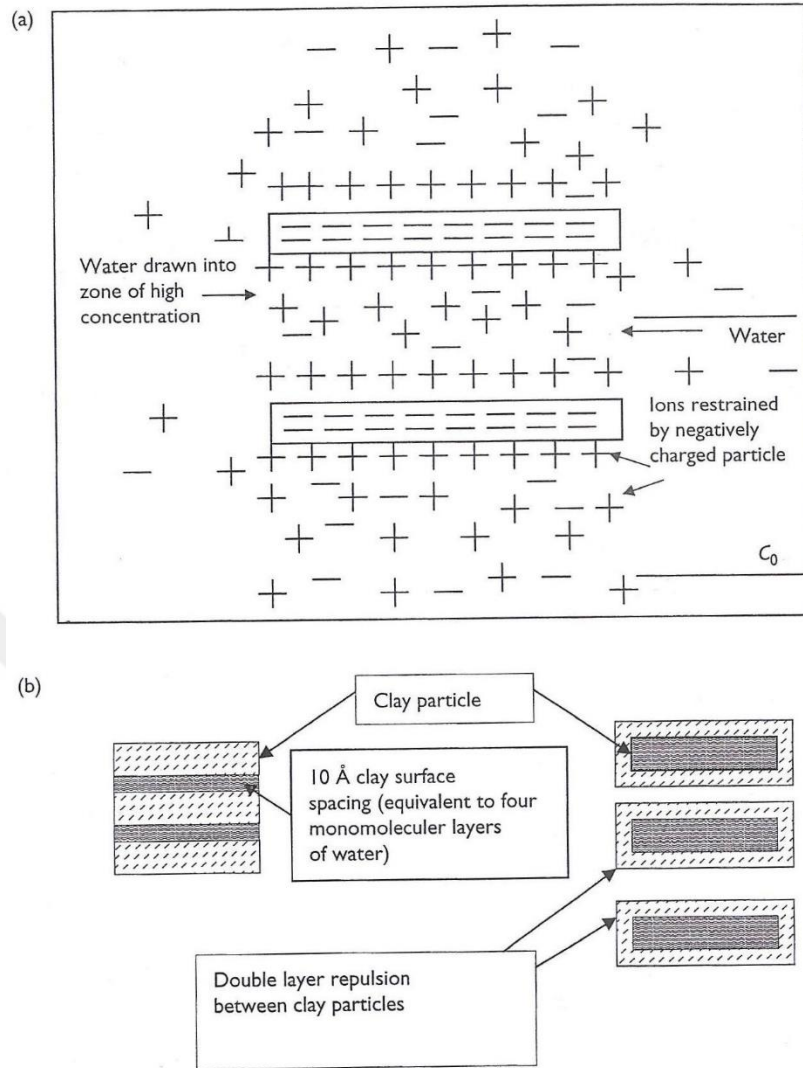


Figure 2.9 : (a) Swelling of soil with entrance of water in between mineral layers, (b) Interparticle swelling (After Mitchell, 1993).

2.2.3.1 Factors affecting swell potential

There are three main categories affecting swell potential of a clayey soil (Nelson and Miller, 1992).

- Soil properties
- Environmental conditions
- Soil's state of stress

In the first category, mineralogical composition of clay (i.e. montmorillonite mineral has the highest swell potential), structure of clay (i.e. flocculated clay particles have a greater tendency to swell compared to clays with dispersed particles), mineralogical composition of pore water (i.e. high concentration of cations as well as increased cation valences reduce the tendency to swell), plasticity (i.e. clays that have high liquid limit generally exhibit a tendency to swell), dry density (i.e. the larger the dry density, the closer the particle spacing, that in turn facilitates repellent forces and hence swell potential) and suction are the factors affecting swell potential.

Within the environmental factors, there are climate (i.e. the problem of expansive clays is very common in arid and semi-arid regions of the world), temperature variations (i.e. an increase in ambient temperature causes soil moisture to dissipate towards cooler areas, mostly under foundations or pavement slabs, causing expansion underneath), initial moisture content and its variations (i.e. natural clayey soils with smaller initial moisture content and with high liquid limit have a potential to swell), location of water table (i.e. superficial and fluctuating water tables promote heave), surface drainage and artificial water sources (i.e. water accumulation from surface drainage or seepage from manmade hydraulic construction contribute to swell), field permeability (i.e. fissures and cracks cause high permeability of the soil, thus leading to higher rates of heave) and vegetation.

In the last category, stress history (i.e. overconsolidation of a clay causes an increase in swell potential compared to the same clay with the same void ratio under normal consolidation conditions), surcharge (i.e. external loading, if large enough, counteracts the repulsive forces and eliminates swell), soil profile (i.e. superficial expansive layers reaching down below the active zone have a greater tendency to swell than the ones that are covered by a nonexpansive layer) and initial in-situ stress state are the important factors affecting swell potential.

Expansive soil problems typically arise as a result of an increase in water content in the upper few meters (i.e. the active zone). The water contents in these upper few meters are influenced by climatic environmental factors. Therefore, the depth of the active zone can vary with time. Since the deep-seated heave cases are quite rare, knowing the properties of the soils in the shallow active zone is very important to take remedial measures against heaving (Nelson and Miller, 1992).

2.2.4 Classification of expansive soils

Expansive soils cause damages to structures, particularly light buildings and pavements, quite often worldwide in every year. It was reported for instance that the annual losses from expansive soils are more than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973, FEMA, 1982). It is essential therefore that a proper and sound evaluation of expansive soil classification should be incorporated in the analysis and design stage of an engineering structure.

The tests for grain size distribution, Atterberg Limits (liquid limit, plastic limit, shrinkage limit) and the clay content are the most widely used in practice for classifying expansive soils. Mineralogical identification procedures such as X-Ray Diffraction or Differential Thermal Analysis may also be used.

Oedometer tests can be utilized as a direct measuring method to determine the swell percentage and swell pressure values. There are also a number of indirect methods to determine the swelling potential of expansive soils. These indirect classification methods generally offer a qualitative rating in which expansive soils are termed as “non-critical” to “highly critical” with respect to their expected expansion. However, any two soils with similar qualitative rating may show very different swell characteristics (Seed et al., 1962). A satisfactory classification method therefore should include the soil properties (i.e. index and/or mineralogical properties) along with the actual stress data and the qualitative rating.

The simplest way to determine the potential for expansion was suggested by Peck et al. (1974). Plasticity index (PI) is the only indicative feature to have an idea about expansion potential in their suggestion (Table 2.1).

Table 2.1 : The relationship between plasticity index (PI) and expansion potential (Peck et al., 1974).

Plasticity Index (PI, %)	Expansion Potential
0-15	Low
0-35	Medium
20-55	High
>35	Very High

However, Zapata et al. (2006) showed that the expansion potential for remoulded expansive soils correlates poorly to the plasticity index alone. They concluded that the correlation is significantly improved by correlating expansion potential with the product of plasticity index and percent passing the No. 200 (75 μ m) sieve.

Another way including the particle size into consideration was suggested by Skempton (1953). He combined the Atterberg limits and clay content (less than 2 μ m size) into a parameter called activity, A_c , which is equal to plasticity index divided by % by weight finer than 2 μ m. Skempton suggested three classes of clays according to activity (Table 2.2).

Table 2.2 : The relationship between activity (A_c) and expansion classification (Skempton, 1953).

Activity value (A_c)	Degree of activity
$A_c < 0.75$	Inactive
$0.75 < A_c < 1.25$	Normal
$A_c > 1.25$	Active

Active clays provide the most potential for expansion in the above table.

The parameter expansion index (EI) can also be used for expansion potential determination. The expansion index test was developed in southern California in the late 1960s. ASTM International has published a standard method of test for the EI test (ASTM D4829). The test is basically a consolidation-swell oedometer test which consists of compacting a soil at a degree of saturation of $50\% \pm 2\%$ under standard conditions. A vertical stress of 7 kPa is applied to the sample, and the sample is inundated with distilled water. The expansion index is calculated by,

$$EI = [(final\ thickness - initial\ thickness) / initial\ thickness] * 1000$$

The expansion potential of the soil is classified according to the expansion index, as shown in Table 2.3 (Nelson et al., 2015).

Table 2.3 : Expansion potential based on expansion index (EI).

Expansion Index (EI)	Expansion Potential
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
> 130	Very high

Yao et al. (2004) proposed the Standard Absorbition Moisture Content (SAMC) method for identification of expansive soils. This method was recommended in China's Specifications for Design of Highway Subgrades (CMC, 2004). The test consists of placing an undisturbed soil sample on a porous plate within a constant humidity container over a saturated solution of sodium bromide. The weight of the soil sample at equilibrium is measured, after which it is oven-dried. The SAMC is calculated as:

$$SAMC (\%) = (W_e - W_s) / W_s$$

where: W_e =weight of sample at equilibrium (77 °F and 60% relative humidity) and W_s =weight of oven-dry sample.

CMC (2004) Specification presents a method for classifying expansive soils based on the SAMC, plasticity index, and free-swell values (Table 2.4).

Table 2.4 : Classification standard for expansive soils (CMC, 2004).

SAMC (%)	Plasticity index (%)	Free-Swell value (%)	Swell potential class
<2.5	<15	<40	Nonexpansive
2.5-4.8	15-28	40-60	Low
4.8-6.8	28-40	60-90	Medium
>6.8	>40	>90	High

The classification system developed by U.S. Army Waterways Experiment Station (Snethen et al., 1977) is one most widely used in the United States. It has also been summarized by O'Neill and Poormoayed (1980), (Table 2.5).

Table 2.5 : Expansive soil classification system** (Snethen et al., 1977).

Liquid limit (LL)	Plasticity index (PI)	Potential swell (%)	Potential swell classification
<50	<25	<0.5	Low
50-60	25-35	0.5-1.5	Marginal
>60	>35	>1.5	High

Note: Potential swell=vertical swell under a pressure equal to overburden pressure.

** Compiled from O'Neill and Poormoayed (1980).

The other classification systems contained in the literature are given in Das (1999). They are given in Figure 2.10 below.

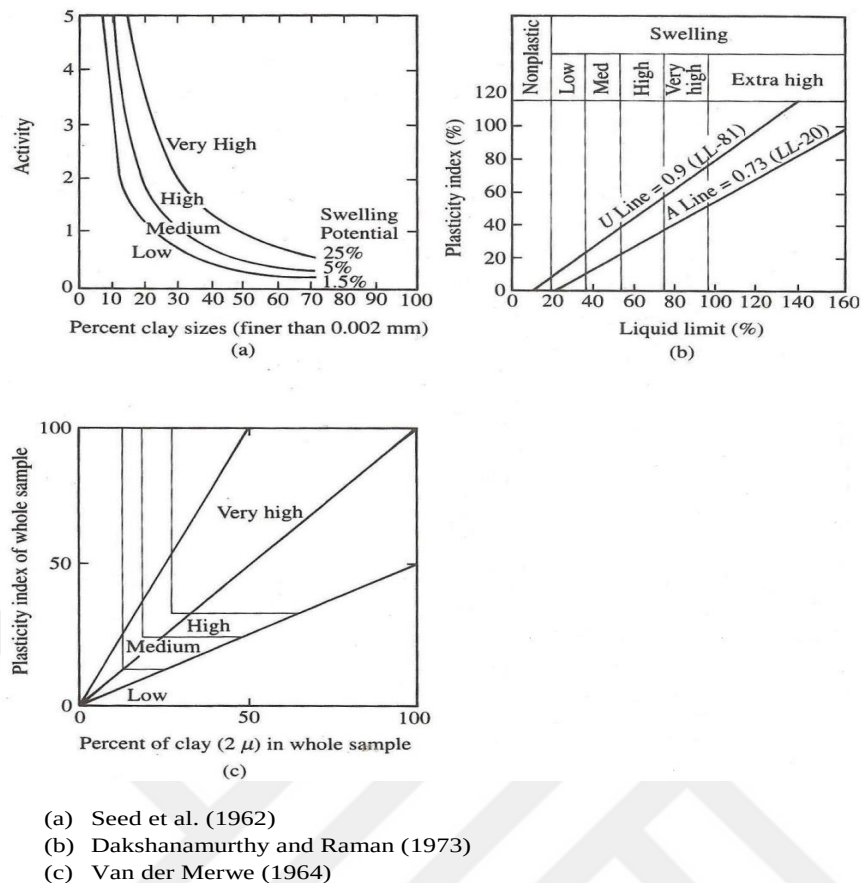


Figure 2.10 : Criteria for determining swell potential (Das, 1999).

2.3 Chemical Stabilization

Chemical stabilization involves major applications of chemicals to improve the behavior of soils, but it generally does not cause the soil particles to be bound together. Although commonly called chemical “stabilization”, the more correct term is chemical modification. Chemical stabilization using various additives has been one of the popular techniques for stabilization of expansive soils (Sharma et al., 2008). Chemical agents can be added by spraying on the surface of the layer to be treated, by intimate mixing of the agent with the soil layer to be treated, or by injection of chemical slurry into the soil subgrade (Petry, 2010).

2.3.1 Lime stabilization

Lime is a source of calcium ions, which happen to easily exchange into a clay for cations that make the clay more active. There are basically two types of lime used for most applications. When lime is produced by driving away carbon dioxide from limestone-

type rock, the result is quicklime or CaO . Although usable in dry form, quicklime has to be hydrated either during the mixing process or as it is made into hydrated lime or CaOH_2 prior to mixing. The most useful form of lime now being added to clay soils is a slurry suspension of quicklime, often hydrated in the field to save the cost of transporting the heavier hydrate to the field. Lime tends to be a preferred choice, since it is relatively inexpensive compared to the results achieved. Modifying a clay soil normally takes less than 6% lime by dry weight of soil (Petry, 2010). Lime additive especially after 1990s has been used in many projects to minimize swelling (Snedker, 1996, Athanasopoulou and Kollaros, 2011, Mellas et al., 2012).

Practically, the most commonly used method of lime application is to mix it mechanically. With this method, however, it is difficult to mix deeper than about 30 cm. Therefore, it is frequently used for treatment of subgrades below pavements or slabs. For deeper depths of treatment the soil needs to be treated in several lifts. Lime can be applied dry, which is fastest, or in a lime slurry (Nelson et al., 2015).

The laying of lime or fly ash agent on subgrade soil and mixing machine is shown in Figure 2.11. Roller compaction process is shown in Figure 2.12 (M.A. Patel and S.A. Patel, 2012).



Figure 2.11 : Laying of lime/fly ash and onsite mixing machine.



Figure 2.12 : Roller compaction process.

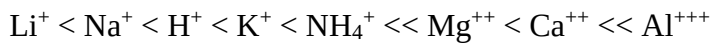
2.3.1.1 Lime-soil reactions

The combination of lime with water and the soluble silica and alumina that are present in clay lead to a process of chemical reaction which eventually results in the formation of new compounds. The fundamental function of this process is the change in the structure of particles as well as the increased shrink-swell resistance and decrease susceptibility to moisture. However, another result is the binding of particles at the time they are combined with clay, leading to a gain in strength. Hence, short-term benefits can be regarded as the cation exchange and flocculation-agglomeration phenomena, whereas pozzolanic reactions are long-term stabilization.

The initial phase of the chemical reaction is the abrupt alterations in the soil texture and soil properties that are caused by the effect of cation exchange. The exchange of free calcium of the lime with the adsorbed cations of the clay mineral results in the reduction of the size of the diffused water layer that covers the clay particles. In cation exchange, bivalent calcium ions (Ca^{+2}) are replaced by monovalent cations. The Ca^{+2} ions link the soil minerals (having negative charge) together, thereby reducing the repulsion forces and the thickness of the diffused water layer. This layer encapsulates the soil particles, strengthening the bond between the soil particles. The remaining anions (OH^-) in the solution are responsible for the increased alkalinity (George et al., 1992, Mallela et al., 2005, Geiman, 2005). This reduction in the diffused water layer makes the clay particles come together with more closer contact which causes the flocculation-agglomeration process of the clay particles leading the clay to transform into a more silt-like and sand-

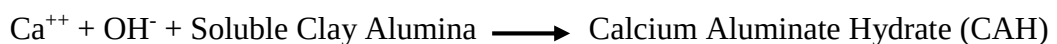
like material. This process causes a structural deformation as illustrated in Figure 2.13. In general, the flocculation-agglomeration phase of lime stabilization results in a soil which is more readily mixable, workable and compactable (Eades and Grim, 1960, TRB Report, 1987).

The fact that calcium will replace most cations available in the water system is documented by the Lyotropic series which generally states that higher valence cations replace those of a lower valence, and larger cations replace smaller cations of the same valence. The Lyotropic series is written as:



where the cation to the right replaces the one to the left. Thus in equal concentrations Ca^{++} can easily replace the cations commonly present in most clays (Little, 1995).

The second phase of the chemical reactions are related to pozzolanic reactions. They occur within the lime-soil mixture and lead to strength gain in the long term. Due to the addition of lime to the soil, the pore water becomes more alkaline and pH value reaches up to 12.4. So, the alumina and the silica of the clay becomes soluble and are released from the clay mineral. Thus, the released silica and alumina react with the calcium from lime in order to form the cementation (i.e. CSH, CAH) and gradually it gains strength over several years (Eades and Grim, 1960, TRB Report, 1987). The reactions of the long term products, CSH and CAH, are illustrated by the following equations:



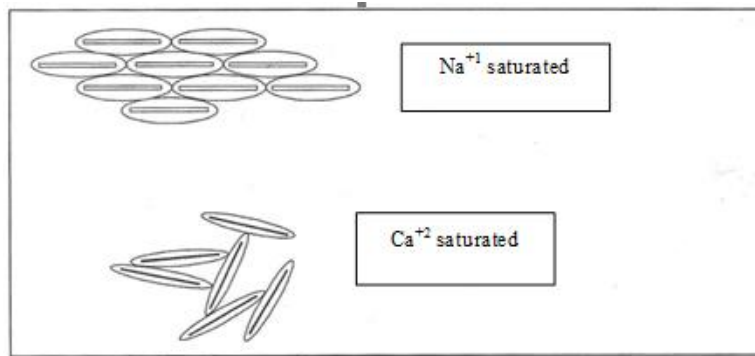


Figure 2.13 : Structural transformation as a result of cation exchange and flocculation-agglomeration.

Lime-treatment process before and after lime addition can be clearly observed in Figure 2.14 (URL-1). The plastic nature of the untreated soil is obvious on the left part of this figure. After mixing lime along with the effect of water, the soil is now sand or silt-like material as shown on the right part of the figure.



Figure 2.14 : Treatment process before and after lime stabilization.

2.3.2. Fly ash stabilization

Since fly ash is a waste material, beneficial use of it decreases the need for large disposal areas and provides a cheap agent that can be used in construction activities.

There are two different fly ash classes, namely class F and class C. Class F fly ash is a non-self-cementing type that is produced from burning of anthracite and bituminous coals. It contains a small amount of lime (CaO) which makes cementation more difficult; thus utilizing this kind of product itself might not be sufficient without any activator such as Portland cement or lime. Class F fly ash contains siliceous and aluminous material which has little or no cementitious value. However, in the presence of moisture, it reacts with lime to form cementitious compounds at ordinary temperatures (Chu and Kao, 1993).

Class C fly ash is a self-cementing fly ash which is obtained from lignite and sub-bituminous coals, which are low-sulfur sub-bituminous coals. They generally involves a considerable amount of lime (Cockrell and Leonard, 1970). Since class C fly ash contains a high percentage of calcium oxide (CaO), ranging from 20-30 %, this type of fly ash is itself sufficient to be a self-cementing one.

2.3.2.1 Fly ash-soil reactions

Many clay soils (expansive soils) undergo extensive volumetric changes when subjected to fluctuating moisture contents. These volumetric changes if not controlled can lead to movements in structures and impose loads which can cause premature failure.

Fly ash reduces the potential of a plastic soil to undergo volumetric expansion by a physical cementing mechanism, which cannot be evaluated by the plasticity index. Fly ash controls shrink-swell by cementing the soil grains together much like a Portland cement bonds aggregates together to make concrete. By bonding the soil grains together, soil particle movements are restricted. Typical addition rates based on dry weight of soil are 12 to 15 percent (FHWA).

Class C and Class F fly ashes can be used as an agent in many geotechnical applications especially in highway constructions.

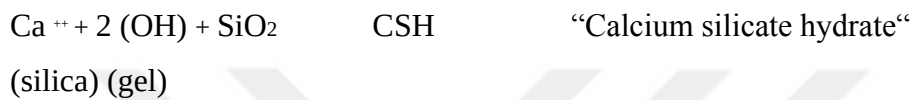
It should be noted that a Class C fly ash may contain 15% or more calcium expressed as calcium oxide, but very little of this calcium is in the form of free calcium oxide (CaO). Therefore, the determination of the plasticity limits is inappropriate when evaluating the effect of fly ash on the shrink-swell characteristics of a soil.

The fly ash used in this study is a Class C material with moderate calcium oxide – CaO (12%).

Fly ash has also been used successfully in many projects to improve the strength characteristics of soils. Fly ash can be used to stabilize bases or subgrades, to stabilize backfill to reduce lateral earth pressures and to stabilize embankments to improve slope stability (FHWA).

Research show that even little amount of lime that is present in the fly ash might cause the pozzolanic reactions to increase which results in the stiffening of the mixture.

The reaction of lime with pozzolans (Al_2O_3 , SiO_2 , and Fe_2O_3) resulting in the formation of cementitious material in the presence of water is known as hydration of fly ash. Cementitious materials such as hydrated calcium silicate or calcium aluminate can react with inert materials. The pozzolanic reactions for soil stabilization are as follows (TRB, 1987):



The lime present in the fly ash reacts with the siliceous and aluminous materials called “pozzolans” in the case of the class C fly ash. Since the fly ash class F has low lime content, lime must be added for a similar reaction to be originated (Şenol et al. 2002).

2.4 Cyclic Properties of Treated Soils

2.4.1 Resilient modulus test

The Resilient Modulus (M_r) is a measure of subgrade material stiffness. A material's resilient modulus is actually an estimate of its modulus of elasticity. While the modulus of elasticity is stress divided by strain for a slowly applied load, resilient modulus is stress divided by strain for rapidly applied loads – like those experienced by pavements. Resilient modulus test is essentially a cyclic version of a triaxial compression test; the cyclic load application is thought to simulate the actual traffic loading more accurately. Thus, the resilient modulus fundamentally, represents the unloading modulus after several repeated cycles of loading allowing to simulate road traffic (Figure 2.15).

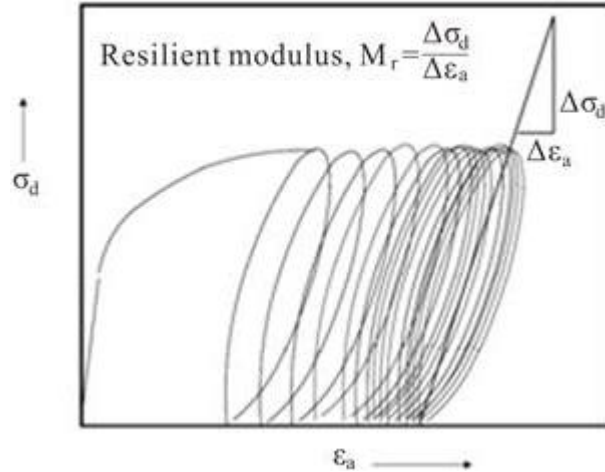


Figure 2.15 : Definition of resilient modulus.

In Figure 2.15, $\Delta\sigma_d = \sigma_1 - \sigma_3$ is the deviatoric stress where σ_1 and σ_3 are the major and minor principal stresses respectively. $\Delta\epsilon_a$ is the resilient axial strain.

Resilient modulus is determined using a special triaxial test. The test applies a repeated axial cyclic stress of fixed magnitude, load duration and cycle duration to a cylindrical test specimen. While the specimen is subjected to this dynamic cyclic stress, it is also subjected to a varying static confining stress provided by a triaxial pressure cell, ranging from (14 kPa, 28 kPa and 42 kPa). These stress levels are in accordance with those encountered at the subgrade design in KGM standards (KGM, 2008). The minimum number of cycles at the pre-conditioning stage of the tests is 500. If the length reduction of the sample continues at the end of 500 cycles, the number of cycles can be extended up to 1000 at this stage. The number of the cycles in all other stages is 100 (AASHTO T307-99, 2007, KGM, 2008).

The resilient modulus value provides a basic relation between the stress and strain of the material in order to perform the structural analysis of the layered pavement systems. This value also enables to define highway materials under various conditions such as water content, density etc. and under different stress conditions that exemplify live wheel loads.

A typical pavement design with the definitions of different layers is given in Figure 2.16 (Christopher et al., 2006).

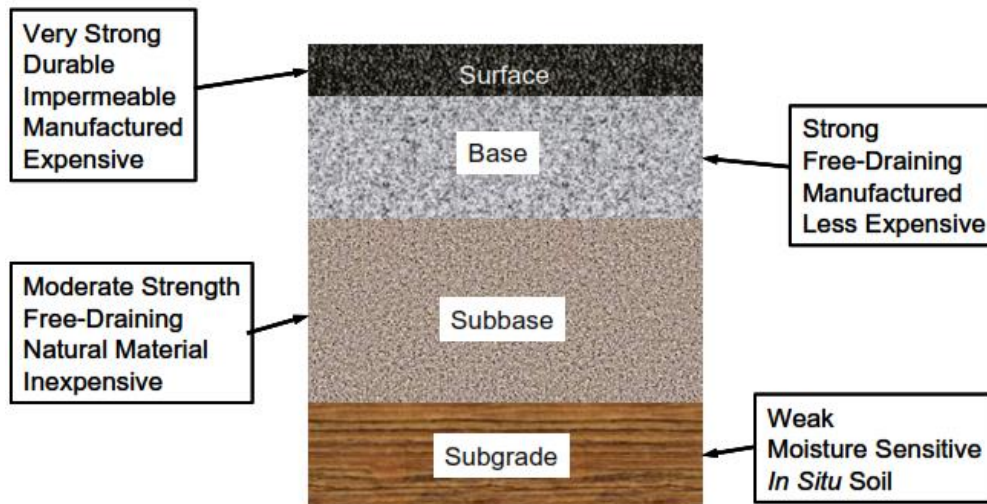


Figure 2.16 : A typical pavement design.

The minimum CBR value for subgrade material is given as 8% in Turkish Highway Standards. For $CBR < 8\%$, soil improvement should be carried out. There is a correlation between CBR and M_r value in Christopher et al. (2006):

$$M_r \text{ (MPa)} = 17.6 * (CBR)^{0.64}$$

Then, the lower limit for M_r value is about 66.000 kPa (9500 psi).

2.4.1.1 Resilient modulus of lime-treated soils

The lack of research regarding the elastic properties of stabilized soils has led to the utilization of empirical or semi-empirical tests such as California Bearing Ratio (CBR) and unconfined compression tests. In spite of the fact that static tests are relatively easier to conduct, the correlation between these tests and field measurements is unsatisfactory. Robnett and Thompson (1976) suggested a good correlation between the field performance and the elastic deflection of flexible pavements. After examining the resilient behavior of various untreated and lime-treated soil samples, they came up with a conclusion that lime treatment can obviously be used for improving the resilient behaviors of soils (Achampong, F., 1996). Sauer and Weimer (1978) also reported that the resilient modulus increases considerably when the lime content increases. Furthermore, soil-lime mixture has basically a linear elastic behaviour and is independent of confining pressure, deviator stress, soil suction and water content.

Resilient modulus of lime treated soils is influenced by various parameters such as the intensity of the stress, number of cycle of loading, type of the soil, lime content, curing time, degree of saturation, density, unit weight and time period between mixing and compaction. The interactions among these parameters can be regarded as the major influencing factor on resilient modulus.

The resilient modulus of lime-treated clayey soils is inversely proportional with stress intensity. However, the percentage decrease in resilient modulus value is smaller when compared to the untreated soil. Sauer and Weimer (1978) concluded that the effect of confining pressure on the lime treated clay is very little because of the fact that lime-clay mixture becomes more rigid.

Singh and Akoto (1981) found that resilient modulus decreases while increasing the load until a certain minimum value and then continues to increase with the increase of the intensity of stress. Thompson (1975) suggested that for many soil-lime mixtures, progressive strength development with increased curing lead to reverse the influence of repeated loading. The resilient modulus of lime-treated soils display both cohesive and frictional characteristics and are highly affected by deviator stress and confining pressure. The value of resilient modulus increases with an increase in confining pressure whereas with a decrease in deviator stress and for a lime-clay mixture, the increase in curing time increases the resilient modulus (Fossberg, 1969).

2.5 Past Studies on Expansive Soils

In one of the early studies in this subject, Baykal et al. (1989) studied the accelerated curing of fly ash-lime-soil mixtures. The morphological, mineralogical and stress-strain properties of compacted bentonite-fly ash-lime, bentonite-fly ash, and bentonite-lime cured at 23°C and 50°C were compared. The elastic moduli and compressive strengths were obtained from unconsolidated, undrained triaxial and unconfined compression tests. Compressive strengths of the mixtures were comparable for one-day 50°C and 28-day 23°C cured mixtures (i.e. the variation is within $\pm 15\%$). In general, the calculated elastic moduli were lower after short-term high-temperature curing. Swelling properties of the mixtures was out of the scope of this study.

Kavak (1996) studied the behavior of lime-stabilized clays under cyclic loading. Kaolinite and bentonite clays, which are commercially available, were used in this

Ph. D. research. Different lime contents by weight (i.e. 2%, 4%, 6%, 8%, 10%, and 12%) were used for the stabilization of clays. A stress-controlled pneumatic cyclic loading device was developed in this study for the application of the cyclic loading to the specimen at a determined frequency. Stress ratio, which is defined as the ratio of the maximum cyclic stress during loading to the unconfined compressive strength of the soil, was another variable in this study. It was found that a decrease in unconfined compressive strength of the 28-day cured stabilized samples is observed after 1000 and 10.000 cycles of load applications depending on the stress ratio. The probable reason of these strength losses as much as 30% around 1000 and 10.000 loading cycles was explained by the breaking of the cement minerals of the lime stabilized soils. It was found in this study also that the stress ratio and the critical number of cycles are inversely proportional for lime-stabilized kaolinite. When the stress ratio of kaolinite is 0.4, the critical number of cycle is 10.000 cycles. The critical number of cycles drops to 1000 for a stress ratio of 0.5.

Baykal and Döven (2000) and Baykal et al. (2006) made research on fly ash as construction material in highway applications. In the first research, utilization of fly ash by pelletization process was studied. They used lime and cement also to enhance the process. Three aggregate groups (i.e. fly ash only, with lime and with cement) can be used as base and subbase material according to their research. However, it is emphasized that construction on soils with swell potential must be paid extreme care due to the lower unit weight of materials. Baykal et al. (2006) studied the use of fly ash in highway embankment constructions in cold regions. It was found that the addition of snow is quite useful to introduce extra water during the compaction of fly ash without causing any workability problem.

Çokça (2001) studied the effects of lime, cement and class C fly ash stabilization on an expansive soil. Artificial soil samples which are made by combining kaolinite and bentonite in the laboratory were tested. Plasticity index values were generally between 12% and 35%. The activity values of the samples were generally smaller than 1.00. It is found in this research that the plasticity index, activity and swelling potential of the samples decreased with increasing percent stabilizer and curing time. Optimum fly ash content was found to be about 20%. Çokça (2001) states on the basis of this research that class C fly ashes can be recommended as effective stabilizing agent for improvemet

of expansive soils. It should be noticed here that the the activity of the samples were smaller than 1.00 and the samples were prepared artificially in the laboratory.

Türker and Çokça (2006) summarized the results of another experimental study they carried out. F and C type fly ash were used on an expansive soil for stabilization. Artificial soil samples with kaolinite, bentonite, sand and fly ash were prepared. The plasticity index values were generally between 46.3% and 53.8%. The activities of the samples were between 1.13 and 1.29. It was concluded in this study that both class C and class F fly ash with sand can be recommended as effective stabilizing agents for improvement of expansive soils.

A.H. Lav and M.A. Lav (2000) studied the effects of cement and lime stabilization on fly ash as pavement base material in terms of chemical composition, crystalline structures, and hydration products. Unconfined compressive strength of samples was also measured to observe the effect of stabilization over time. The fly ash used in this study was low-calcium fly ash (Class F). Other significant elements present in the fly ash were found to be silicon and aluminum, which are present as quartz and mullite. They found that cement- and lime-stabilized fly ash has produced similar hydration products. Cement-stabilized samples gained higher strength than lime-stabilized samples in all curing times (i.e. 7, 14, 28, 90, 180 and 360 days).

The effect of different contents of cement (i.e. 2%, 4%, 8%, and 10% of cement by weight) in fly ash-treated samples was studied by Lav et al. (2006). The purpose of this research was to investigate the stabilization of fly ash without aggregate and to see the suitability of the stabilized material to be used as pavement base material. Texas triaxial test, which is an undrained test measuring the deformation resistance of pavement layers. In the design charts they present, cement content is the most important parameter controlling the design life (fatigue performance) of stabilized layers. Since the strength of aggregate free stabilized fly ash is lower than the ones mixed with aggregate, cement content and layer thickness are recommended to be not less than 8% and 300mm, respectively.

M.A. Lav and A.H. Lav (2014) used the cement and lime agents separately to stabilize the class F fly ash material with low calcium content (i.e. 1.61%). The effects of stabilization on resilient characteristics of fly ash as pavement material were investigated in this study. Repeated load indirect tensile and cyclic triaxial tests were

carried out. They found in conclusion that stabilized fly ash is a stress dependent material and cyclic triaxial results showed that an apparent relationship between resilient modulus and the first stress invariant (the sum of the three principal stresses). Therefore, the resilient modulus of the material increases with the increasing magnitude of the first stress invariant. It was proved in this research that the samples were not subjected to significant permanent strains after removing the confining stresses. They recommended as a result that the material used in this study should be designed in thicker layers than conventional pavement layers to keep strains lower in practical applications.

Şenol and Edil (2004) used two types of soil samples which were taken at about 0.6 to 1m depths in their research. Type-A soil was low plasticity clay (CL) with a plasticity index of 20% and Type-B soil was low plasticity silt (ML) with a plasticity index of 15%. Class C type of fly ash was used as an stabilizing agent. The improvement process was studied in terms of CBR values after 7 days of curing. Fly ash contents by weight were between 10% and 20%. It was found that CBR values of stabilized soil samples were at least 10 times larger than the ones of untreated soil samples. It has been concluded that the improvement level is always larger at ideal laboratory conditions than the level that can be attained at chaotic field conditions. Thus, it is recommended to use a safety factor when a transition from laboratory to field is required.

Swelling behaviour of expansive soil treated with fly ash and ground granulated blast furnace slag (GGBS) was studied by Sharma and Sivapullaiah (2017). The investigation was conducted on an artificially mixed soil containing 80% Indian expansive soil (popularly known as Black cotton soils) and 20% commercial bentonite on dry weight basis. The plasticity index and the shrinkage limit of the expansive soil obtained were 33% and 15% respectively. The binder to stabilize the expansive soil was obtained by mixing fly ash and GGBS. The treatment of soil with binder resulted in considerable reduction in the free swell index, swell potential and swelling pressure with the increase in binder content. Further, addition of 1% of lime had significant effect on the swelling characteristics. The test results tend to promote the use of fly ash admixed with GGBS as potential binder for expansive soils. It is to be noted that the plasticity index of the artificial expansive soil in this study was only 33%.

Kumar and Sharma (2004) investigated the effect of fly ash on engineering properties of a natural expansive soil. The plasticity index and the clay fraction of the soil samples

were 52% and 69% respectively. It was found by this study that the free swell index of expansive soils can be effectively reduced by addition of fly ash. For the expansive soil used, the free swell index was reduced by about 50% by adding 20% fly ash. The addition of fly ash to expansive soil also reduced the swell potential and swelling pressure. Both the swell potential and swelling pressure were reduced by 50% at 20% fly ash. The undrained cohesion (c_u) of the expansive soil blended with fly ash increased with the fly ash content.

Volume change behavior of fly-ash stabilized clays was studied by Phanikumar and Sharma (2007). Three types of clays were used in this study. Two types of expansive clays and one type of nonexpansive clay were mixed with different percentages of fly ash. For the type of fly ash and expansive clays used, 20% fly ash content reduced free swell index, swell potential, and swelling pressure (as determined by the free swell method) by about 50%. It was found that the maximum dry unit weight increased and optimum moisture content decreased with increasing fly ash content. At 20% fly ash content, maximum dry unit weight of nonexpansive clay increased by about 7%, and optimum moisture content decreased by about 15%. The compression index of both the expansive and nonexpansive clays decreased by about 50% at 20% fly ash content indicating that addition of fly ash reduced compressibility characteristics of both expansive and nonexpansive clays. Mineralogical properties of the clays used were not given in this study. Therefore, a complete evaluation is not possible by the data given in this paper.

Bozbey and Garaisayev (2010) used a local expansive clay from İstanbul in their research. They concentrated on the effects of soil pulverization quality on lime stabilization of the expansive clay. The effect of mellowing the soil-lime mixtures for 24 hours was also studied. It was found that the improvements in unconfined compressive strength and secant modulus of elasticity values depended strongly on soil pulverization quality. In this research, the effects of mellowing for 24 hours did not have a positive effect on any measured or calculated geotechnical property.

The effects of lime on resilient modulus properties of a clayey soil were investigated by Bozbey et al. (2015). A highly plastic clay (CH) with a plasticity index of 41 was used in this study. 4% lime content by weight was utilized for stabilization. The level of improvement in resilient modulus properties was found to be depended on soil pulverization level significantly. The results showed for instance that, in case soil

pulverization was not fine (i.e. not passing No. 4 sieve by 100%), 4% lime stabilization could not be beneficial. Stress state (confining and deviatoric stresses) and curing time (7 and 28 days) were also very important parameters according to the findings in this research.

A reappraisal of the lime stabilization of soils was carried out in the study of Dash and Hussain (2012). Six different soil samples were constituted using two extreme types of soils. The expansive soil is a commercially available bentonite. The nonexpansive soil used is a residual soil classified as clay with low plasticity. Silica content of the residual soil is found to be quite high. Quick lime (CaO) was used as the stabilizing agent. It was found that the liquid limit of soils initially decreases with an increase in lime content. However, beyond 5% of lime content, the consistency limits no longer change. For silica-rich soils at relatively higher content of lime and prolonged curing period, on the other hand, both the liquid limit and plastic limit of the soils increase and the soils turned to be increasingly plastic. Correspondingly, the swell potential of soils initially decreases with increased percentage of lime to a practically negligible value. However, it again increases as lime content is increased further. These lime contents were found to be 5% and 9% for fine-grained and coarse-grained soils respectively. Similar results were obtained in strength tests. It was found specifically in this study that excessive lime treatment is not effective in case of silica-rich soils.

Phanikumar (2009) made a comparative study in which the effect of lime and fly ash on swell, consolidation and shear strength characteristics of expansive clays were investigated. The expansive soil used in the experimental study was collected in the State of Andhra Pradesh, India. The plasticity index (PI) and the clay fraction of the expansive soil were 52% and 69% respectively. The plasticity index decreased with increasing additive content for both lime and fly ash. Free swell index also decreased when lime content and fly ash content were increased. Swell potential and swelling pressure decreased significantly with increasing lime and fly ash contents. The compression index increased initially up to a certain additive content and thereafter decreased for both additives. The activity of the clay was less than 1.00 and the mineralogical properties of the clay were not stated in this study.

Another comparative study was conducted by Phanikumar et al. (2015). Swell and compressibility characteristics of lime-blended and cement-blended expansive clay were investigated. The plasticity index (PI) of the natural clay used was 74%. The clay

fraction was 69%. It was found that lime reduced swelling more than did cement. Coefficient of compressibility, coefficient of volume compressibility and compression index decreased significantly with increasing lime and cement contents. Reduction in compressibility was more pronounced in cement-blended specimens than in lime-blended specimens. Linear shrinkage and degree of saturation decreased significantly with lime and cement contents. The mineralogical properties of the clay were not investigated in this study.

Ismaiel (2006) studied three types of fine grained soils in a Ph.D. thesis research. The plasticity index values of the tertiary clay (CH), weathered soil (MH) and organic silt (OH) are 32.9%, 31.9% and 20.8% respectively. It was found that the addition of fly ash and lime resulted in a reduction of the plasticity index and the swell potential especially in tertiary clay. In case of lime stabilization, the optimum lime content was found between 3% and 5% when the strength properties are concerned. Whereas, in the case of fly ash stabilization the optimum fly ash content was between 16% and 35%. It was concluded that both lime and fly ash react strongly with the tertiary clay which contains a high amount of clay particles (i.e. clay content is 47%) including kaolinite, montmorillonite and halloysite minerals.

Nalbantoğlu (2006) studied the lime stabilization effect on an expansive clay which was collected from a depth of 1.5 below the ground surface at a specified site. The plasticity index and the clay fraction of the soil are 45.6% and 33% respectively in this study. According to mineralogical analysis results, it was understood that smectite, illite and kaolinite amounts in the clay were 17%, 3% and 21% respectively. Soil-hydrated lime mixtures were prepared with lime of 3%, 5%, and 7% by dry weight of soil. All specimens were compacted before testing by using a standard Proctor compaction effort at optimum water content. It was found that lime was very effective in increasing the unconfined compressive strength of treated soils. Curing time was investigated up to 30 days; it was found that curing time was also important to record an increase in unconfined compressive strength.

Edil et al. (2006) studied the stabilization effect of fly ash on soft fine-grained soils. California bearing ratio (CBR) and resilient modulus (M_r) tests were conducted on mixtures prepared with seven soft fine-grained soils. The plasticity indices of the soils were between 15 and 38%. The mixtures were prepared at different fly ash contents (i.e. between 10% and 30%) in this laboratory study. It was found that adding 10 and 18%

fly ash to fine-grained soils compacted 7% wet of optimum (i.e. typical in situ condition) resulted in increase in CBR by a factor 4 and 8 respectively. With 10% fly ash and fine-grained soil compacted 7% wet of optimum typically will have lower resilient modulus than soil alone compacted at optimum water content. However, when fly ash content is on the order of 18%, the resilient modulus typically will be higher. It is 30% higher than the resilient modulus of the soil alone compacted at optimum water content in this study. The effect of curing time on resilient modulus was evaluated using one soil and two fly ashes. Between 7 and 14 days, the resilient modulus increased modestly. However, between 14 and 56 days, the resilient modulus increased by 20-50%. Thus, fly ash stabilized subgrades should stiffen over time, resulting in increased pavement support.

Lautrin (1987) states that the methylene blue test is rapid method for the identification of clays. Behaviour of clayey soils depends on many factors, among these mineralogy and cation exchange capacity are important. However, mineralogical identification by using classical methods is a difficult task. Hazard coefficient (N) by using methylene blue value has been evaluated by Lautrin (1987).

$N = \text{Hazard coefficient} = \text{methylene blue value (MBV) for clay fraction.}$

Lautrin (1987) compares the MBV with the mineralogical composition of the soil determined by X-ray method by using around 100 samples. After this study, the variation of the Hazard coefficient as a function of the mineralogical concentration of clays can be summarized as below:

For $N \leq 1 \longrightarrow$ Silts, sands, gravels (there is no clay in the soil)

For $1 < N \leq 3 \longrightarrow$ Inactive soils, clay fraction contains pure kaolinite or kaolinite and illite, there is no montmorillonite.

For $3 < N \leq 5 \longrightarrow$ Little active soils, clay fraction contains montmorillonite but it is less than 10%.

For $5 < N \leq 8 \longrightarrow$ Normal soils, clay fraction contains montmorillonite up to 50%.

For $8 < N \leq 12 \longrightarrow$ Active soils, montmorillonite % in the clay fraction is between 50% and 75%.

For $12 < N \leq 18 \longrightarrow$ Harmful soils, montmorillonite % in the clay fraction is between 75% and 90%.

For $N > 18 \longrightarrow$ Very harmful soils, montmorillonite % in the clay fraction is higher than 90%.

Yükselen and Kaya (2006) recommended a rapid way of estimating cation exchange capacity (CEC) which is an important property of a clayey type of soil especially if the swelling behavior is concerned. Simple correlations were investigated between CEC and specific surface area, clay fraction, activity, and Atterberg limits by using 16 soil samples from different parts of Turkey. Simple linear correlations between CEC and Atterberg limits show that there is a relationship between CEC and liquid limit (LL). However, no significant relationship was found between CEC and plastic limit (PL), plasticity index (PI) or shrinkage limit (SL). The results of this study indicate that specific surface area and liquid limit are the most important factors that affect the CEC of the soils.



3. EXPERIMENTAL PROGRAM

In this chapter, the complete experimental program that has been executed within the scope of this thesis is explained in a detailed manner. To do so first, two different sites are introduced and corresponding material properties of clays in these sites are presented. Then, certain properties of the chemical agents that are utilized during the experiments are studied. After mentioning about the preparation of the samples, corresponding index, strength, oedometer, resilient modulus, methylene blue and Scanning Electrone Microscope (SEM) tests and their procedures are examined.

3.1 Obtaining Soil Samples from Different Sites

In this study, there are two different soil samples which have been taken from different sites of Ankara, Turkey. They are namely, Esenboğa and Bilkent Clays. Esenboğa Clay samples have been obtained from clay deposits in the Akyurt district of Ankara, near Esenboğa Airport. Bilkent Clay samples have been obtained from clay deposits of the Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway. Disturbed and undisturbed soil samples have been taken from both sites in order to conduct laboratory tests. The maps for Turkey and Ankara are given in Figure 3.1 and Figure 3.2 respectively. Akyurt district from where the Esenboğa Clay was obtained can be seen in Figure 3.2. Bilkent Clay was obtained from the Çankaya district shown in Figure 3.2.



Figure 3.1 : The location of Ankara in Turkey.



Figure 3.2 : The location of Akyurt and Çankaya in Ankara.

Undisturbed soil samples have been taken from both sites by means of metal tube samplers without losing their moisture content. The thickness, t of the metal tube sampler is 5 mm with an outer diameter of 17 cm and an inner diameter of 16 cm and the height of the cylindrical sampler is 18 cm. Metal tube samplers are shown in Figure 3.3.



Figure 3.3 : Metal tube samplers.

3.1.1 Esenboğa site

In Figure 3.4 satellite image of Esenboğa site taken from Google Earth is illustrated. Esenboğa site is located in the Akyurt district of Ankara, near Esenboğa Airport.

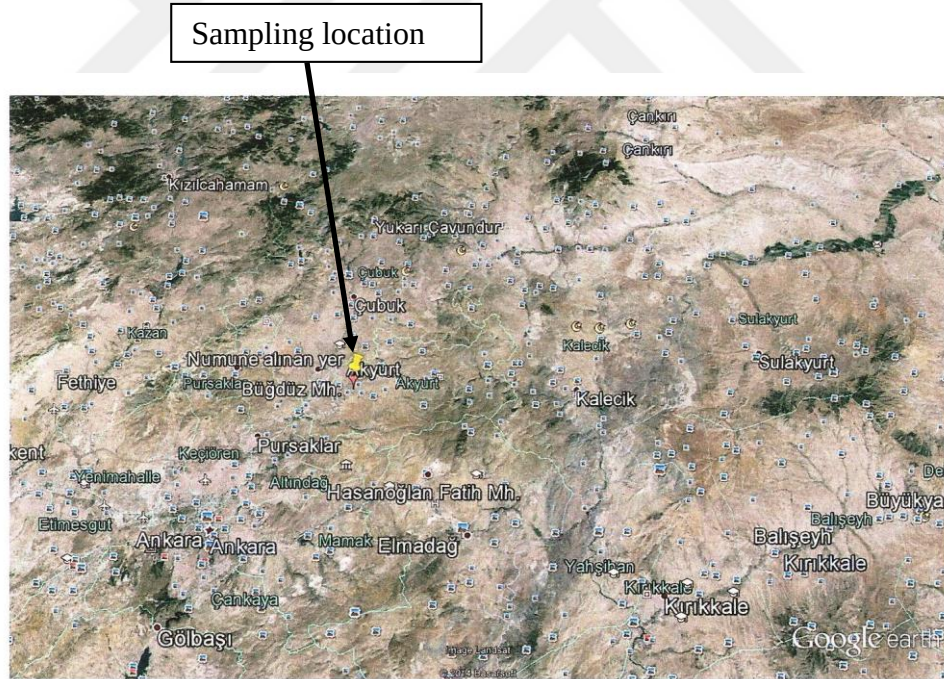


Figure 3.4 : Satellite image of Esenboğa site.

Also the local view of the site is illustrated in Figure 3.5.

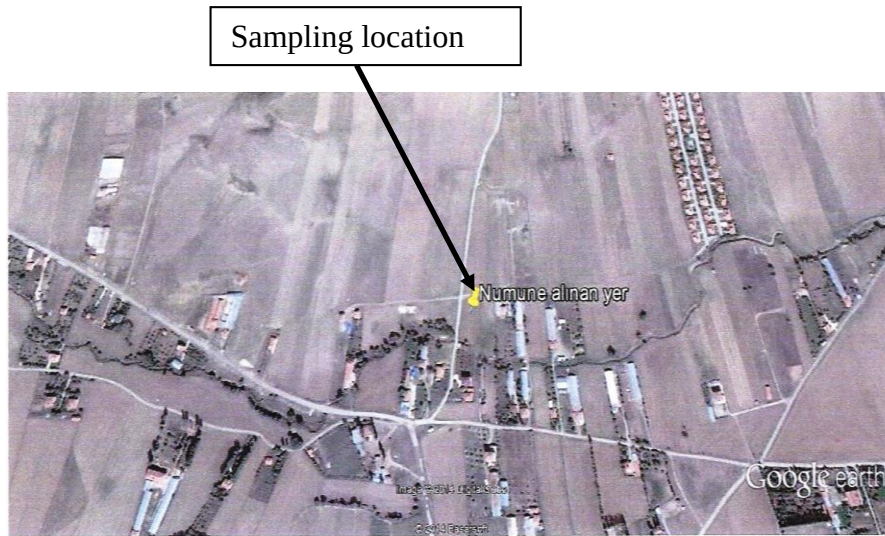


Figure 3.5 : Local view of Esenboğa site.

An excavation pit approximately 3 m depth was opened to obtain undisturbed and disturbed soil samples at Esenboğa site as shown in Figure 3.6.



Figure 3.6 : A photograph of the sampling pit at Esenboğa.

In Figure 3.7, metal tubes were pushed into the ground vertically by means of the bucket of an excavator in order to take undisturbed soil samples.



Figure 3.7 : Pushing the samplers into the ground at Esenboğa.

In Figure 3.8, metal tubes filled with undisturbed soil samples were immediately sealed with wax from top and bottom aiming to maintain their moisture content.



Figure 3.8 : Sample tubes sealed with wax.

After this process, sealed sample tubes were all put into the moisture room in the laboratory as shown in Figure 3.9.



Figure 3.9 : Sample tubes in moisture room.

Disturbed soil samples were put into the plastic bags as shown in Figure 3.10.



Figure 3.10 : Disturbed soil samples of Esenboğa Clay.

3.1.2 Bilkent site

In Figure 3.11, satellite image of Bilkent site taken from Google Earth is illustrated. Bilkent site is located in the Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway.



Figure 3.11 : Satellite image of Bilkent site.

In Figure 3.12, metal tubes were pushed into the ground vertically by means of the bucket of an excavator in order to take undisturbed soil samples.



Figure 3.12 : Pushing the samplers into the ground at Bilkent.

In Figure 3.13, metal tubes filled with undisturbed soil samples were immediately sealed with wax from top and bottom in order to maintain their moisture content.



Figure 3.13 : Sealing of undisturbed soil samples with wax.

3.2 Experimental Schedule

Within the scope of this research, laboratory experiments have been conducted in different laboratories in Ankara generally. The big deal of the tests have been performed at the soil mechanics laboratory of METU. These are the tests to determine the index properties (i.e. Atterberg Limits, sieve analyses), strength, compressibility and swelling parameters. Methylene blue tests have also been executed in METU. The other tests for mineralogical analyses have been executed at MTA (General Directorate of Mineral Research and Exploration). Resilient modulus tests have been conducted at soil laboratory of KGM (General Directorate of Highways). Finally, SEM images were obtained at the soil mechanics laboratory of Boğaziçi University (Research and Development Department-ARGE).

Firstly, several tests were conducted on clays without any agent. An extremely highly plastic clay type of soil was taken from clay deposits of Akyurt district of Ankara, near Esenboğa Airport (Esenboğa Clay) and a very highly plastic clay type of soil was taken from clay deposits of Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway (Bilkent Clay). Atterberg Limits, sieve analyses, unconfined compression tests, oedometer tests (free swell, swell pressure and consolidation tests) both for untreated Esenboğa and Bilkent Clays. Then, both Esenboğa and Bilkent Clays were mixed with lime and fly ash at different percentages. Lime has been considered as chemically active additive. Fly ash, on the other hand, is considered as filler material and its experimental results can be expected as reference values to compare them with

the results of lime. Natural soils have been mixed with hydrated lime at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%) and with fly ash at another percentages of dry weight of the soil (5%, 10%, 15%, 20% and 25%). Atterberg Limits, sieve analyses, strength, compressibility and swelling properties of the treated soil samples have been examined by increasing the lime and fly ash content steadily.

Also, the above tests were performed on lime-treated samples of Esenboğa Clay for curing periods 7, 28, 56 and 90 days (for 1%, 3%, 5%, 7% and 9% percentages).

Resilient modulus tests for both Esenboğa and Bilkent Clays with only lime agent were performed without cure and with 7, 28, 56 and 90 days cure periods for each lime percentages (1%, 3%, 5%, 7% and 9%). Resilient modulus tests were also conducted on natural clays.

Finally, 7% lime-treated sample of Esenboğa Clay for curing period of about 30 months was imaged under SEM (Scanning Electron Microscope).

Hence, the mechanism of treatment process can be explained more precisely by these data.

XRF, XRD and Rietveld tests have been performed in order to better understand the material properties of the clays that are to be treated and as well as the agents utilized in this study.

Methylene Blue Tests have been conducted on natural and lime treated soil samples for both Esenboğa and Bilkent Clays.

All the tests mentioned above can be seen in Table 3.1.

All the tests have been performed according to the ASTM Standards.

Table 3.1 : Experimental schedule in this research.

	Tests conducted	
Test condition	Esenboğa Clay	Bilkent Clay
Natural Clay (Untreated)	Index, unconfined compression*, free swell, swell pressure, consolidation, resilient modulus, SEM, Methylene Blue.	Index, unconfined compression, free swell, swell pressure, consolidation, resilient modulus, SEM, Methylene Blue.
Lime agent (1%, 3%, 5%, 7% and 9%)	Index, unconfined compression*, free swell, swell pressure, consolidation, resilient modulus, Methylene Blue, SEM (only for 7% lime content with 30-month curing period). ALL ABOVE TESTS WERE ALSO CONDUCTED FOR 7, 28, 56, 90-DAY CURE PERIODS EXCEPT METHYLENE BLUE TEST AND SEM.	Index, unconfined compression, free swell, swell pressure, consolidation, resilient modulus (without cure and for 7, 28, 56, 90-day cure periods), Methylene Blue.
Fly ash agent (5%, 10%, 15%, 20% and 25%)	Index, unconfined compression*, free swell, swell pressure, consolidation.	Index, unconfined compression*, free swell, swell pressure, consolidation.

*: unconfined compression tests were conducted for soaked and non-soaked conditions.

Note: XRF, XRD and Rietveld tests were conducted on natural clays and on the agents, lime and fly ash.

3.3 Untreated (Natural) Soil Properties

First index properties of two types of clays used in this research are given in the following tables. Then, sieve analysis results together with index properties for Esenboğa Clay and Bilkent Clay are given in Table 3.2 and Table 3.3 respectively.

Table 3.2 : Index properties of Esenboğa Clay.

G_s (Specific Gravity)	Void Ratio* e_o	Sieve analysis			Atterberg Limits				
		+No.4	- No.200	Clay Size	LL	PL	PI	Ls	SL
		(%)	(%)	(%)	(%) Liquid Limit	(%) Plastic Limit	(%) Plasticity Index	(%) Linear Shrinkage	(%) Shrinkage Limit
2.701	1.4730	0	99.8	67.5	135	34	101	28	14

Table 3.3 : Index properties of Bilkent Clay.

G_s (Specific Gravity)	Void Ratio* e_o	Sieve analysis			Atterberg Limits				
		+No.4	- No.200	Clay Size	LL	PL	PI	Ls	SL
		(%)	(%)	(%)	(%) Liquid Limit	(%) Plastic Limit	(%) Plasticity Index	(%) Linear Shrinkage	(%) Shrinkage Limit
2.737	1.040	0.5	79.2	45	109	33	76	25	9

The grain size distribution of the Esenboğa Clay is shown in Figure 3.14.

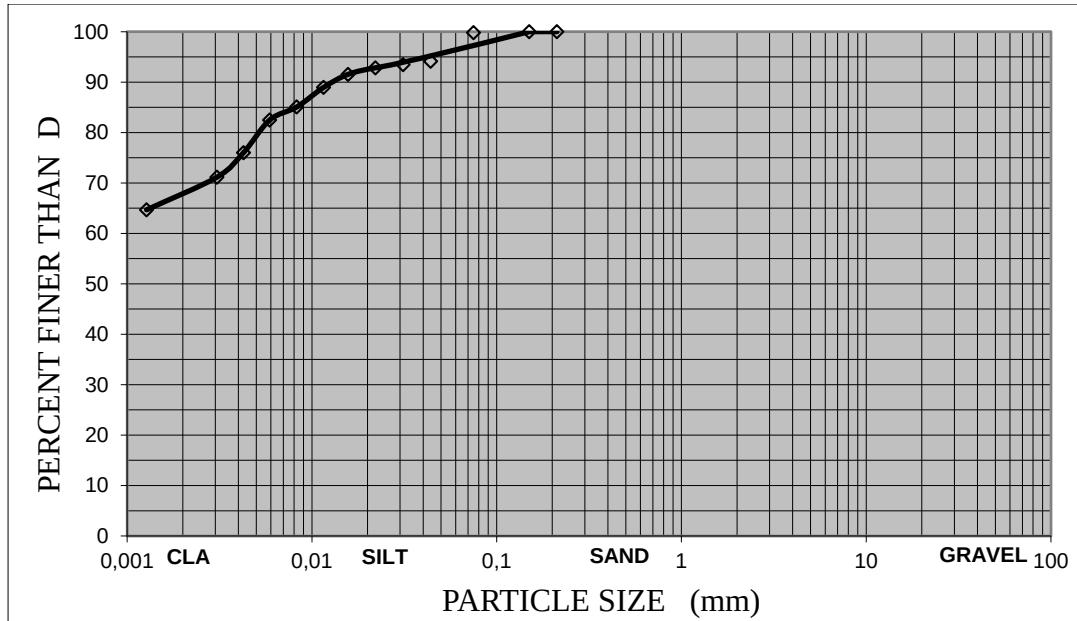


Figure 3.14 : Grain size distribution of the Esenboğa Clay.

The grain size distribution of the Bilkent Clay is shown in Figure 3.15.

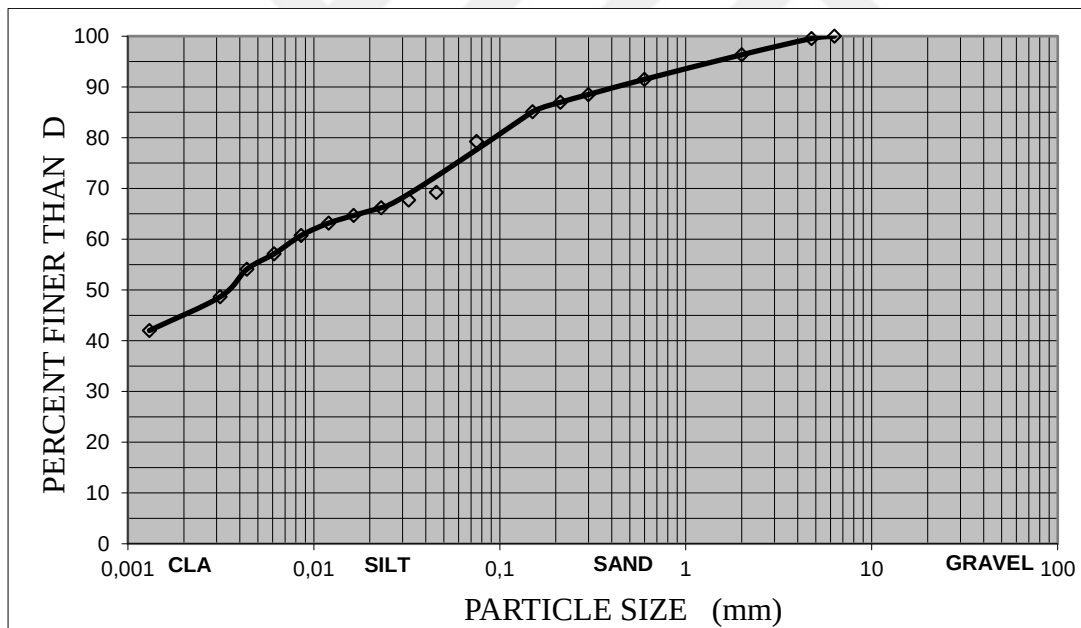


Figure 3.15 : Grain size distribution of the Bilkent Clay.

3.3.1 Mineralogy of Esenboğa Clay

The soil mineralogy of the clays used in this research has been determined by XRD and Rietveld tests that are supported by the petrographical, XRF and SEM analyses. The results are given in the subsequent section (See Appendix A for detailed results). The

tests in this part were all executed in the laboratory of MTA (General Directorate of Mineral Research and Exploration).

3.3.1.1 XRD and Rietveld tests

In this study, detailed mineralogical identifications in accordance with the ASTM Database (ICDD, 2017) and quantitative mineral analysis by utilizing appropriate software and databases are conducted by means of the standard (2° - 70°), normal (2° - 30°), ethylene glycol, 300°C thermal and 550°C thermal process imaging.

The quantitative analysis has been performed for the Esenboğa sample by using Cu-X ray tubed Panalytical X'Pert Powder XRD analysis device as well as by utilizing the Panalytical X'Pert Highscore Plus programme and Inorganic Crystal Structure Database (ICSD) with Rietveld method. Corresponding results of this analysis are given in Table 3.4 below. In the first part of this table clay and non-clay mineral quantities can be seen. The clay minerals only are quantified in the second part of the table. All these XRD analyses are supported by the petrographical, XRF and SEM analyses.

X-Ray diffractograms of Esenboğa Clay for bulk (whole) sample and XRD patterns for clay fraction are illustrated in Figure 3.16 and 3.17 respectively.

Table 3.4 : Soil mineralogy of Esenboğa Clay.

Clay and Non-Clay Minerals (%)							Clay Minerals (%)		
Quartz	Cristobalite	Calcite	Dolomite	Plagioclase	Muscovite	Clay	Smectite	Kaolinite	Illite
7	4	8	5	5	25	46	4	9	87

XRD Patterns of Esenboğa Clay for bulk (whole) sample is given in Figure 3.16 in which the clay and non-clay mineral peaks can be observed based on diffraction angle of X-Ray, 2θ .

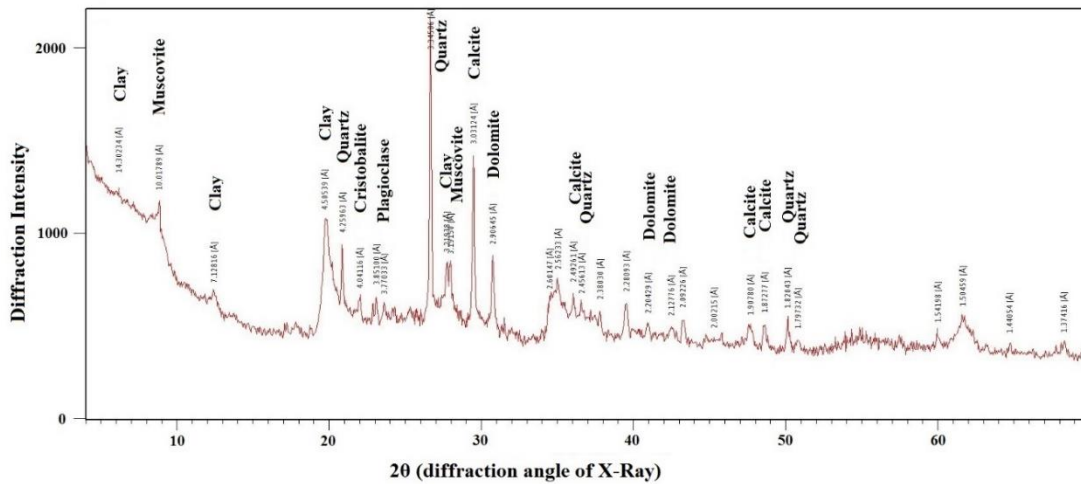


Figure 3.16 : X-Ray diffractograms of Esenboğa Clay for bulk (whole) sample.

XRD Patterns for clay fraction are illustrated in Figure 3.17 below. It should be noted that the clay fraction is composed of smectite, illite and kaolinite minerals.

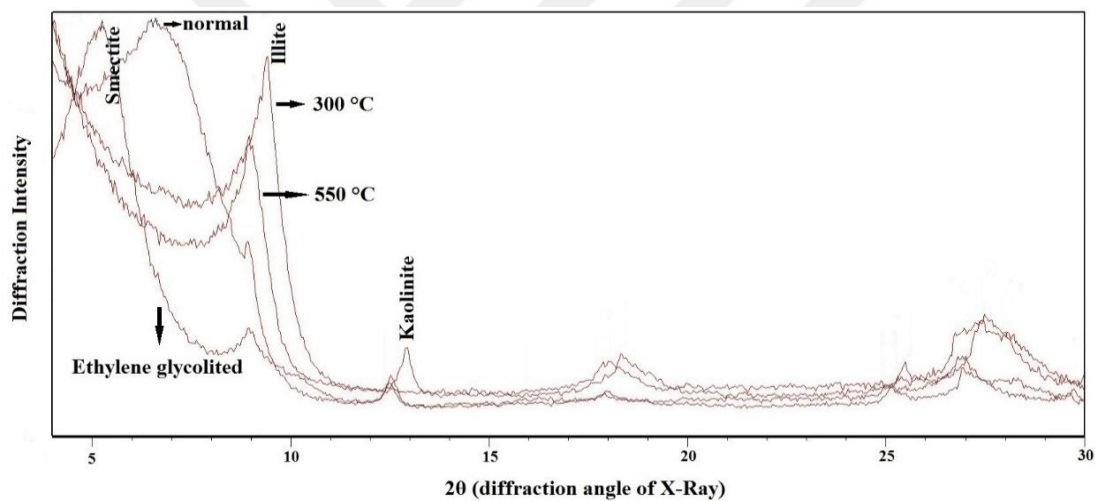


Figure 3.17 : XRD patterns for clay fraction in Esenboğa Clay.

3.3.1.2 XRF tests

Samples are prepared and analyzed as pellet structures after being dried at 65°C during one night and then at 105°C during 2 hours. These samples are studied within the range of Bor-Uranium and the results are presented in the form of oxide as shown in Table 3.5, for Esenboğa Clay.

Major components of Esenboğa Clay are given based on these analyses.

Table 3.5 : Major chemical components of Esenboğa Clay.

Chemical Elements	Esenboğa Clay (%)
SiO ₂	48.70
Al ₂ O ₃	16.70
CO ₂	11.00
Fe ₂ O ₃	7.05
CaO	6.57
MgO	4.30
K ₂ O	2.59
Na ₂ O	1.53

3.3.2 Mineralogy of Bilkent Clay

The soil mineralogy of the clays used in this research has been determined by XRD and Rietveld tests that are supported by the petrographical, XRF and SEM analyses. The results are given in the subsequent section (See Appendix A for detailed results). The tests in this part were all executed in the laboratory of MTA (General Directorate of Mineral Research and Exploration).

3.3.2.1 XRD and Rietveld tests

In this study, detailed mineralogical identifications in accordance with the ASTM Database (ICDD, 2017) and quantitative mineral analysis by utilizing appropriate software and databases are conducted by means of the standard (2°-70°), normal (2°-30°), ethylene glycol, 300°C thermal and 550°C thermal process imaging.

The quantitative analysis has been performed for the Bilkent sample by using Cu-X ray tubed Panalytical X'Pert Powder XRD analysis device as well as by utilizing the

Panalytical X'Pert Highscore Plus programme and ICSD database with Rietveld method. Corresponding results of this analysis are given in Table 3.6 below. In the first part of this table clay and non-clay mineral quantities can be seen. All these XRD analysis are supported by the petrographical, XRF and SEM analysis.

Table 3.6 : Soil mineralogy of Bilkent Clay.

Clay and Non-Clay Minerals (%)					Clay Minerals (%)
Quartz	Cristobalite	Calcite	Dolomite	Clay	Palygorskite
20	4	26	13	37	100

X-Ray diffractograms of Bilkent Clay for bulk (whole) sample and XRD patterns for clay fraction are illustrated in Figure 3.18 and 3.19 respectively.

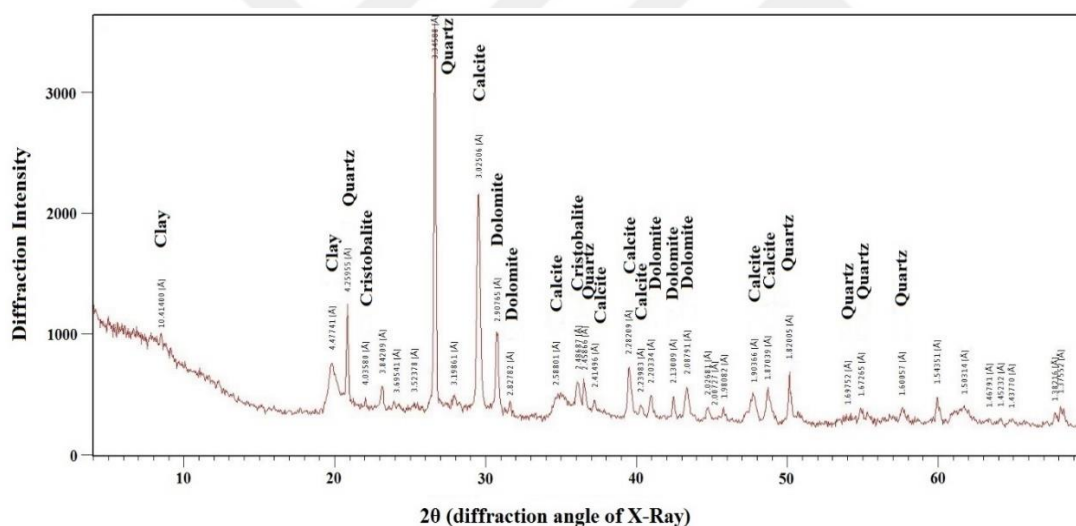


Figure 3.18 : X-Ray diffractograms of Bilkent Clay for bulk (whole) sample.

It should be noted that the whole clay minerals are composed of palygorskite in Bilkent site (Figure 3.19).

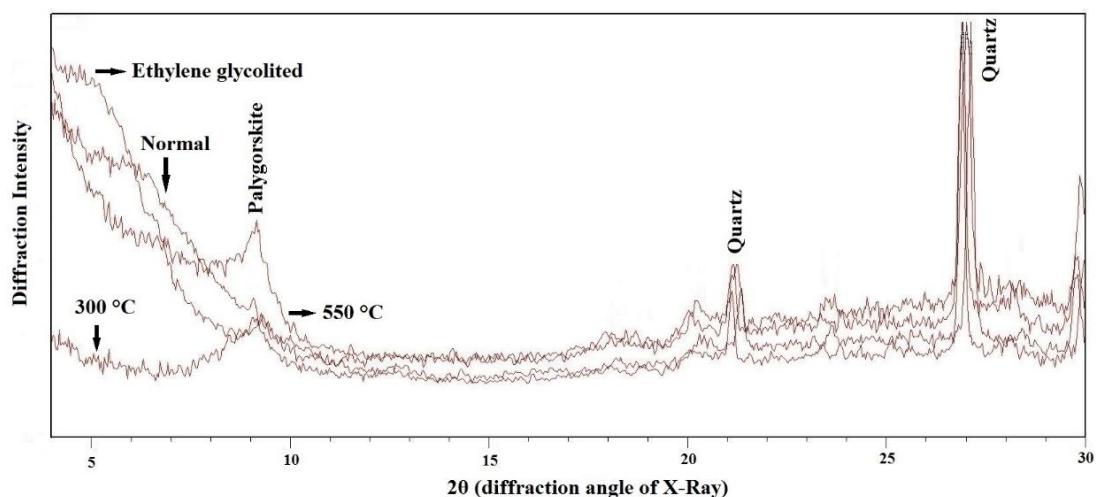


Figure 3.19 : XRD patterns for clay fraction in Bilkent Clay.

3.3.2.2 XRF tests

Samples are prepared and analyzed as pellet structures after being dried at 65°C during one night and then at 105°C during 2 hours. These samples are studied within the range of Bor-Uranium and the results are presented in the form of oxide as shown in Table 3.7, for Bilkent Clay.

Table 3.7 : Major chemical components of Bilkent Clay.

Chemical Elements	Bilkent Clay (%)
SiO ₂	40.40
CO ₂	19.80
CaO	14.70
Al ₂ O ₃	12.50
Fe ₂ O ₃	5.14
MgO	3.95
K ₂ O	1.37
Na ₂ O	0.96

3.4 Properties of Chemical Agents

Hydrated lime and Soma Class C Fly ash have been utilized as a chemical agent in this study.

3.4.1 Lime

The lime used in this research is a high quality hydrated calcium lime, produced by Baştaş Lime Factory and it satisfies the standards of TS EN ISO 9001:2008, providing the general requirements for construction purposes. XRF test has been executed in the laboratory of MTA (General Directorate of Mineral Research and Exploration) in order to determine the chemical composition of the lime.

3.4.1.1 XRF test

Samples are prepared and analyzed as pellet structures after being dried at 65°C during one night and then at 105°C during 2 hours. These samples are studied within the range of Bor-Uranium and the results are presented in the form of oxide. The major chemical composition of the hydrated lime are shown in Table 3.8.

As understood in Table 3.8, the lime used in this research is a high calcium hydrated lime. However, it can be recommended here to use lime agent, which is available, with more than 90% calcium content.

Table 3.8 : Chemical composition of hydrated lime used in treatment.

Chemical Elements	Lime (%)
CaO	71.10
CO ₂	25.80
MgO	1.77
SO ₃	0.75
SiO ₂	0.30

3.4.2 Fly ash

The fly ash used in this research was obtained from the burning coal of Soma in Manisa. The grade of this fly ash is "C" grade according to ASTM C 618.

Soma Thermal Power Plant is in the Soma district of Manisa city in Turkey. The brown coal (lignite) burned in the boilers of the power plant is supplied by Aegean Lignite Management. About 8.000.000 tonnes brown coal is burned in the power plant in a year. The amount of fly ash generated from this burning is about 3.600.000 tonnes per year. About 270.000 tonnes of this fly ash, which is used as a concrete admixture, is sold to private sector (URL-5).

Baba and Kaya (2004) investigated the leaching characteristics of Soma fly ash. They utilized two toxicity testing procedures in their research. One of them is used in U.S.A namely EPA-TCLP method. The other one is CEN method used in Europe. The results show that the classification of ash waste as hazardous and non- hazardous is procedure dependent. Test results indicated that pH is an important parameter affecting the leaching rate of metals from ash deposits. The results of EPA-TCLP method classify the Soma fly ash as “toxic waste”; whereas the results of CEN procedure classify the same ash as “non-toxic” waste.

XRF test of the fly ash used in this research has been executed in the laboratory of MTA (General Directorate of Mineral Research and Exploration) in order to determine the chemical composition of the fly ash.

3.4.2.1 XRF test

Samples are prepared and analyzed as pellet structures after being dried at 65°C during one night and then at 105°C during 2 hours. These samples are studied within the range of Bor-Uranium (BU) and Fluor-Uranium (FU); then the results are presented in the form of oxide. The major chemical compositions of Soma Fly Ash are given below in Table 3.9.

Table 3.9 : Chemical composition of Soma fly ash (BU), (FU) and typical chemical composition of both class C and F fly ashes.

Chemical Elements	Soma Fly Ash (BU) (%)	Soma Fly Ash (FU) (%)	Typical Class C (%)	Typical Class F (%)
SiO ₂	45.80	48.20	39.90	54.90
Al ₂ O ₃	26.60	27.70	16.70	25.80
Fe ₂ O ₃	4.38	4.84	5.80	6.90
CaO	11.60	12.40	24.30	8.70
MgO	1.20	1.26	4.60	1.80
SO ₃	1.79	1.91	3.30	0.60
CO ₂	5.14	-		
LOI			6	6

The fraction of fly ash (i.e. SiO₂+Al₂O₃+Fe₂O₃) is more than 50% of the total content. Since lignite coal is used in Soma Thermal Power Plant, fly ash used in this study can be classified as class C (ASTM C 618), with moderate calcium oxide - CaO (12%).

3.5 Preparation of Samples for Tests

The natural soil samples have been obtained in large diameter special moulds as mentioned before. The smaller tubes (diameter: 36 mm, length: 72 mm) are then used to get testing samples in natural conditions.

Disturbed soil samples were obtained in large heavy duty bags (i.e. about 30 kg) at the site. They were oven-dried at 60 °C in accordance with FHWA reports for treatment of samples with lime and fly ash. Then the samples were pulverized by means of a wooden

hammer. After that soils were sieved by No. 40 sieve in order to ensure that the samples are to be uniform. The soils samples were mixed with the agent until they are thoroughly blended. A high degree of homogeneity is achieved by this way. It should be noted here that a time, which is 1 hour, should be passed between the mixing and compaction process according to ASTM D3551. This time is called as “mellowing time” which can be accepted as a resting time for samples. The purpose of allowing mellowing time is to obtain homogeneous, moisture-equalized and friable mixture.

The other samples for treated soil conditions are prepared in such way that the samples are to be compacted with maximum dry density at optimum water content. Harvard miniature compaction test apparatus has been used for this purpose. This apparatus was developed by Wilson (1950) for small amount of fine grained soils. Since the size of the sample to be tested is small, many specimens can be produced in a short time. Strength and compressibility characteristics of the samples can be determined in addition to determination of index properties of the soils. The soils to be tested is sieved through No.4 sieve first in the Harvard miniature compaction method. The height and the diameter of the compaction mould are 0.0715 m and 0.033 m respectively. The mould has a cylindrical tamping foot the diameter of which is 0.0127m. The soil is compacted in three layers. The number of tamps for each layer is 25. Harvard miniature compaction test apparatus is given in Fig. 3.20.



Figure 3.20 : Harvard miniature compaction test apparatus.

3.5.1. Unconfined compression test

Unconfined compression tests in this research have been performed for non-soaked and soaked conditions. For testing natural soil samples, steel and brass tubes of internal diameter 36 mm and length 72 mm are pushed into the large diameter sampling moulds to get undisturbed samples for testing. The tips of the steel and brass tubes are sharpened for easy penetration to provide undisturbance (Figure 3.21). A manual hydraulic jack has been utilized for penetration (Figure 3.22).



Figure 3.21 : Obtaining undisturbed soil samples for unconfined compression test.



Figure 3.22 : Obtaining undisturbed soil samples by hydraulic jack.

After penetration is completed, the soil around the sampling tubes is trimmed by means of a spatula (Figure 3.23 and Figure 3.24).



Figure 3.23 : Trimming process to get undisturbed soil samples.



Figure 3.24: The undisturbed soils samples after trimming.

The natural soils have been mixed with different agents at different percentages of dry weight of the soil to test treated samples. The mixed soils have been put into a teflon mould (inner diameter: 3.6 cm, length: 14 cm) and then compacted by means of teflon rods utilizing a manually operating hydraulic jack (Figure 3.25 and 3.26). The compaction is in three stages. The lengths of the teflon rods are 6.8 cm, 9.2 cm and 11.6 cm without cap (See Figure 3.25). The length of the prepared soil sample is 7.2 cm.

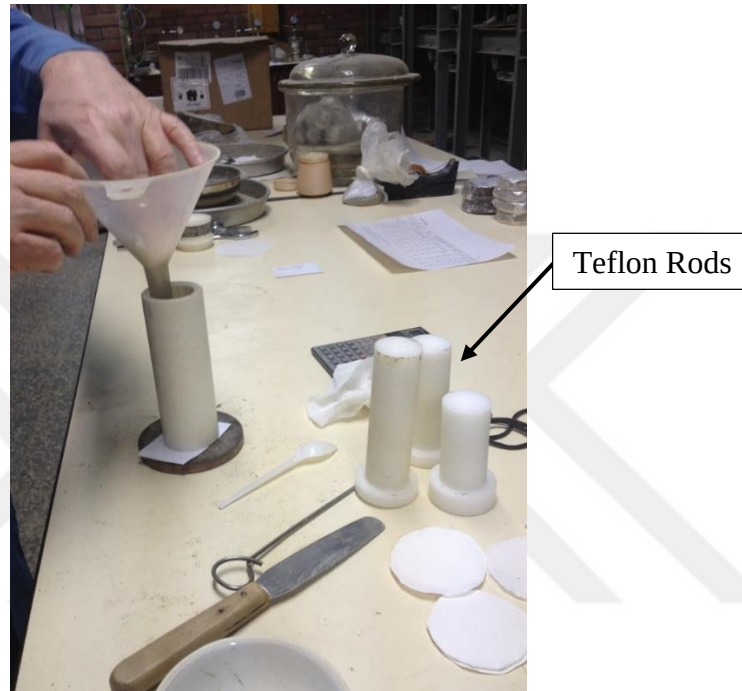


Figure 3.25 : Teflon mould and rods to get samples for unconfined compression test.



Figure 3.26 : Compaction process to get samples for unconfined compression test.

The soil inside the teflon mould is then extracted by means of a metal extractor for testing (Figure 3.27). The final soil sample is shown in Figure 3.28.



Figure 3.27 : Extracting of soil samples for unconfined compression test.



Figure 3.28 : Soil sample for unconfined compression test.

The two pieces perforated teflon mold is shown in Figure 3.29 for preparation of soaked samples. The diameter of perforations is 2.25 mm. There are 154 no. perforations in one mold. The inside of the perforated mold including top and bottom is covered by a filter paper to prevent hole clogging (Figure 3.30 and 3.31). There are porous stones at the top and bottom of the mold for proper drainage.

The perforated teflon consists of two separate parts which are held together with a screw clamp ring. Samples are compacted in maximum dry density and optimum water content. At the end of the inundation process, the sample is taken out by dismantling the screw. The specimen is cut so that the standard height to diameter ratio, is preserved according to ASTM D 2166.



Figure 3.29 : Apparatus used for preparation of the samples for unconfined compression test (soaked condition).



Figure 3.30 : The mould with filter paper for unconfined compression test (soaked condition).



Figure 3.31 : Filter paper covering soil samples for unconfined compression test (soaked condition).

The soil samples in water-filled glass beakers are shown in Figure 3.32. The weight of the plates at the top is enough to apply a minimum pressure of 1 kPa as stated in ASTM D 4546. The inundation process is accepted to be completed at 100% swelling. The readings of dial gauges should be compatible with each other at this stage. The sensitivity of dial gauges is 0.01mm. This process is ended when the dial gauge readings for both of the samples are same in successive 12 hours.



Figure 3.32 : Inundation process of the samples for unconfined compression test (soaked condition).

3.5.2. Oedometer test

The prepared soil samples are compacted in three stages. The top of each soil layer is roughened as shown in Figure 3.33 and Figure 3.34. Two different teflon rods, whose heights are different, are used for compaction for which a manually operating hydraulic jack is utilized (Figure 3.35). The height and the inner diameter of the ring is 1.9 cm and 6.35 cm respectively (Figure 3.36). The outer diameter is 8.34 cm.

Test samples are placed in 6 cm height glass containers (Figure 3.37).

Filter papers are placed both at the bottom and top of the samples (Figure 3.38). One of the filter papers is at the top of the test sample and two porous stones present over this upper filter paper. Whereas, one porous stone is placed below the lower filter paper at the bottom. Before conducting the experiment, these porous stones are de-aired and saturated in boiling water. Over the top of the porous stone, a metal covering is found and rings are put in the modified Bishop device. On the top of this metal cover, deflection dial gauge is placed vertically. Once the first dial gauge recording is obtained, the test sample is inundated and in order to avoid the possible effects of anions and cations, distilled water is utilized during this process of inundation.

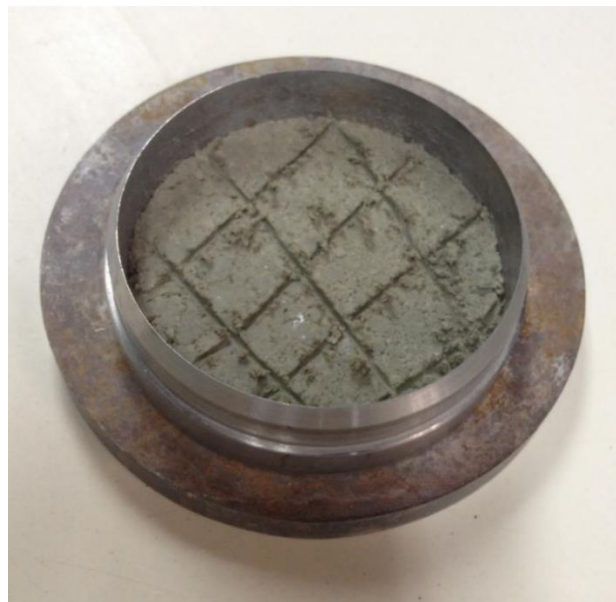


Figure 3.33 : Roughened soil samples for oedometer tests of Esenboğa Clay.



Figure 3.34 : Roughened soil samples for oedometer tests of Bilkent Clay.



Figure 3.35 : Compaction of soil samples for oedometer tests.



Figure 3.36 : Compacted soil sample in oedometer ring.



Figure 3.37 : Oedometer ring in glass container.



Figure 3.38 : Filter paper on the sample for oedometer tests.

3.5.3. Resilient modulus test

The resilient modulus test samples are prepared by using 3 apparatus manufactured in different heights that are suitable for compacting the soil in three layers by means of a hydraulic compacter as shown in Figure 3.39. The diameter of the apparatus is 10 cm and the heights are 6.66 cm, 13.32 cm and 20 cm respectively. Then, the diameter and the height of the resilient modulus test sample are 10 cm and 20 cm respectively. They are made of wood and are screwed to the square metal plates.

The metal compaction mold consists of three separate parts which are held together with a screw clamp ring. Samples are compacted in maximum dry density and optimum water content. At the end of the compaction process, the sample is taken out by dismantling the screw.



Figure 3.39 : Apparatus of resilient modulus test samples.

Prepared samples are kept in the moisture room during the curing periods as shown in Figure 3.40.



Figure 3.40 : Prepared soil samples of resilient modulus test for different cure periods in the moisture room.

After the compaction, the soil sample is placed at the bottom of the cell. A porous disc should be placed between the sample and the cell bottom. Then, the sample is confined with a membrane as shown in Figure 3.41. At the upper part of this confined sample, a porous disc is placed and also at the upper part of this disc, a plate is placed which enables the loading. Two O-rings are attached to both upper and lower plates in order to provide the imperviousness of the membrane. Since the resilient modulus test is performed in drained conditions, drainage cables are attached to the upper loading plate. Drainage channels should be open during the test. Then, the cell is placed and fixed to the bottom plate. The cell and the sample which are ready for the experiment are placed to the resilient modulus test apparatus. While placing the cell into the apparatus, a full centration should be provided between the loading piston and the loading cell which is crucial for applying the load to the sample properly. E/P control unit is attached to the cell in order to apply the lateral load to the sample. It is essential to switch on the hydraulic power engine at least 15 minutes before the beginning of the test in order to heat the hydraulic oil so that a proper test can be performed.

After all the above-mentioned procedures, the required data for the test sample are entered to the corresponding experimental software. The most important parameters that are introduced to the software are the loading series and the loading values. Resilient modulus test software suggests 16 different loading series in which haversine load form is used. For these 16 loading series, different deviator stresses and lateral pressure values are utilized. The fundamental reason that lies behind this fact is the difference among the material locations at the cross-section of the highway.



Figure 3.41 : Soil sample confined by the membrane for resilient modulus test.

Resilient modulus test equipment is composed of a loading frame, cell, hydraulic power unit, E/P control unit, signal conditioning unit, servo-valve and a computer as shown in Figure 3.42 and Figure 3.43.

Loading frame includes a control system and members that apply load to the sample and the devices for the load-displacement measurement. Cell is basically the part in which the test sample is placed.

The unit that produces the load that is to be applied to the sample is called the hydraulic power unit, whereas the E/P control unit provides the control of the lateral pressure that emerges throughout the cell.

Signal conditioning unit controls the load cell and the displacement measurement devices so that they can work properly and it includes electronic circuits that provides communication between these elements.

Servo-valve regulates hydraulic pressure for the production of sinusoidal load. The computer is necessary for the link with resilient modulus apparatus.



Figure 3.42 : Resilient modulus test loading frame.



Figure 3.43 : Resilient modulus test equipment.

3.5.4. Methylene blue test

Preparation of the methylene blue solution:

10±1 grams of methylene blue is added to 1 liter of distilled or demineralized water at room temperature, and then they are mixed very well using a mixer, as illustrated in Figure 3.44.



Figure 3.44 : Preparation of methylene blue.

Preparation of the soil solution:

Oven-dried 30 grams of soil sample which passes from No.40 sieve is put into a beaker and then 200 cc of distilled water is added and mixed by means of a mixer at a speed of 700 revolutions per minute (rpm). Then, the speed shall be decreased to 400 rpm and this speed should be kept constant till the end of the test, as shown in Figure 3.45.



Figure 3.45 : Methylene blue test.

3.5.5 SEM (Scanning Electron Microscope) analyses

Sample preparation:

Since conventional SEM samples are required in this study, the samples have to be dry, vacuum compatible, and electrically conductive (i.e. coated by a suitable material like gold or carbon). Therefore, the stub-mounted samples must be oven-dried over-night before coating is applied because clay minerals are quite hygroscopic. The samples should be kept dry after being coated to prevent rupture of the conductive layer and to shorten the microscope's pump-down time (Bohor and Hughes, 1971).

There are several methods to prepare specimens for SEM analyses such as oven-drying, freeze cut drying, epoxy sublimation method or vacuumed oven drying.

In this research, 3 methods have been tried to prepare samples to be tested in the SEM machine as explained below.

In this study, the natural undisturbed soil samples have been taken by steel tubes which were later waxed and stored in moisture room. The other disturbed and lime-treated soil

samples, also waxed, have been maintained in a dessicator for about 30 months so that they are effectively cured.

First, the soil samples have been cut so that they can fit in 3cm diameter molds. They have been placed into the molds after cutting. After that a mixture consisting of 15 ml of resin and 2 ml of hardener epoxy have been poured into the molds. At this stage, the moulds have been put in an impregnator for 10 minutes under the pressure of 0.2 bar to evacuate the air bubbles as shown in Figure 3.46. After waiting for 1 day, the samples have been extracted. However, since the natural water content of the natural samples are high the samples have been maintained in a hot plate not exceeding 50 °C for another 2 hours. Then, the epoxy and impregnator stage have been repeated. After extracting the samples from the molds again, their surfaces have been sanded continuously using 120, 600, 800 and 1200 mesh grit diamond wheels respectively so that the sample surface has been reached (Figure 3.47). Finally, the samples have been cut by means of a special rotating disc without using water. The resulting samples are cubes for which the volume is not greater than 1 cm³.

Since the clay samples are generally problematic materials for coating the method explained above was unsuccessful. This can be due to the partially saturated samples that could not be dried completely or volume might be bigger than it should be for coating. The cube samples are given in Figure 3.48.



Figure 3.46 : SEM samples in impregnator.



Figure 3.47 : Sanding process of samples.



Figure 3.48 : Cube SEM samples.

In the second method, the soil samples have been maintained in a mica vacuumed dessicator for 3 days (Figure 3.49). Then they have been put in an oven for which the temperature is not greater than 40 °C because of mica material of dessicator. The coating process was unsuccessful again due to the probable reasons mentioned above.



Figure 3.49 : Vacuumed dessicator.

Finally, conventional method has been used (i.e. drying the samples in an oven not more than 50 °C). This limit for temperature is required in order not to remove structural water from clay minerals. This method was successful for coating at the end.

The next step is to get a conductive sample which is essential for SEM analysis. Coating process can be performed by several materials such as carbon, gold, or gold-palladium alloy. Carbon coating has been used in this study. Coating process has been carried out by means of a special coater machine.

3.6. Definitions and Method Statements for Tests

The definitions of all tests and the way of executing these tests are explained in this section. First, the index tests for physical description of soil samples are explained. Then, the strength, the swelling and the compressibility tests are examined in detail.

3.6.1. Index tests

Atterberg limit tests (ASTM D4318-10, D427-04), specific gravity (ASTM D854) test and hydrometer test (ASTM D422) were performed in order to determine the index properties of the natural and treated clays used in this research.

The Atterberg limits are the water contents at which the soil mass passes from one state to the next state. These limits are presented as percentages of moisture present inside the soil. The Atterberg limits which are commonly used for engineering purposes are: a) Liquid limit, b) Plastic limit, and c) Shrinkage limit.

Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil. It is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing.

Plastic limit is the water content corresponding to an arbitrary limit between the plastic and semi-solid states of consistency of a soil. It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3 mm in diameter.

The term shrinkage limit, expressed as a water content in percent, is typically assumed to represent the amount of water required to fill the voids of a given cohesive soil at its minimum void ratio obtained by drying (usually oven). Thus, the concept shrinkage limit can be used to evaluate the shrinkage potential or possibility of development, or both, of cracks in earthworks involving cohesive soils (ASTM D427-04). Linear shrinkage, on the other hand, is the decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

Mercury method was used in this research for the determination of shrinkage limit (Figure 3.50). The photographs of the linear shrinkage test are given in Figure 3.51, Figure 3.52 and Figure 3.53.

Hydrometer analysis is a widely used method of obtaining an estimate of the distribution of soil particle sizes from the No. 200 (0.075 mm) sieve to around 0.01 mm. The hydrometer analysis is based on Stokes' Law, which gives the relationship among the velocity of fall of spheres in a fluid, the diameter of the sphere, the specific weights of the sphere and of the fluid, and the fluid viscosity.



Figure 3.50: Preparation of samples for shrinkage limit test.



Figure 3.51 : Preparation of samples for shrinkage limit tests.



Figure 3.52 : Oven-dried soil sample of linear shrinkage limit test.



Figure 3.53 : Oven-dried soil sample of shrinkage limit test.

3.6.2. Unconfined compression tests

This test is conducted in accordance with ASTM D 2166. Strain controlled loading is adopted. The shearing strain rate is 0.7 mm/min. The unconfined compressive strength, q_u is calculated as the maximum load per unit area. If the maximum load can not be reached, 15% axial strain is taken as strength calculation criterion. The loading frame for unconfined compression test is given in Figure 3.54. A failed sample in the test can be seen in Figure 3.55.



Figure 3.54 : Unconfined compression test loading frame.



Figure 3.55 : A failed sample in unconfined compression test.

The samples under water for inundation process for soaked tests in unconfined compression test are shown in Figure 3.56.



Figure 3.56 : Inundation process of the samples for unconfined compression test (soaked condition).

3.6.3. Oedometer tests

3.6.3.1. Free swell test

This test is conducted in accordance with ASTM D 4546-08 (Method B). Test samples are placed in 6 cm height rings. The inner diameter of these rings is 63.5 mm and the outer diameter is 83.4 mm.

Filter papers are placed both at the bottom and top of the samples. One of the filter papers is at the top of the test sample and two porous stones present over this upper filter paper. Whereas, one porous stone is placed below the lower filter paper at the bottom. Before conducting the experiment, these porous stones are de-aired and saturated in boiling water. Over the top of the porous stone, a metal covering is found and rings are put in the modified Bishop device. On the top of this metal cover, deflection dial gauge is placed vertically. Once the first dial gauge recording is obtained, the test sample is inundated and in order to avoid the possible effects of anions and cations, distilled water is utilized during this process of inundation.

Deflection values in the dial gauge are recorded with time. These are obtained for 0.1, 1, 2, 4, 8, 15, 30, 60 minutes, 2, 4, 8 hours, 1, 2, 3, 4 and finally 8 days of time periods. Once the recorded deflections stop or are less than 0.01 mm/day, the final reading of swell is recorded and the oedometer is dismantled. Then, filter papers are taken off from

the surface of the test samples. Final weight of the test sample is measured. The sensitivity of the dial gauges used in all oedometer tests is 0.01 mm. Free swell test frame is given in Figure 3.57.



Figure 3.57 : Free swell test.

3.6.3.2. Swell pressure test

Preparation of the samples and test set up are same as given in the free swell test explanation above.

To begin the test, at least 1 kPa vertical seating pressure is applied on the soil sample for half an hour. After this loading, an initial vertical stress that represents the estimated vertical pressure for in-situ conditions is applied on the sample for the next half an hour. The void ratio that is observed at this step of loading is called as the in-situ void ratio. The soil specimen is immediately inundated and vertical stress increments are applied to avoid swelling. When the soil specimen is inundated at initial stress level, the range of the deformation reading shall be kept between 0.005 mm and 0.01 mm. The specimen is loaded until no more tendency to swell is observed. Then the inundated soil specimen is loaded in increments which enable to observe the maximum curvature and to calculate the slope of the virgin compression curve. The value of the swell pressure can be determined from the void ratio (e) versus log pressure (kPa) curve. The test frame is same as given in Figure 3.57.

3.6.3.3. Consolidation test

This test is conducted in accordance with ASTM D 2435-96. Test samples are placed in 6 cm height rings. The inner diameter of these rings is 63.5 mm and the outer diameter is 83.4 mm.

Samples are subjected to time-dependent loading during oedometer testing. A seating load of 5 kPa is applied at the beginning of the test to provide an appropriate contact between top cap and porous stone. The maximum pressure applied in this study was 1600 kPa in order to see the consolidation behavior at large loads. Vertical static loads of 25, 50, 100, 200, 400, 800 and 1600 kPa with a load increment ratio of $\Delta\sigma/\sigma=1.0$ (where $\Delta\sigma$ = increase in pressure and σ = pressure before the increase) are applied to all samples.

The unloading is performed using the steps, 400, 100 and 25 kPa. Since a large amount of consolidation of samples occurs rapidly (i.e., within about 60 seconds), the duration of the load increment in this study is kept as 121 minutes, instead of a standard 24 hours. The consolidation test set-up in the laboratory is shown in Figure 3.58 below. Oven-dried samples of consolidation test are in Figure 3.59.



Figure 3.58 : Consolidation test.



Figure 3.59 : Oven dried samples of consolidation test.

3.6.4. Resilient modulus test

This test is conducted in accordance with AASHTO T 307. The sample compacted by using static compactor is placed to the resilient modulus apparatus. In this test, 16 different loading series are applied to the sample in which the Haversine load form is utilized. The first of these 16 different loading series is the phase of conditioning. In this phase, 1000 cycles of loading are fulfilled in order to eliminate the permanent deformation at the sample that might emerge during the experiment, to provide the contact between the sample caps and the sample and to avoid the potential problems that might arise during the compaction of the sample. For each of the other 15 loading series to determine the value of the resilient modulus, the number of cyclic loading is 100. Each of the loading, both for the 1000 cycles in the conditioning phase and for the hundreds of loading repetitions in the other 15 loading series, is executed by applying the vertical load to the sample for 0.1 seconds and by not applying the vertical load for 0.9 seconds.

The loadings that are applied during the experimental procedure are in sinusoidal form in Haversine loading form as shown in Figure 3.60. The axial load, which is applied for providing the contact between the loading piston and the sample, is called P_{contact} . The maximum load is (P_{max}) is equal to the sum of the contact load P_{contact} and the applied periodical load P_{periodic} . The equation of this loading that is used in the resilient modulus test is given as follows:

$$\text{Haversine Load} = \frac{(1 - \cos\theta)}{2}$$

where, θ is time.

The capacity of the hydraulic pump is 40000 kPa which should be sufficient for sudden loadings. The experiment lasts for approximately 45 minutes after the initiation of loading process.

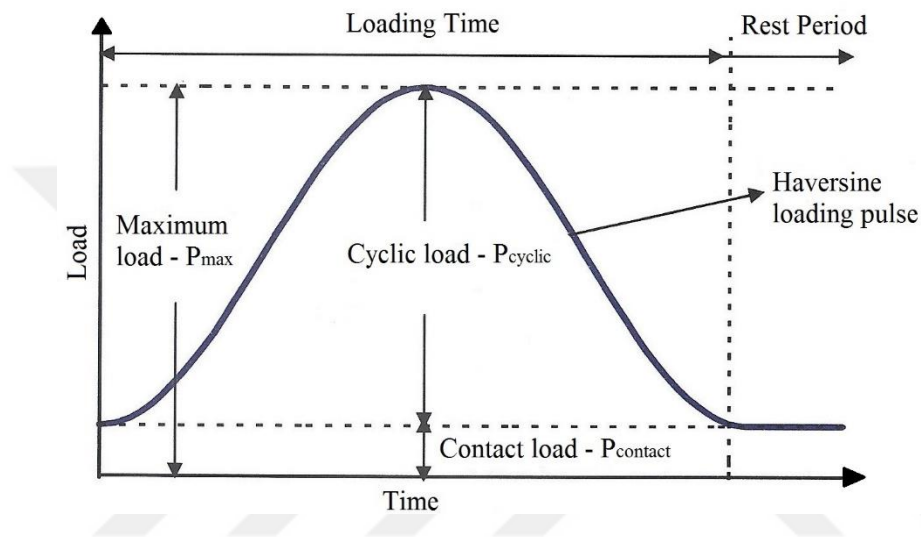


Figure 3.60 : Haversine loading pattern for resilient modulus test.

The strains under repeated loading for resilient modulus test is shown in Figure 3.61. Elastic and plastic strains can be observed in this figure.

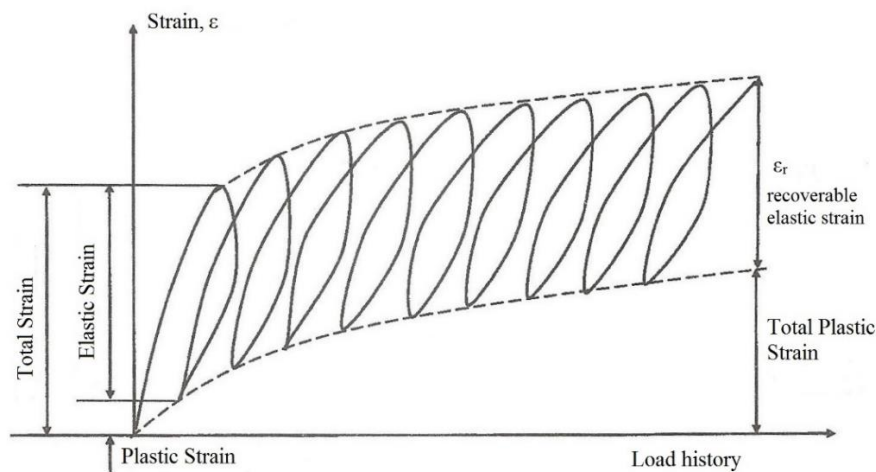


Figure 3.61 : Strains under repeated load in resilient modulus test.

3.6.5. Methylene blue test

Methylene Blue test is performed according to NF P 94- 068 (The Association Française de Normalization) in order to determine the activity of a clay. The basic principle of the test is the exchange of cations with active clay cations in the methylene blue solution. It gives an idea about the swell potential, index properties and specific surface area. This test enables to measure the capacity of the clay to adsorb the methylene blue. The methylene blue is especially adsorbed by the clay, organic material and iron hydroxydes. This capacity indicates the surface activity of these elements. The quantity of methylene blue which is adsorbed by 100g of clay is called “the value of blue”.

Preparation of the methylene blue solution:

10±1 grams of methylene blue is added to 1 liter of distilled or demineralized water at room temperature, and then they are mixed very well using a mixer.

Preparation of the soil solution:

Oven-dried 30 grams of soil sample which passes from No.40 sieve is put into a beaker and then 200 cc of distilled water is added and mixed by means of a mixer at a speed of 700 revolutions per minute (rpm). Then, the speed shall be decreased to 400 rpm and this speed should be kept constant till the end of the test.

Test procedure:

After the preparation of the solutions, an injector having a capacity of 5 ml methylene blue solution is used to add the methylene blue solution into the soil solution step by step. At first, in each step, 5 ml of methylene blue solution is added to the soil solution and this mixture is mixed for approximately 1 to 2 minutes. Then a drop is taken from the solution by means of a burette and left on a filter paper (German made Schleicher and Schuell, diameter: 125 mm) and a dark blue spot occurs on the paper. 5 ml of methylene blue solution is continued to be added until there exists a light blue halo that surrounds the dark blue spot occurs. The mixture is checked at 1 minute intervals with 5 drops after halo having a 1 mm thickness is seen. Provided that the halo shape vanishes in one of the drops, the test continues with methylene blue solution of 2 ml by following the same procedure.

Methylene Blue value (MBV) is calculated as follows:

$$\text{MBV} = V_{cc} / f'$$

where, MBV is for 100 grams of soil. V_{cc} (cm^3) is the volume of the methylene blue solution injected to the soil solution. f' (g, grams) is the dry weight of soil sample used.

Adsorption of methylene blue by clay minerals (Çokça, 1991):

The chemical formula of the methylene blue ion can be written as $\text{C}_{16} \text{H}_{18} \text{N}_3 \text{S} \text{Cl}$ (Figure 3.62).

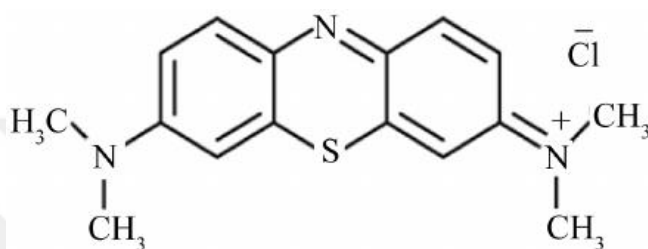


Figure 3.62 : Chemical composition of methylene blue ion.

When methylene blue reacts with a clay solution, calcium clay plus calcium chloride converts into methylene blue clay plus calcium chloride. Methylene blue cations and chloride anions are formed when methylene blue dye is dissolved in water. After that, these cations and anions can replace the existing cations associated with a given clay.

When a solution of methylene blue hydrochloride is added to a sample of clay, the amount of dye adsorbed is equal to the amount added until the cation exchange capacity (CEC) of the clay has been reached. Beyond this point of CEC the dye added is not entirely adsorbed. The drawing shown in Fig. 3.63 illustrates this phenomenon. Light blue halo shows that the excess dye cannot be adsorbed. The soil is fully saturated with methylene blue at this point.

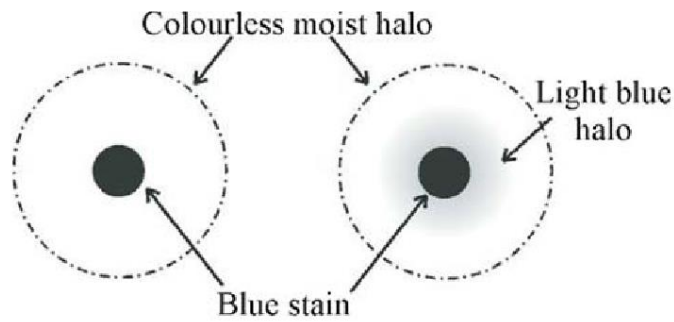


Figure 3.63 : Adsorption of methylene blue by clay minerals.

Surface area (internal and external) of the clay minerals also affects the methylene blue adsorption. For instance, montmorillonite clay mineral has quite high surface area than the kaolinite mineral. Thus, it can adsorb considerably more methylene blue than the kaolinite mineral. It should be noted here that the methylene blue is accepted to be permeated through the soil particles as monomolecular layer. The flow chart for the above process is given in Figure 3.64 (Çokça, 2002).

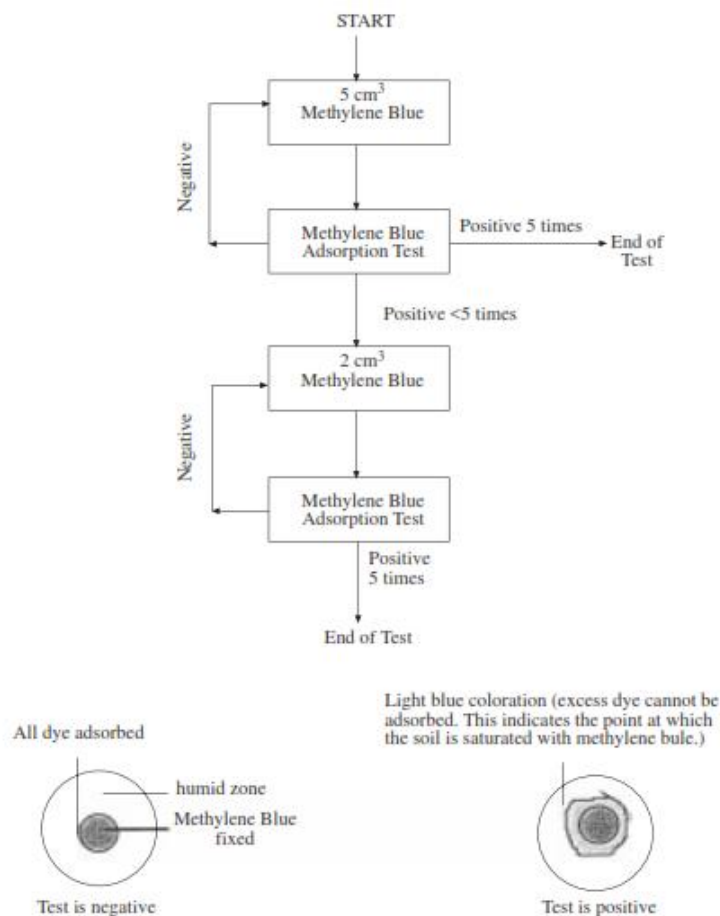


Figure 3.64 : Flow chart for the methylene blue test.

The specific surface is defined as the surface area per gram of mass (SSA). It ranges from 5 to 800 m²/g for clays, 1 to 0.4 m²/g for silt and 0.04 to 0.001 m²/g for sand. The ability of a mineral to adsorb external cations is known as cation exchange capacity (CEC). The CEC of a soil is the number of cations in milliequivalents that neutralize one hundred grams of dry clay (meq/100g). Typical values of SSA and CEC for some clay minerals are given in Table 3.10 (Mitchell and Soga, 2005).

Table 3.10 : Typical values of SSA and CEC.

Clay Mineral	Cation Exchange Capacity (CEC, meq/100g)	Specific Surface Area (SSA, m ² /g)
Kaolinite	1-6	5-55
Illite	15-50	80-120
Montmorillonite	80-150	600-800

Note: SSA for attapulgite is given between 140-170 m²/g in Santamarina et al. (2002).

3.6.6 SEM (Scanning Electron Microscope) analyses

SEM analysis has been used to look at the microstructure of samples. This type of analysis is a relatively quick way to investigate the microstructure of a soil sample. The soil sample is prepared and mounted on a stub and placed in the chamber of a machine with SEM capabilities. Stub is shown in Figure 3.65.

In this study, FEI-Philips XL30 Environmental Scanning Electron Microscope with Field Emission Gun (Equipped with EDAX-Energy Dispersive X-ray Analysis Unit) machine has been used. Although morphological aspects of the samples can be determined in detail by SEM analysis, it is possible to get chemical composition of the sample by EDAX equipment.

During the SEM analysis, the observation lens can be rotated in any way that different areas of the sample can be focused. A variety of images can be produced under different magnifications. In this study, the maximum magnification is 5000.

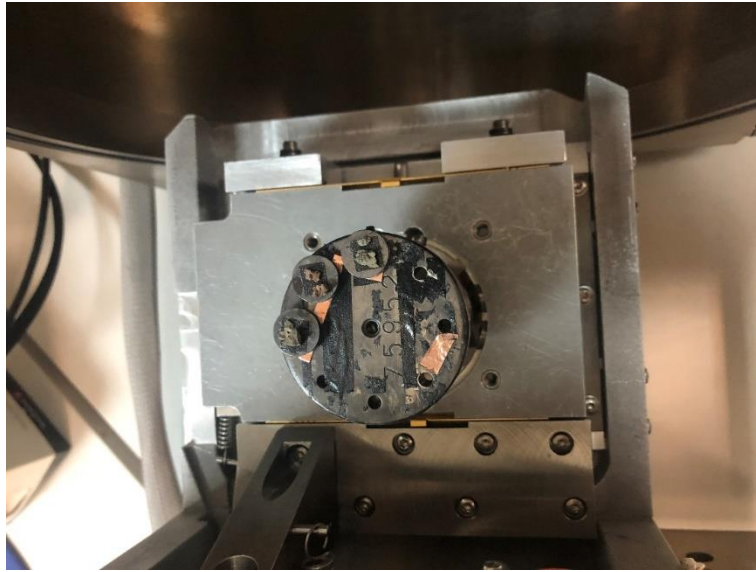


Figure 3.65 : Coated specimens on stub.



4. TEST RESULTS

The test results for natural clays and treated samples are all given in the following sections. There are two types of clays from different sites in Ankara (Esenboğa and Bilkent Clays) in this research. Lime and fly ash agents have been used for stabilization.

4.1. Properties of Esenboğa Clay

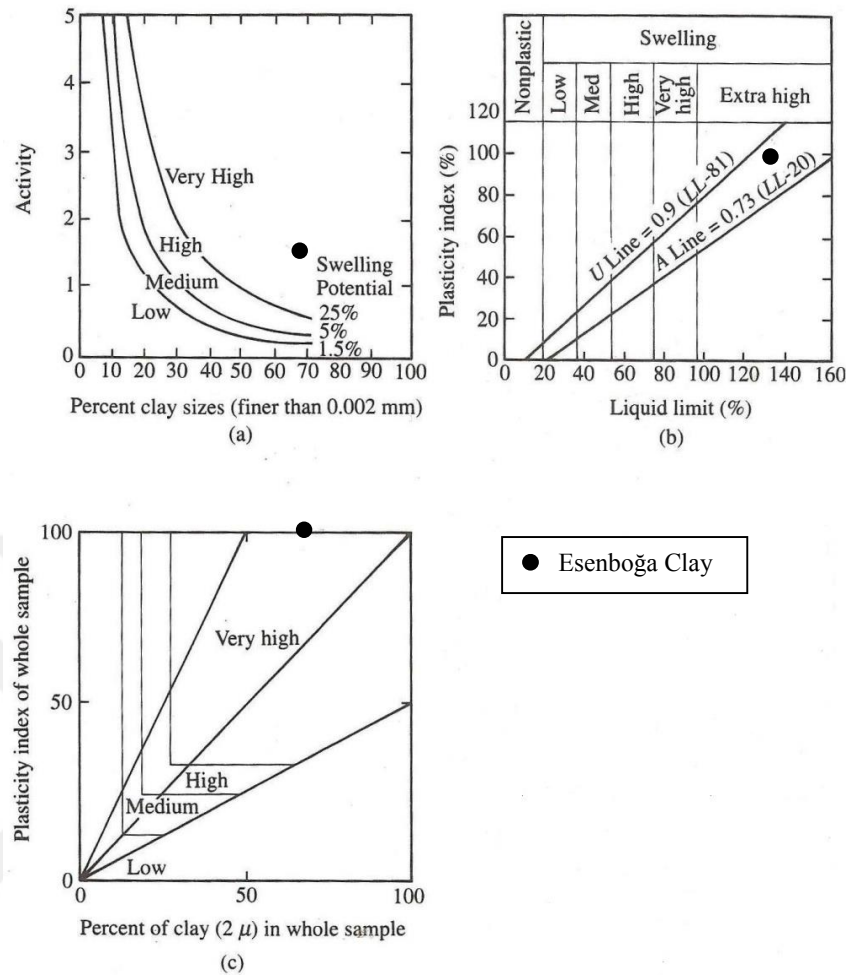
Soil properties of the Esenboğa clay are given in Table 4.1 below. Laboratory experiments have been conducted both on disturbed and undisturbed clay samples. As seen in Table 4.1, the clay is extremely highly plastic ($PI=101\%$). Clay-size fraction is 67.5%. Also, it is noticed that the clay is quite active (i.e., activity, $A=101/67.5=1.5$). Therefore, swell potential of Esenboğa clay is “very high” when the Atterberg Limits, clay-size fraction and activity are taken into account as given in Figure 4.1 for 0% lime content (i.e. untreated soil) (Das, 1999). As seen in these figures, the swell potential of natural clay (i.e. for 0% agent) is “very high”. This is also confirmed by the swell pressure and percentage values found via oedometer tests (Table 4.1). It should be noted here that the natural water content is 53%. Since this water content is more than the optimum water content ($w_{opt.}=39.5\%$), the swell percentage and swell pressure of the undisturbed sample are quite “low” (i.e., swell percentage=2%, swell pressure=30 kPa). However, swell percentage and swell pressure are “very high” for the remoulded samples prepared at optimum water content and maximum dry density as given in Table 4.1.

Table 4.1 : Properties of Esenboğa Clay.

		Sieve analysis			Atterberg Limits				
G _s (Specific Gravity)	Void Ratio* e _o	+No.4	No.200	Clay Size	LL	PL	PI	Ls	SL
		(%)	(%)	(%)	(%) Liquid Limit	(%) Plastic Limit	(%) Plasticity Index	(%) Linear Shrinkage	(%) Shrinkage Limit
2.70	1.47	0	99.8	67.5	135	34	101	28	14
Unconfined Comp. Test *	Unconfined Comp. Test *	Proctor Test		Swell	Swell	Permeability Coeff.*			
q _u	q _u	γ _{drymax}	W _{opt.}	Pressure**	Percentage**	k			
(kPa) (Non-soaked)	(kPa) (Soaked)	g/cm ³	%	(kPa)	(%)	(m/sec)			
74	27	1.16	39.5	408.9	35.5	5.95 x 10 ⁻¹⁰			

*Experiments have been performed on undisturbed samples.

**Experiments have been performed on remoulded samples prepared at optimum water content and maximum dry density.



- (a) Seed et al. (1962)
 (b) Dakshanamurthy and Raman (1973)
 (c) Van der Merwe (1964)

Figure 4.1 : Criteria for determining swell potential (Das, 1999).

4.2. Properties of Bilkent Clay

Soil properties of the Bilkent clay are given in Table 4.2 below. Laboratory experiments have been conducted both on disturbed and undisturbed clay samples. As seen in Table 4.2, the clay is very highly plastic ($PI=76\%$). Clay-size fraction is 45%. Also, it is noticed that the clay is quite active (i.e., activity, $A=76/45=1.7$). Therefore, swell potential of Bilkent clay is very high when the Atterberg Limits, clay-size fraction and activity are taken into account as given in Figure 4.2 for 0% lime content (i.e. untreated soil) (Das, 1999). As seen in these figures, the swell potential of natural clay (i.e. for 0% agent) is “very high”. This is also confirmed by the swell pressure and percentage

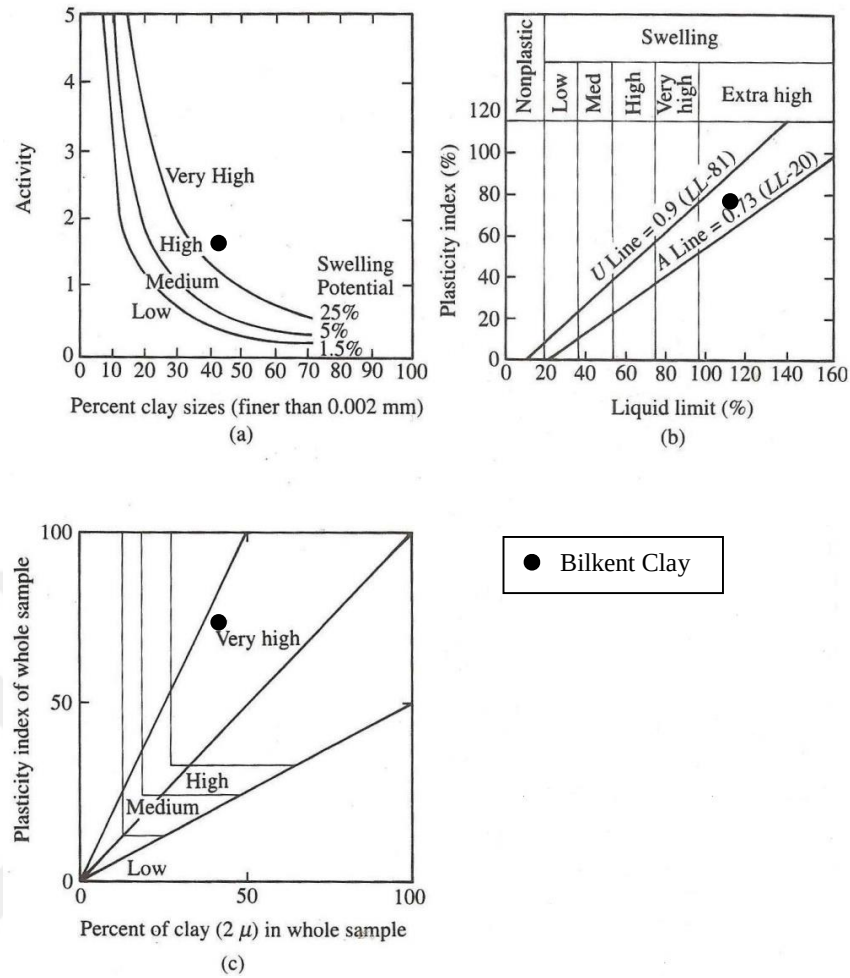
values found via oedometer tests (Table 4.2). It should be noted here that the natural water content is 41%. Since this water content is more than the optimum water content ($w_{opt.}=29\%$), the swell percentage and swell pressure of the undisturbed sample are quite “low” (i.e., swell percentage=4%, swell pressure=50 kPa). However, swell percentage and swell pressure values are “very high” for the remoulded samples prepared at optimum water content and maximum dry density as given in Table 4.2.

Table 4.2 : Properties of Bilkent Clay.

G _s (Specific Gravity)	Void Ratio* e _o	Sieve analysis			Atterberg Limits				
		+No.4	- No.200	Clay Size	LL	PL	PI	Ls	SL
		(%)	(%)	(%)	(%) Liquid Limit	(%) Plastic Limit	(%) Plasticity Index	(%) Linear Shrinkage	(%) Shrinkage Limit
2.737	1.040	0.5	79.2	45	109	33	76	25	9
Unconfined Comp.Test *	Unconfined Comp.Test *				Swell	Swell	Permeability Coeff.*		
		Proctor Test							
q _u	q _u	γ _{dry} max	W _{opt.}	Pressure**	Percentage**	k			
(kPa)	(kPa) (Soaked)	g/cm3	%	(kPa)	(%)	(m/sec)			
385	37	1.369	29	374.6	26.9	5.007 x 10-9			

*Experiments have been performed on undisturbed samples.

**Experiments have been performed on remoulded samples prepared at optimum water content and maximum dry density.



- (a) Seed et al. (1962)
 (b) Dakshanamurthy and Raman (1973)
 (c) Van der Merwe (1964)

Figure 4.2 : Criteria for determining swell potential (Das, 1999).

4.3. Lime Stabilization

The effect of lime stabilization on both clays have been tested in many respects such as index and swelling properties, strength values for both soaked and non-soaked conditions, resilient modulus and methylene blue values and compressibility characteristics. Moreover, time effect by allowing different curing periods has constituted another research subject. The outcomes are given below.

4.3.1. Esenboğa Clay

An extremely highly plastic clay type of soil has been mixed with hydrated lime at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%). Samples

have been prepared at optimum water content and maximum dry density by means of Harvard miniature compaction test (Wilson, 1950, ASTM D698- 07). Grain size distribution, Atterberg limits, strength and swelling properties of the lime treated clay samples have been determined at each lime content which is increased step by step. The effect of lime stabilization on Atterberg limits, strength and swelling characteristics are given in Figure 4.3 to Figure 4.14.

Atterberg limits, unconfined compression tests and odometer tests have been conducted on the soil mixtures. The tests have been performed according to the ASTM Standards (ASTM 427, 1998; ASTM 2166, 2013; ASTM 4318, 2010; ASTM 4546, 2008).

4.3.1.1 Effects of stabilization on index parameters

When the Figure 4.3 is examined, the effective lime content to decrease the liquid limit and plasticity index can be considered as 4 to 5%. But this effectiveness does not guarantee the allowable swell percentages as will be explained below.

Shrinkage limit increases steadily as the lime content increases as given in Figure 4.3. The soil exhibits the highest value of shrinkage limit at 9% lime content. The average increment for shrinkage limit is 9.5% for each lime content increment.

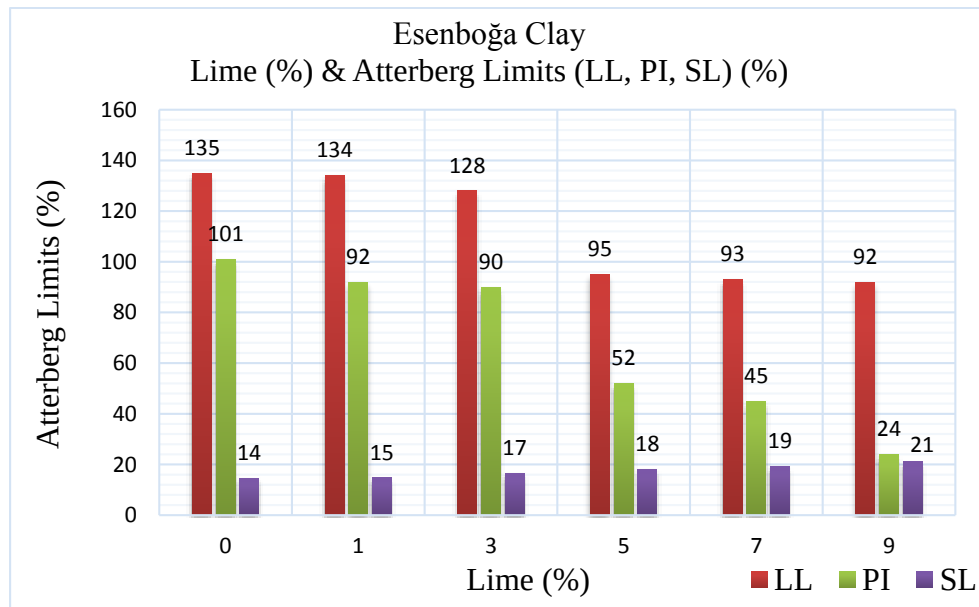


Figure 4.3 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with lime content.

Proctor test results for stabilized samples using lime are given in Table 4.3.

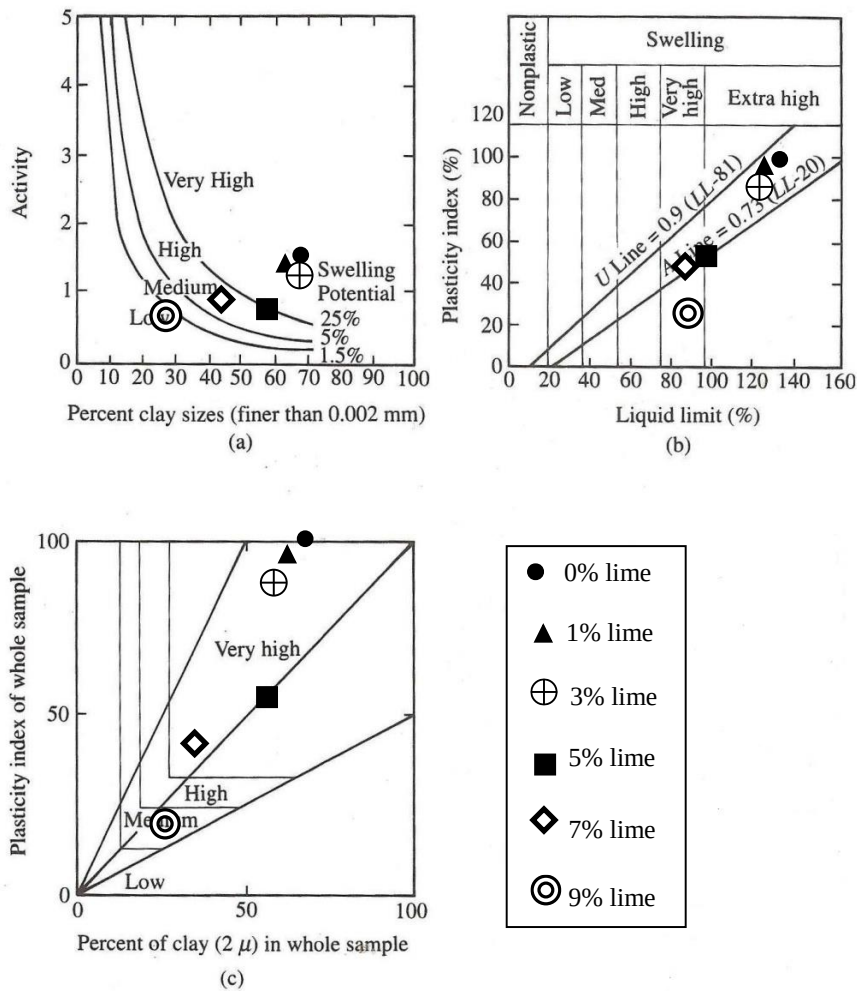
Table 4.3 : Proctor test results for treated samples with lime for Esenboğa Clay.

Lime Content (%)	0 (Natural)	1	3	5	7	9
γ_{drymax} (kN/m ³)	11.60	11.74	12.65	12.40	11.30	11.94
W_{opt} (%)	39.5	40.8	36.8	45.2	45.2	43.0

4.3.1.2 Effects of stabilization on swell parameters

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on lime treated samples are marked on the charts in Figure 4.4, the corresponding swelling potentials can be specified. According to Seed et al. (1962) and Van der Merwe (1964) approaches, the swelling potential of the sample containing 9% lime, decreases to “low-medium”. However, Dakshanamurthy and Raman’s (1973) chart still gives “very high” swell potentials for 9% lime content. The results of odometer swell tests in this study show that Dakshanamurthy and Raman’s (1973) method, which only takes into account liquid limit, is quite conservative. The methods in which the clay fractions are taken into consideration seems more reliable. As a result, Seed et al. (1962) and Van der Merwe (1964) approaches are more compatible with the results of odometer swell tests performed in this study (Figure 4.4). However, some data on Seed at al.'s chart (1962) seems not very compatible; therefore, after a preliminary guess with these charts site-specific oedometer swelling tests are needed for a definite description.

It has been observed that the activity values decrease generally with the increase in lime content; whereas these values increase with the increase in fly ash content of treated samples. This is because the rate of change in plasticity index values are different from that of clay size fraction values for different agents. Çokça (2001), on the other hand, found in a laboratory experiment program, in which an artificial expansive soil (kaolinite+bentonite) was treated with lime and fly ash, that activity decreased with increasing percent stabilizer.



- (a) Seed et al. (1962)
 (b) Dakshanamurthy and Raman (1973)
 (c) Van der Merwe (1964)

Figure 4.4 : Criteria for determining swell potential (the marks in the figures are for Esenboğa Clay) (Das, 1999).

Variation of swell percentage with lime content found in the laboratory experiments is given in the following graph (Figure 4.5). As understood from this figure, there is a sharp decrease in swell percentage at 3% lime content. The swell percentage values, on the other hand, decrease below 1% after 7% lime content.

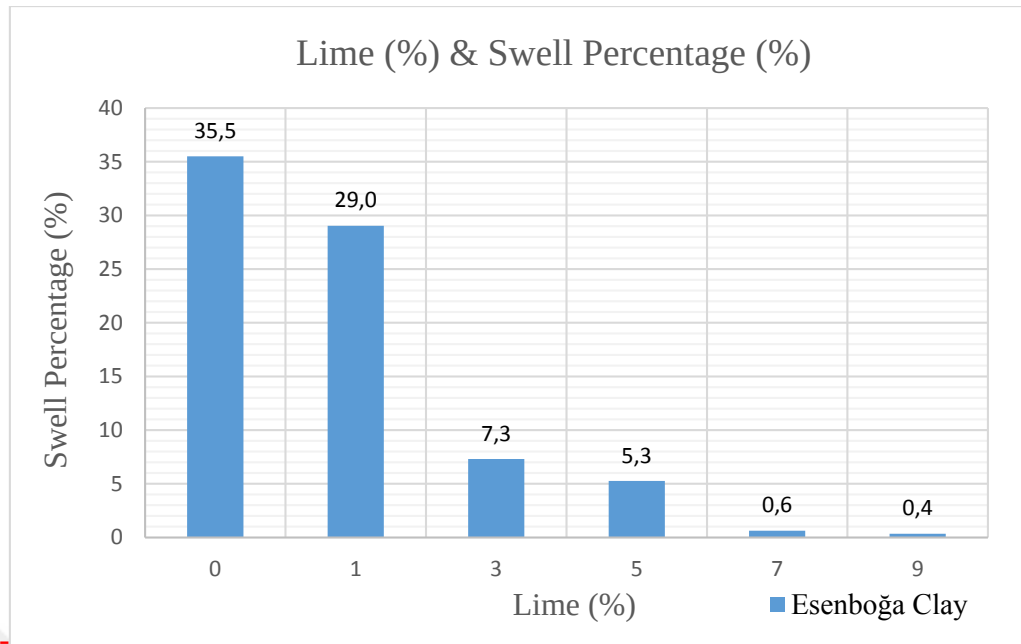


Figure 4.5 : Variation of swell percentage with lime content.

Figure 4.6 below shows the net swell percentages with increasing lime content. This chart has been prepared based on a theoretical calculation in which the net swell percentages are calculated with proportional to the measured natural clay swell percentage (i.e. the natural clay is composed of 100% clay and its swell percentage has been measured as 35.5% in experiment; then with 1% lime, the clay content is 99%, thus the net swell percentage is calculated as 35.2%, the calculations are similar with other lime contents).

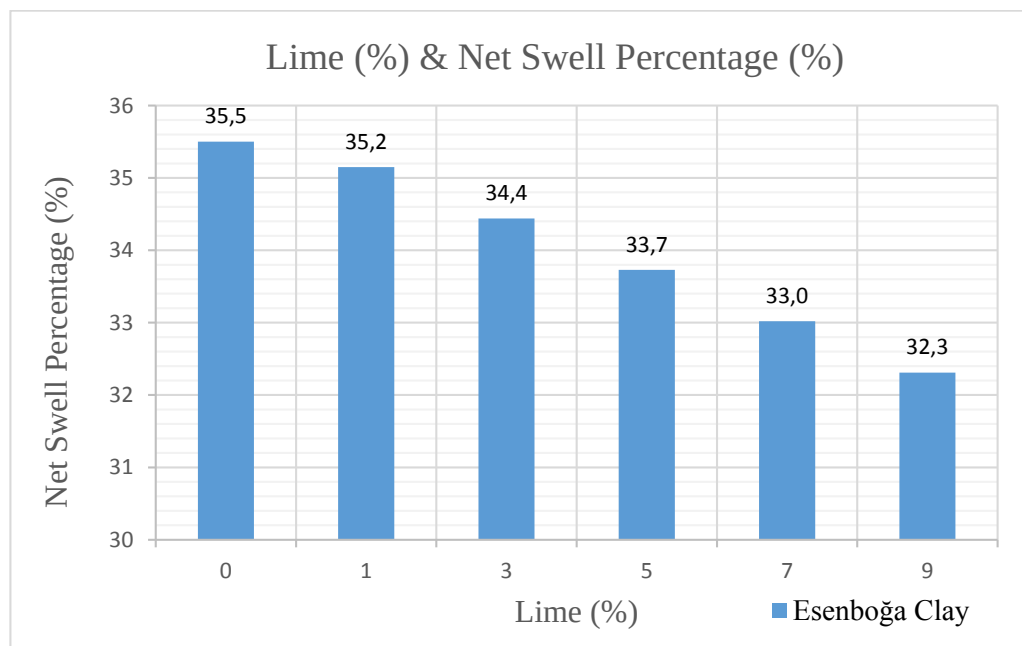


Figure 4.6 : Variation of net swell percentage with lime content.

Figure 4.7 shows the effect of lime on swell percentage as lime content increases. This chart is simply the difference between the charts given in Figure 4.5 and 4.6. It should be noted here that the swell percentage decrease is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between lime and clay. Figure 4.7 shows only the latter effect. Thus, the effect of lime on swell percentage is maximum when the lime content increases from 1 to 3%. The effect is moderate starting from 3% lime content and after 7% lime content there is no effect of lime on swell percentage (Figure 4.7). The optimum lime content in Figure 4.7 is about 3% as far as the swelling is concerned. In a design of sub-base of transportation structures (i.e. highways, railways etc.), this optimum lime content can be taken as a reference. Swell tests should be executed with this reference value. In case of not satisfying the design criteria, this optimum lime content should be increased. For instance, the allowable swell percentage for transportation structures is generally accepted as 0.5% as given in Turkish Highway Standards, KGM, 2005. This criterion is compatible with the Seed et al.'s (1962) categorization in which the degree of expansion is “low” for potential swell 0 to 1.5%. Lime contents larger than 3% optimum value may especially be necessary to satisfy the design criteria in the extremely highly plastic clays such as used in this study. For example, 0.5% swelling is found at about 8% lime content in laboratory experiments for Esenboğa Clay (Figure 4.4).

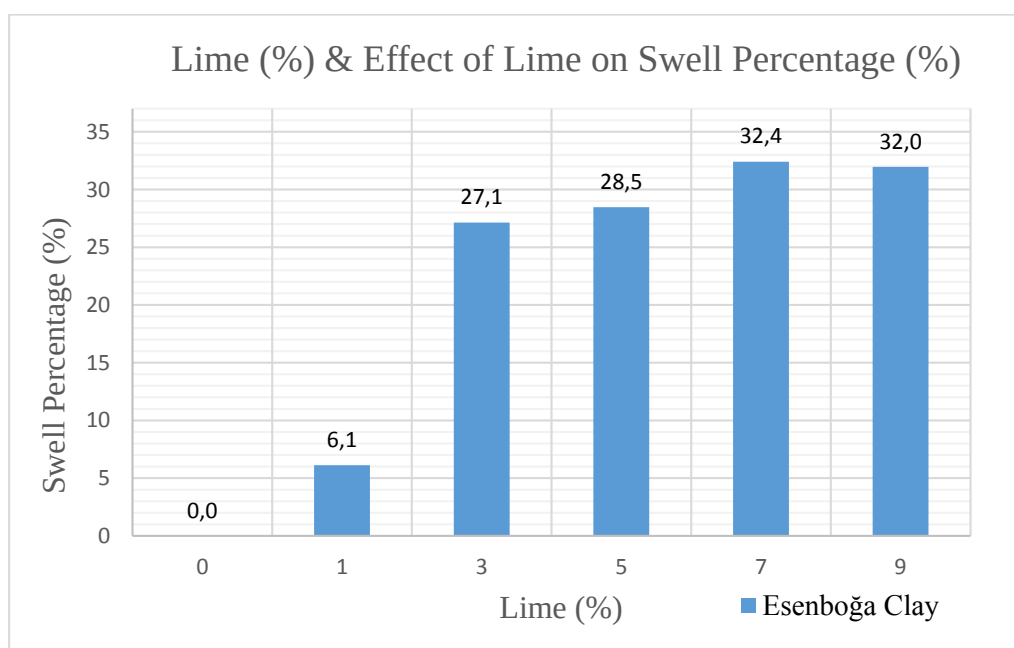


Figure 4.7 : Effect of lime content on swell percentage.

The swell pressures for various lime contents found in the laboratory are given in Figure 4.8. The swell pressure values are quite low (i.e. $q=35$ to 40 kPa approximately) at 7% to 9% lime contents. This result is compatible with the results of swell percentage subject explained above.

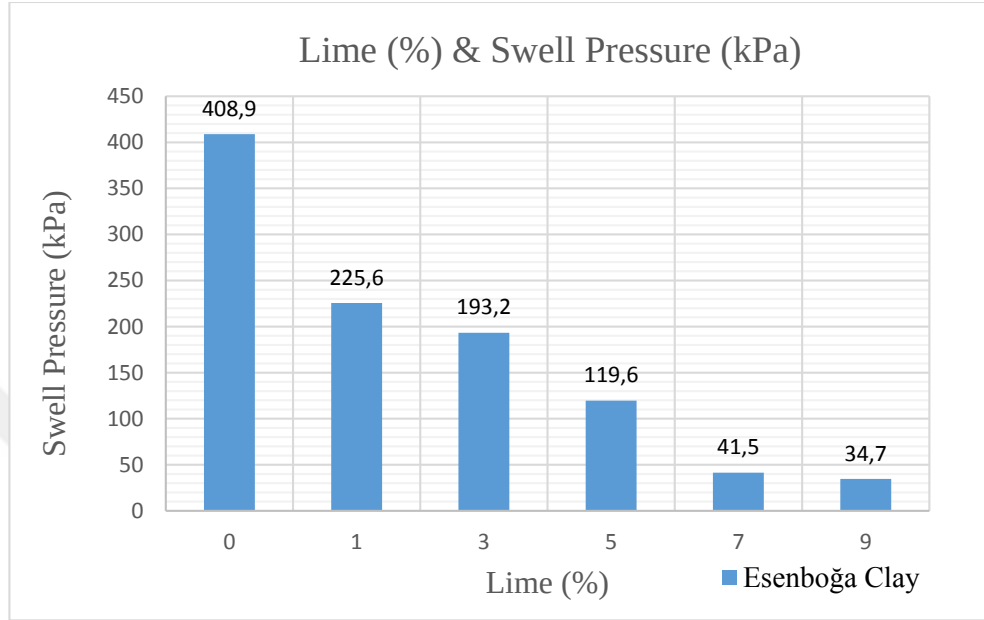


Figure 4.8 : Variation of swell pressure with lime content.

Figure 4.9 and Figure 4.10 show the net swell pressure and the effect of lime on swell pressure with increasing lime content respectively. The explanations are similar to ones given in swell percentage subject above. The effect of lime on swell pressure starts quickly when 1% lime is added as observed in Figure 4.10. This effect then increases non-linearly as lime content increases. However, the effect of lime is almost same for 7% and 9% lime contents.

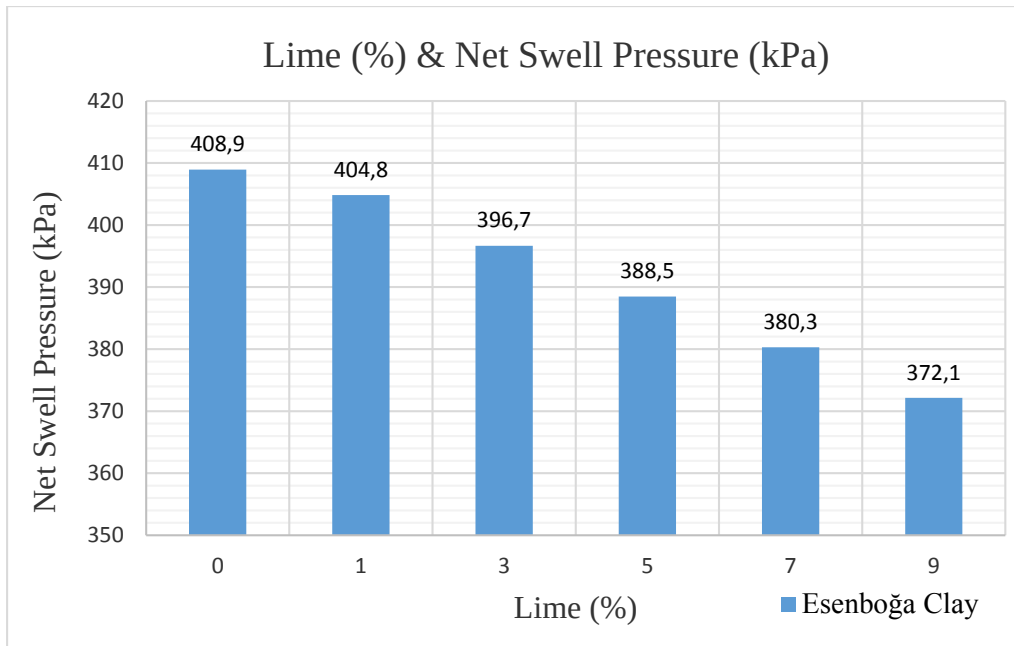


Figure 4.9 : Variation of net swell pressure with lime content.

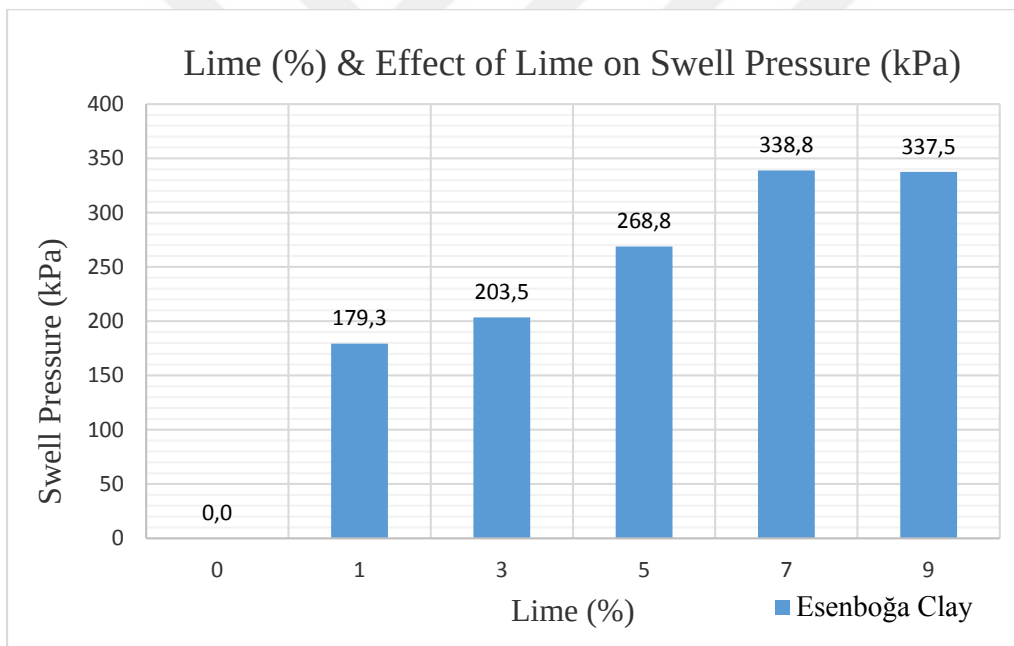


Figure 4.10 : Effect of lime content on swell pressure.

4.3.1.3 Effects of stabilization on strength parameters

Figure 4.11 gives the results of unconfined compressive strengths with increasing lime content for non-soaked and soaked conditions. Unconfined compressive strength of the natural clay in the soaked condition is considerably lower than the one in non-soaked condition as expected. This behavior is also valid for treated samples up to 3% lime content, although the gap between q_u (non-soaked) and q_u (soaked) is significantly decreased at 3% lime content. The behavior is reversed at 5% and 7% lime contents.

At 7% lime content especially, the unconfined compressive strength in soaked condition is about 68% more than that of the non-soaked condition. This reversal of behavior is attributed to considerable pozzolanic reactions between lime and clay starting from 7% lime content in soaked condition with the effect of water. This effective lime percentage is compatible with the findings above.

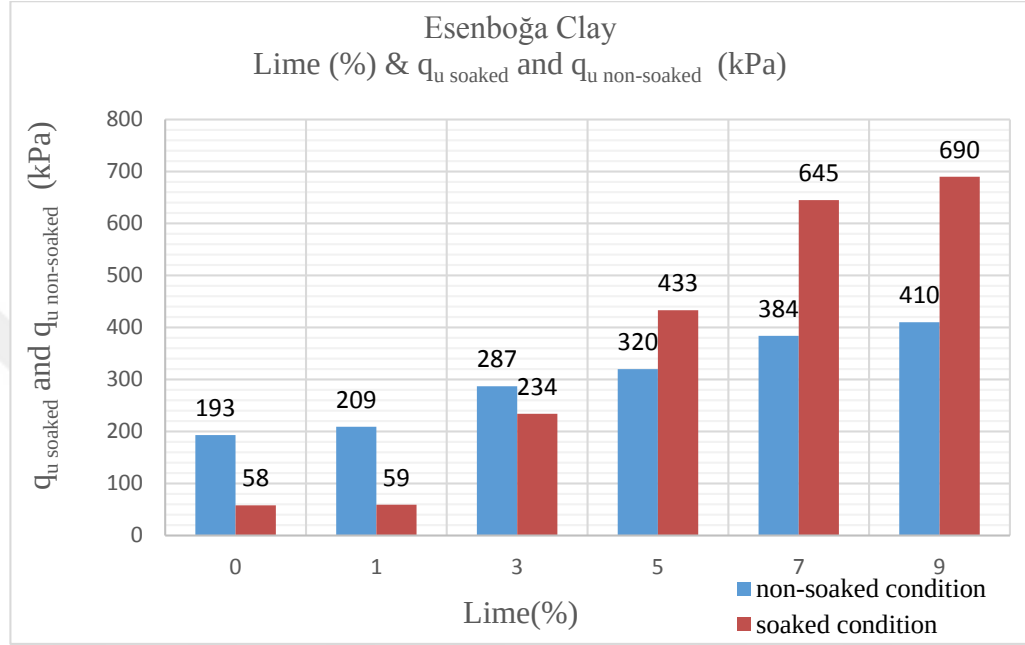


Figure 4.11: Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with lime content.

4.3.1.4 Effects of stabilization on compression parameters

In this part, effect of lime stabilization on compression parameters for Esenboğa clay is explained. Using the results of odometer tests, void ratio (e) versus effective stress (σ') graphs are prepared first. Void ratio (e) versus $\log(\sigma')$ graphs can be drawn then so that the graphs can be approximated to straight lines. Both compression and expansion part of the test can be monitored to get compression and expansion indices from the slopes of the straight lines. C_c is the slope of the virgin compression line of the test. High value of C_c means high compressibility. The expansion index (C_e) is the slope of the unloading part of the test plot. In the same way, the higher the C_e value, the higher the expansion capacity of the clay. The compressibility and expansion potential of highly plastic clays cause problems in foundation engineering applications as stated before. Lime stabilization effect in this context is to be evaluated below for Esenboğa Clay.

$e-\sigma'$ and $e-\log \sigma'$ graphs are given in Figure 4.12 and Figure 4.13 respectively. The variations of compression index (C_c) and expansion index (C_e) values with lime content are given in Figure 4.14 and Figure 4.15 respectively. It is noticed that the clay sample is quite compressible in natural state. Both compression and expansion (recompression) curves are steep in natural state of the soil.

The preconsolidation pressure of the natural Esenboğa Clay can be determined as $\sigma_c=1.7\sim1.8 \text{ kg/cm}^2$ from Figure 4.13. Since the depth of sampling is about 3m for Esenboğa Clay, the over-consolidation ratio (OCR) for it can be calculated as about OCR=3.

It is seen that 1% lime content cannot change compression and expansion indices. Starting from 5% lime content the compression index is markedly reduced. The effective lime content to reduce the expansion index, on the other hand, is 3% (Figure 4.14 and Figure 4.15). It is believed that the flocculation process causes different void ratio values in lime treated samples. The standard of sample preparation process may also result in different void ratio values. Thus, the optimum lime content as far as the compression properties of the Esenboğa Clay are concerned, can be accepted as 3% to 5%.

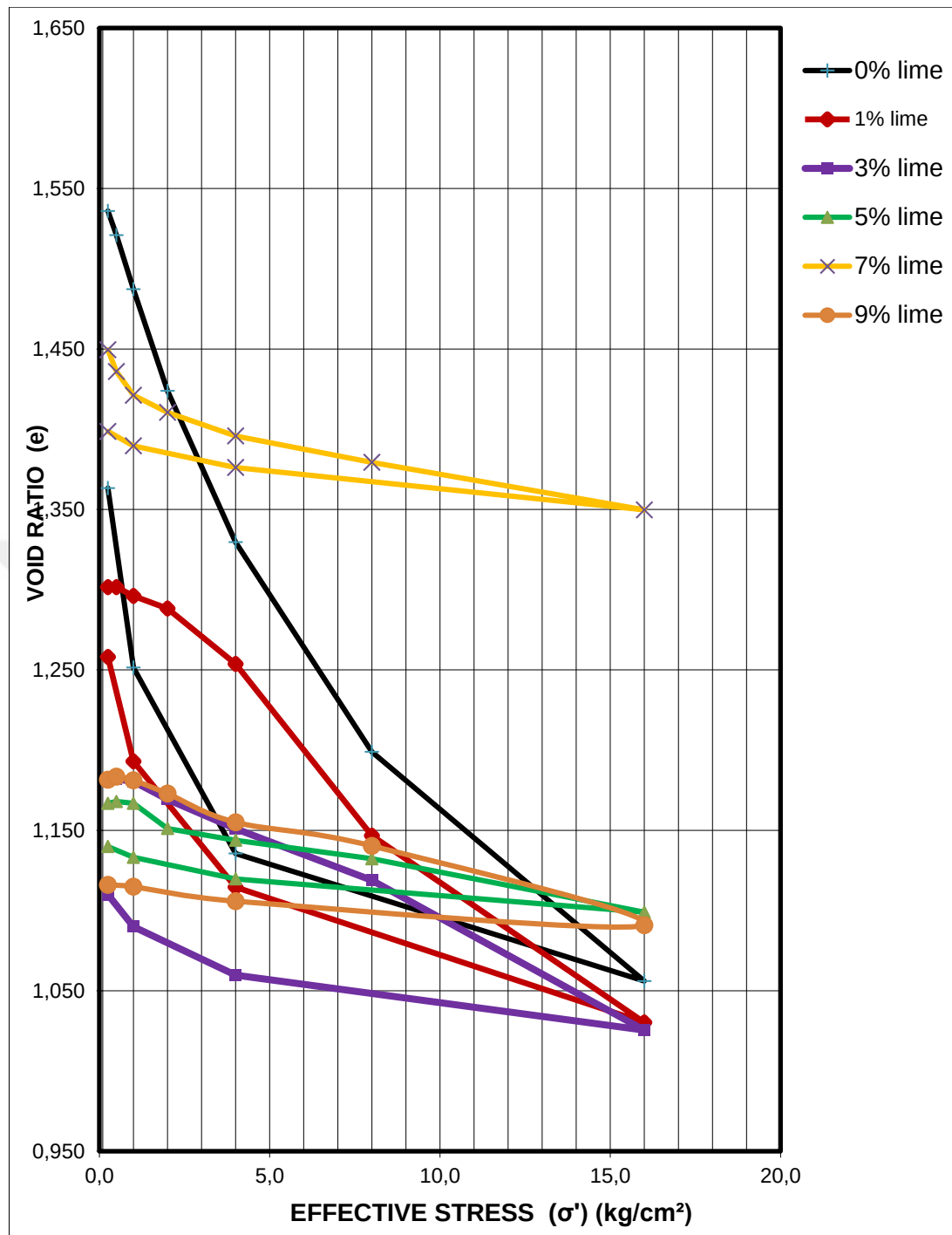


Figure 4.12 : e - σ' relationship for natural and lime-treated soil samples for Esenboğa Clay.

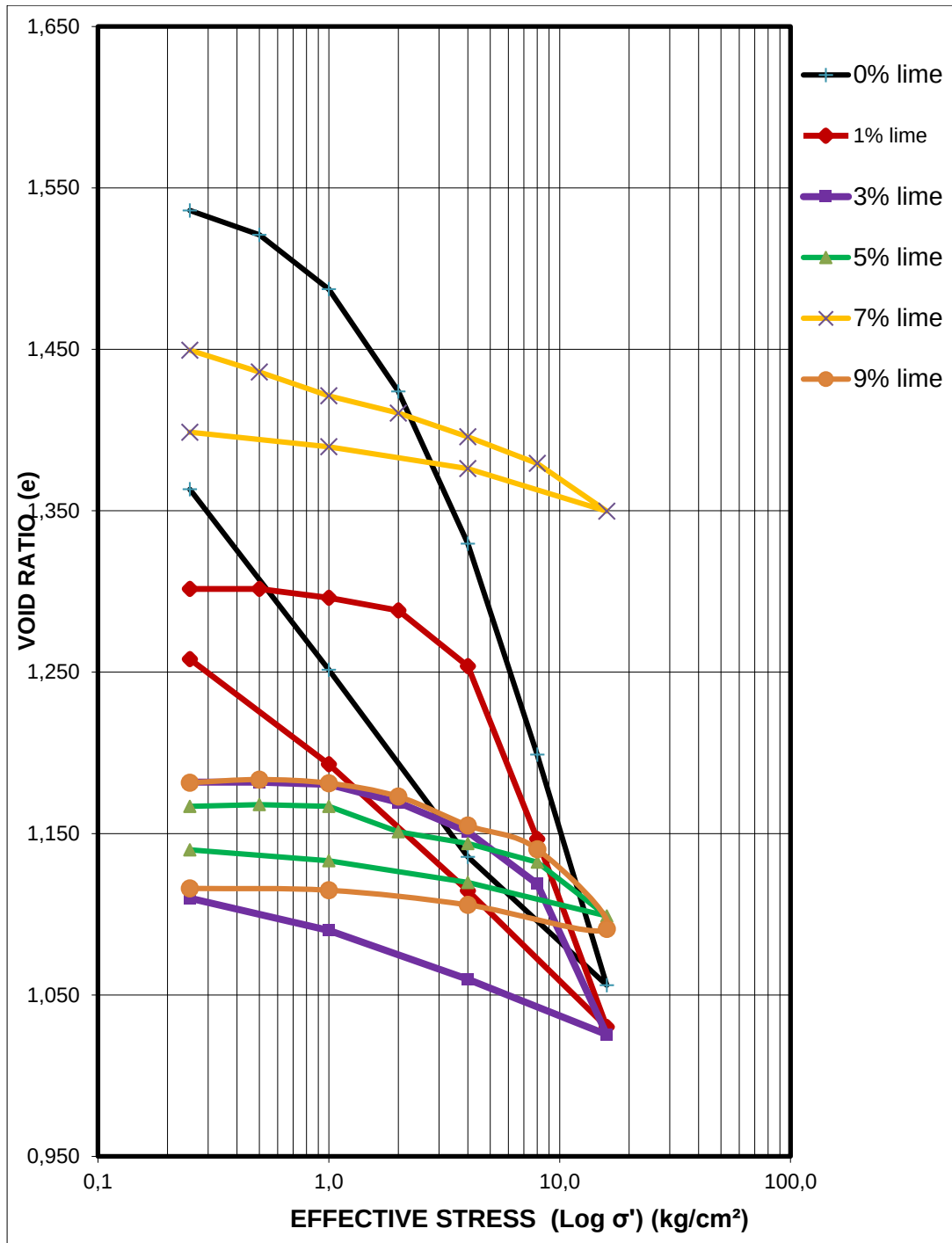


Figure 4.13 : e - $\log \sigma'$ relationship for natural and lime-treated soil samples for Esenboğa Clay.

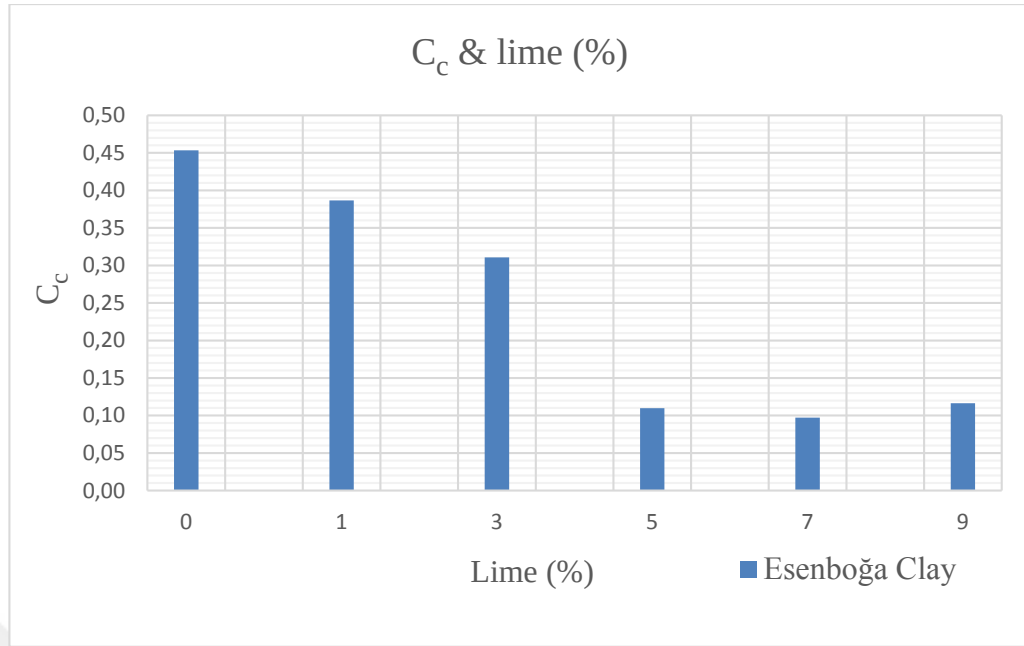


Figure 4.14 : Compression index (C_c) values with increasing lime content for Esenboğa Clay.

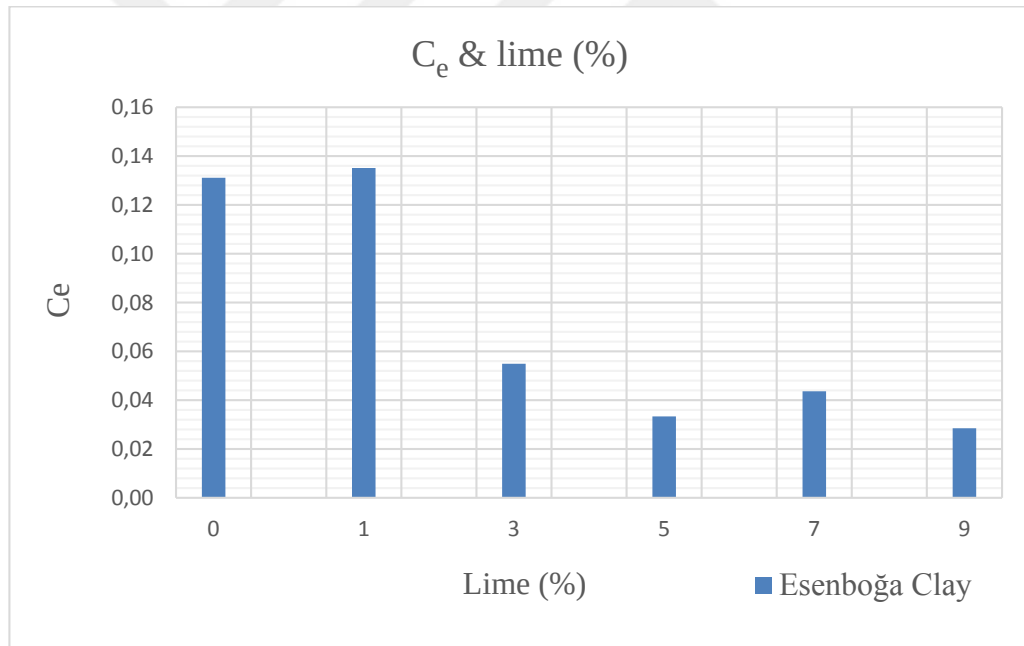


Figure 4.15 : Expansion index (C_e) values with increasing lime content for Esenboğa Clay.

4.3.1.5 Effects of curing time on lime-treated Esenboğa Clay

In this study, soil samples that are extremely highly plastic ($PI=101\%$) have been taken from clay deposits from the Akyurt district of Ankara, near Esenboğa Airport. Both disturbed and undisturbed soil samples have been taken from the site in order to perform laboratory tests and hydrated lime has been used as a chemical stabilizer.

Firstly, reference tests have been carried out on natural soil samples. Then the clay is mixed with the agent at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%) and also soil samples for each percentages prepared for 7, 28 and 56 days curing time.

Effects of stabilization on swell parameters

Figure 4.16 gives the swell percentages for the condition without cure. The allowable swell percentage for surficial structures is generally accepted as 0.5% as given in Highway Standards (KGM, 2005). Therefore, the lime content to obtain allowable swell percentage is about 8% without cure. It should be remembered that the optimum lime content for Atterberg Limits was about 4 to 5%. Thus swelling criterion controls any design in this manner.

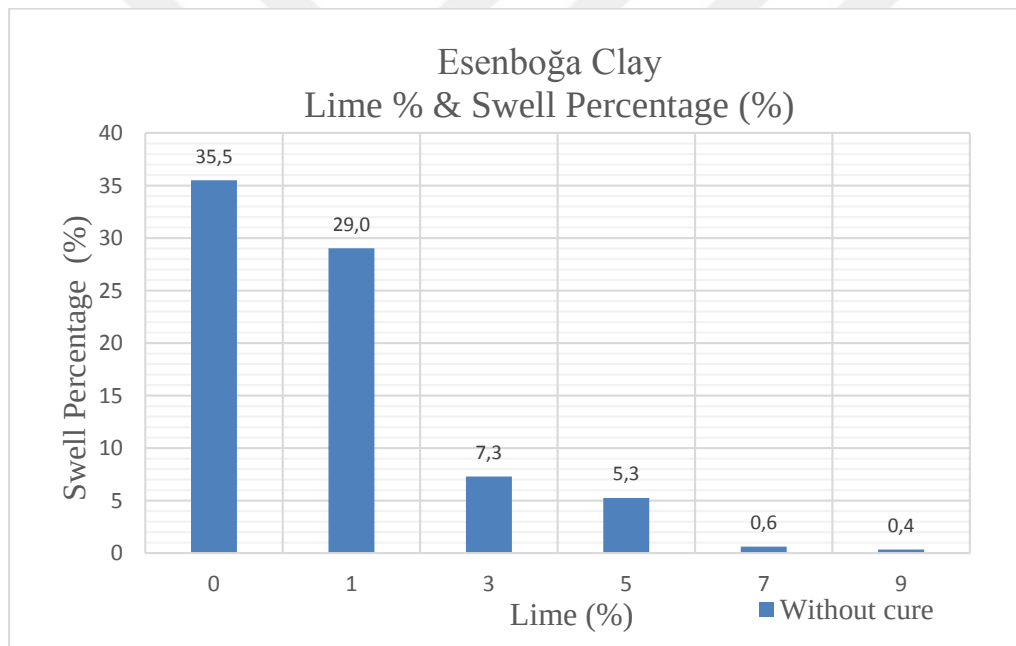


Figure 4.16 : Variation of swell percentage with lime content without cure.

Figure 4.17, 4.18 and 4.19 below show the swell percentages for curing periods of 7 days, 28 days and 56 days respectively. It is seen that about 7% lime content is enough to catch the allowable swell percentage which is 0.5% in 7 days (Figure 4.17). When the curing period is increased to 28 days and 56 days, 5% lime content is the optimum content at which the swell percentage is decreased quite dramatically. It is interesting to note that 28 days curing period, which resembles the optimum time for strength gain in concrete, is quite adequate to get allowable swell percentages at about 5% lime content (Figure 4.18). Then, it can be concluded that 56 days curing period may not be

necessary as far as the swell percentage criterion is concerned (Figure 4.19). Time effects on swell percentages with increasing lime content are all given in Figure (4.20).

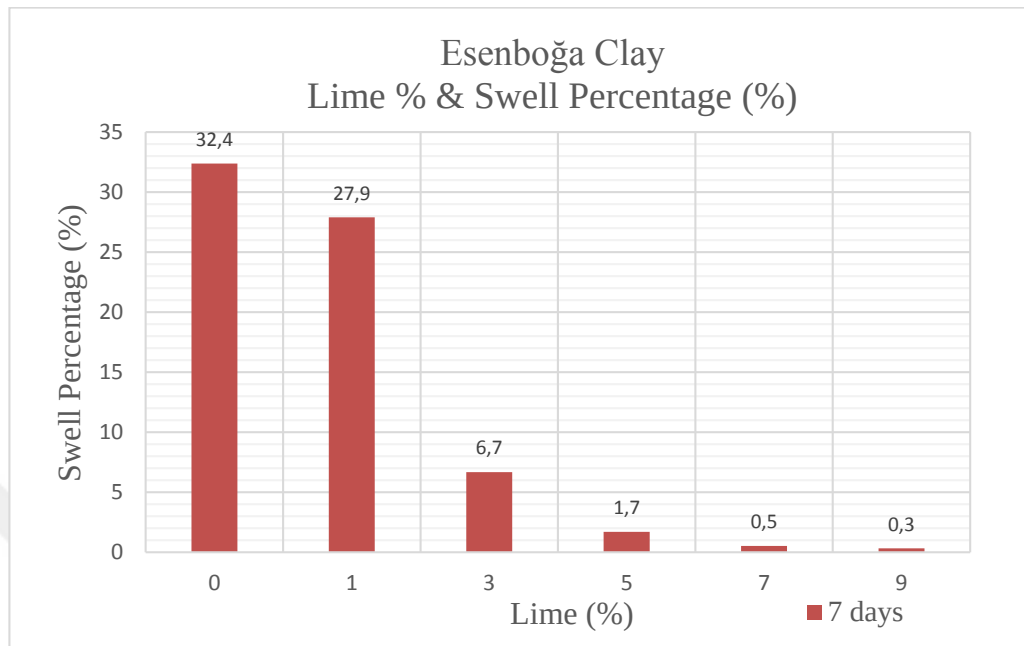


Figure 4.17 : Variation of swell percentage with lime content for 7 days cure period.

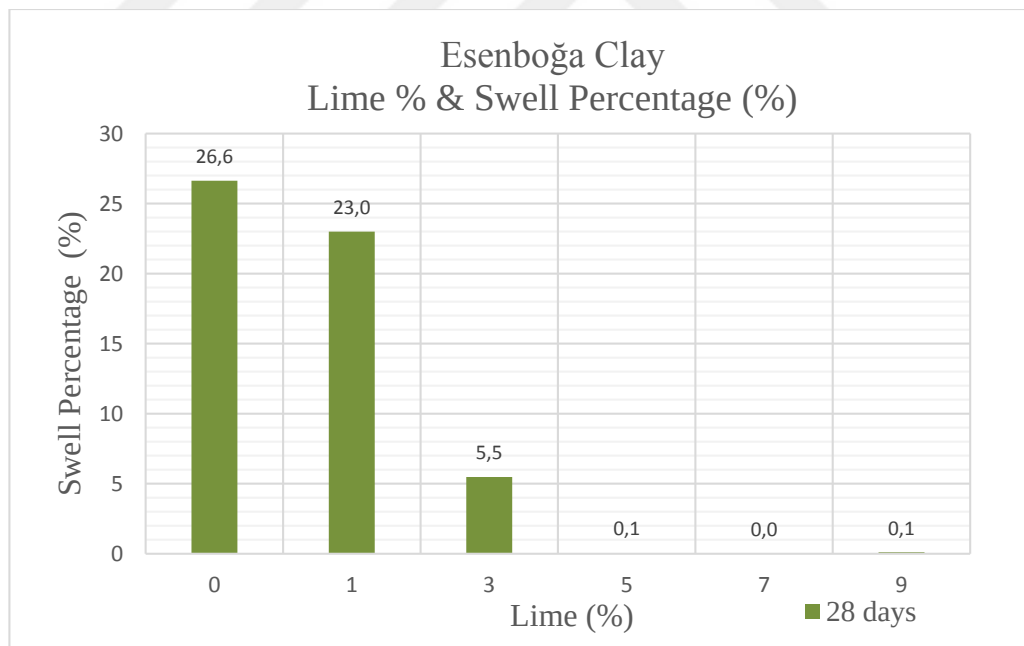


Figure 4.18 : Variation of swell percentage with lime content for 28 days cure period.

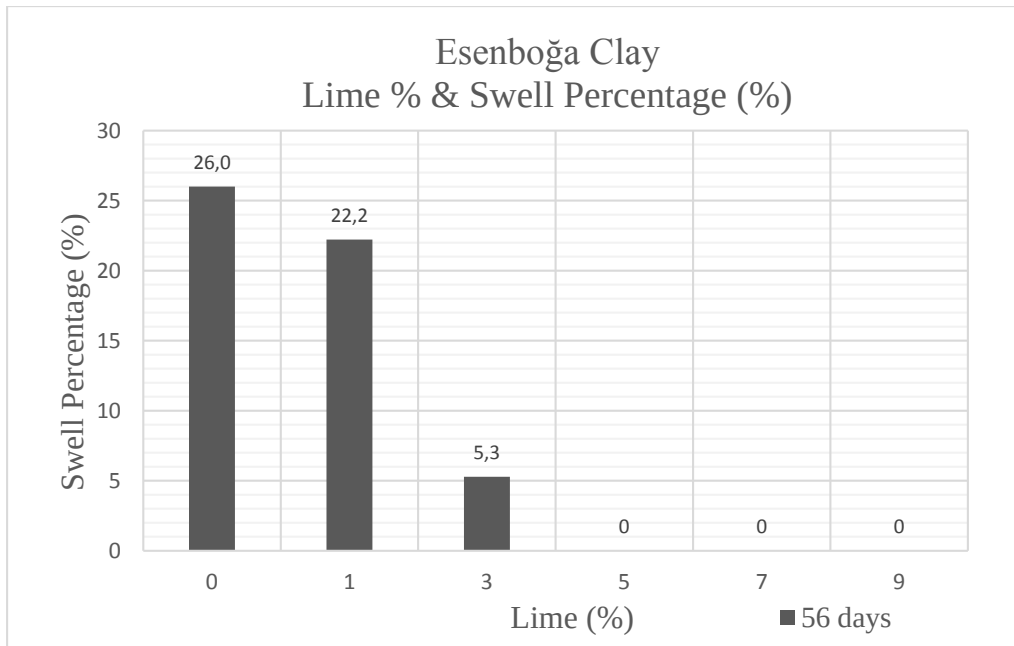


Figure 4.19 : Variation of swell percentage with lime content for 56 days cure period.

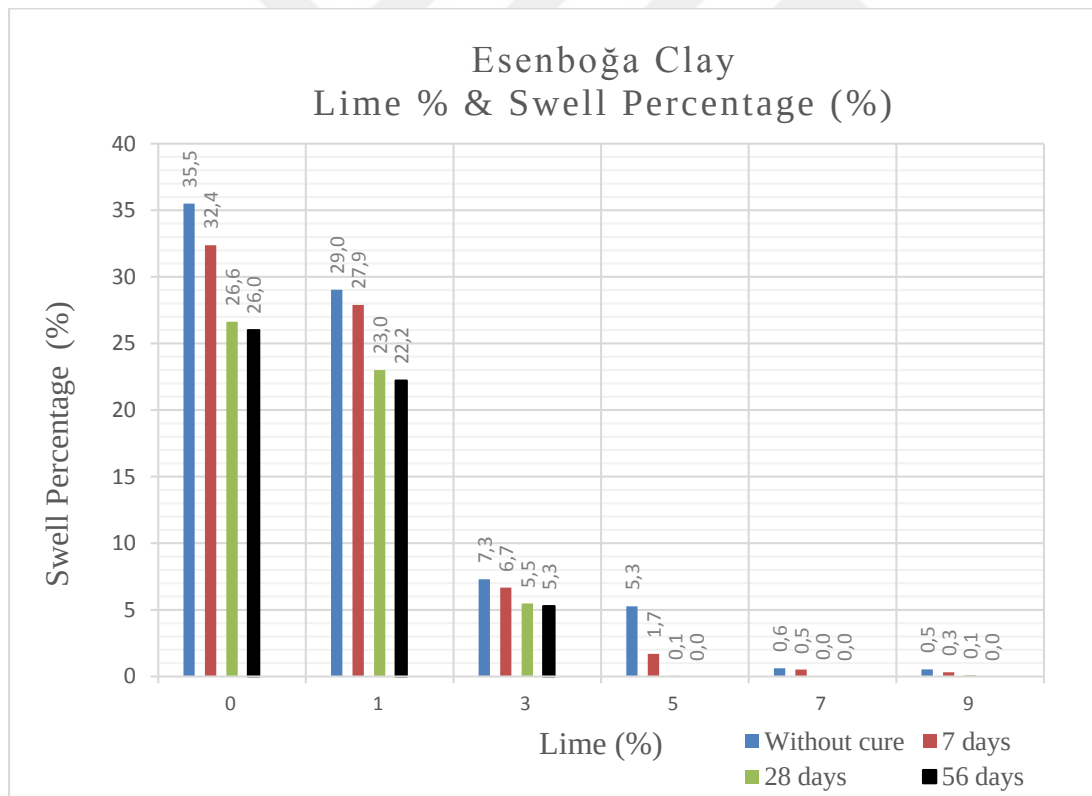


Figure 4.20 : Variation of swell percentage with lime content for 7, 28 and 56 days cure periods.

Variation of swell pressure with increasing lime content without cure is given Figure 4.21 below. It is noticed that the swell pressures are still not negligible when the lime content reaches 7 to 9%.

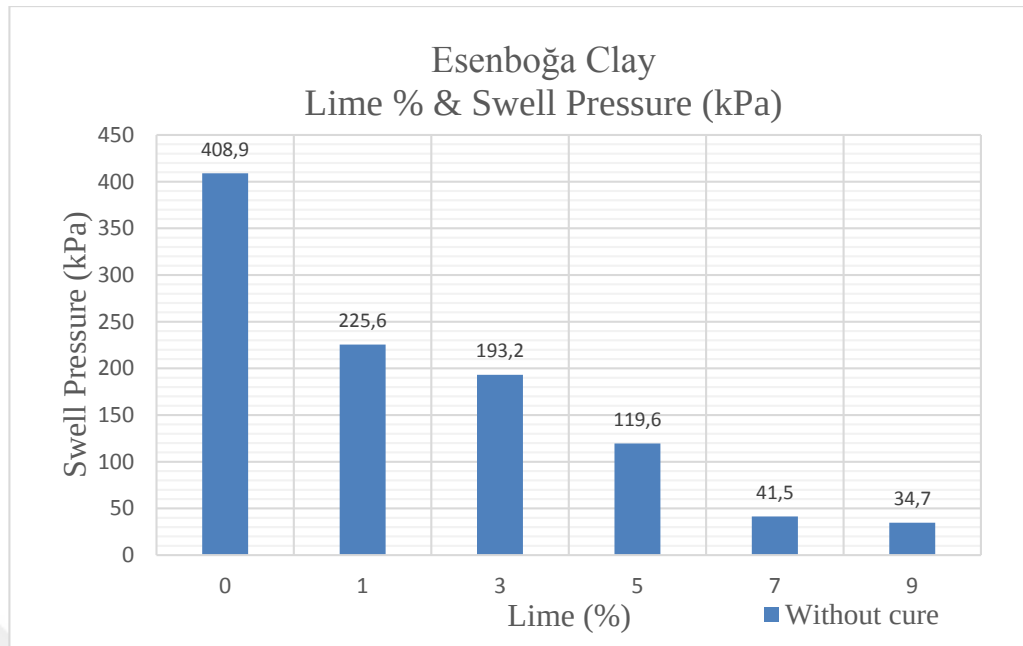


Figure 4.21 : Variation of swell pressure with lime content without cure.

Swell pressures are given in Figure 4.22, 4.23 and 4.24 for curing periods of 7 days, 28 days and 56 days respectively. It is observed in all figures that 5% lime content is sufficient to decrease the swell pressures considerably. This result is compatible with the swell percentage results above. It can be concluded again that 28 days curing period is quite effective to decrease the swell pressure at about 5% lime content. Time effect is clearly seen in Figure 4.25 in which all the results are given. It should be noted that the swell pressures in 28 and 56 days are smaller than the pressures that the 1 m of base and subbase soil material will exert in pavement design. This indicates a satisfactory soil stabilization at 5% lime content in most designs.

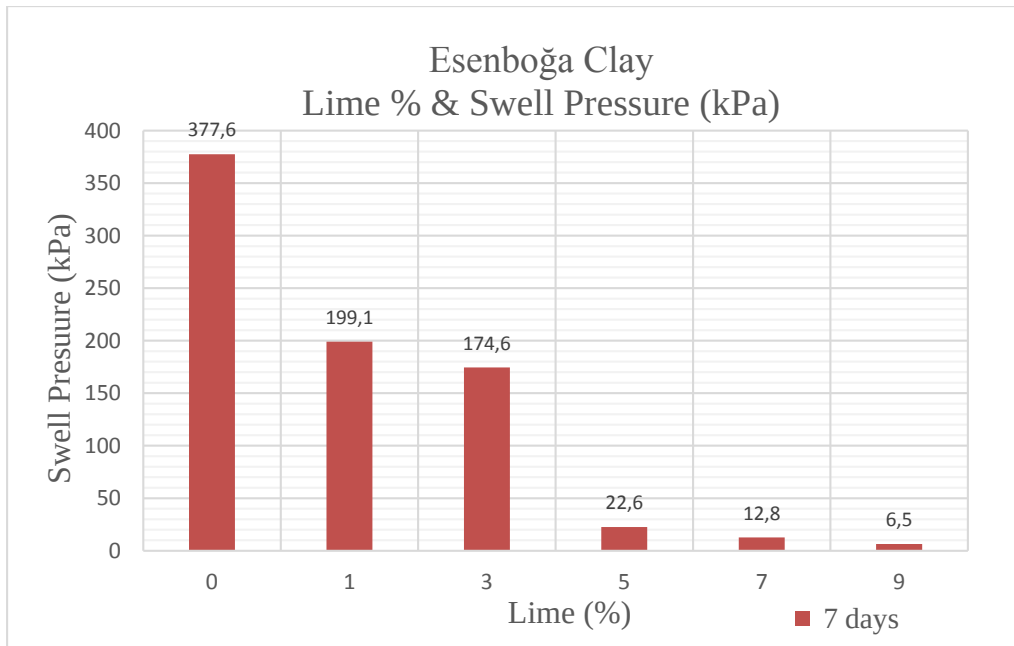


Figure 4.22: Variation of swell percentage with lime content for 7days cure period.

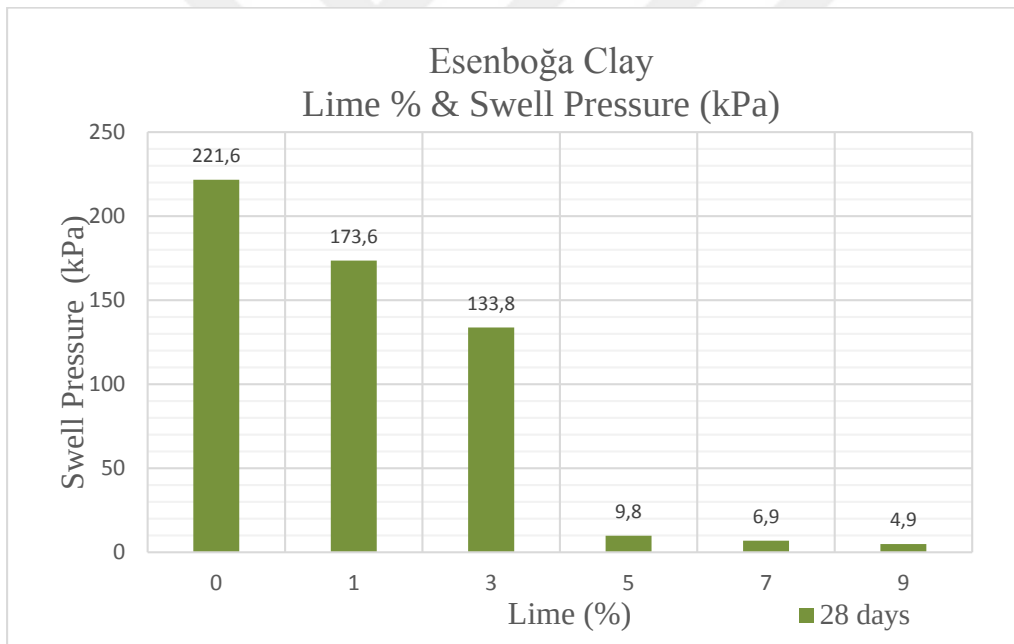


Figure 4.23 : Variation of swell percentage with lime content for 28 days cure period.

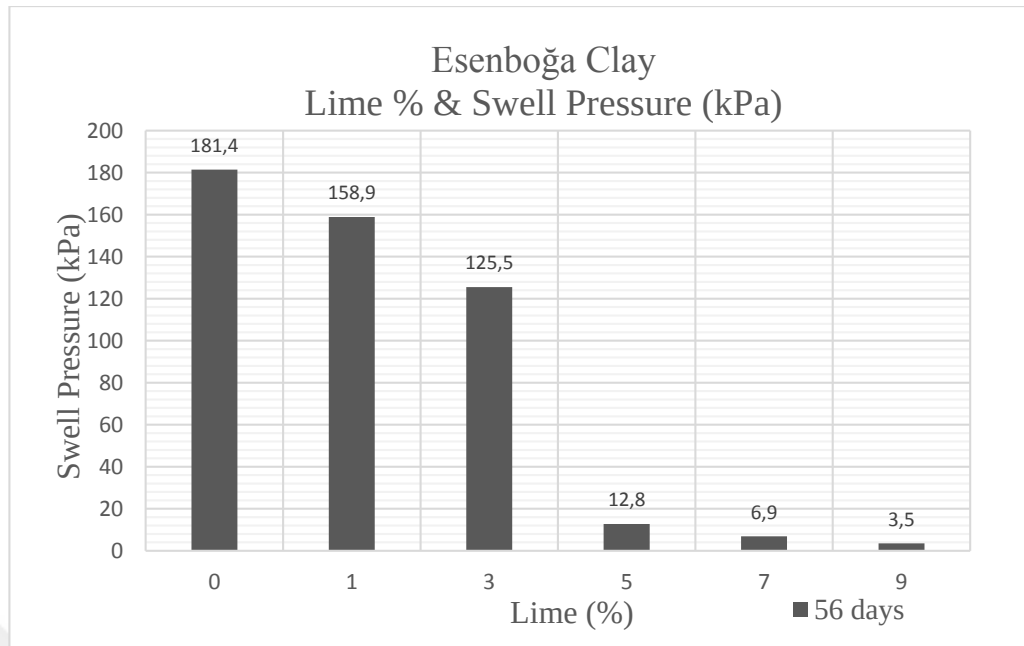


Figure 4.24 : Variation of swell percentage with lime content for 56 days cure period.

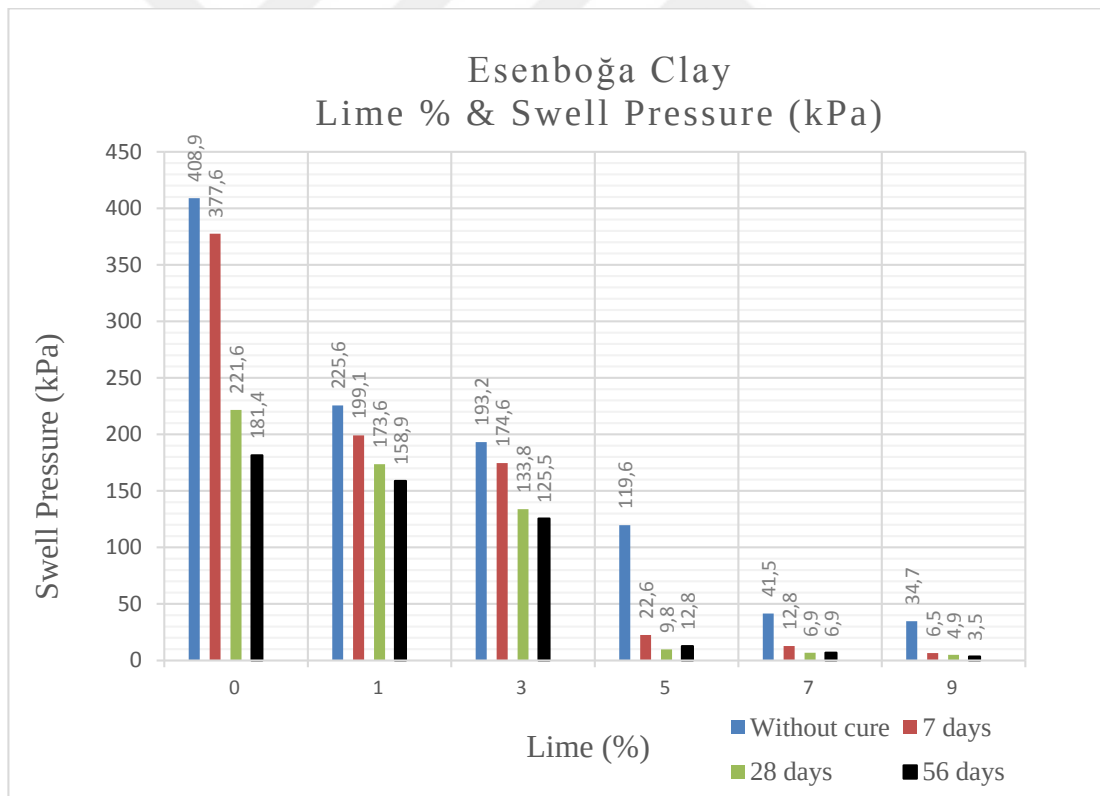


Figure 4.25 : Variation of swell pressure with lime content for 7, 28 and 56 days cure periods.

Effects of stabilization on strength parameters

Figure 4.26 below shows the variation of unconfined compressive strength depending on the lime content for the non-cured and non-soaked conditions. It can be accepted that there is a linear increase in unconfined compressive strength starting from 1% lime content.

The unconfined compressive strengths are given for different curing periods in Figure 4.27, Figure 4.28 and Figure 4.29 for non-soaked condition. The strength is increasing for 7-day curing period as lime content increases but in an unstable manner. The strength increase is moderate up to 3% lime content for 28-day curing period (about 12% increase for each lime content increment as an average). The strength gain is quite remarkable when the lime content increases from 3% to 5% (i.e. about 68% increase). The strength gain rates and strength magnitudes are not very different for 56-day curing period.

It is seen that the unconfined compressive strengths increase dramatically as compared to non-cured strengths for all cure periods. It can be stated finally that 28-day curing period can again be accepted as optimum as far as the unconfined compressive strength is concerned for non-soaked condition similar to the results of swell percentage and swell pressure.

Figure 4.30 shows the all results together supporting the conclusions mentioned above. It has been observed in this figure that time is very effective for the non-soaked compressive strengths. The magnitudes of the strength values have been doubled as an average even in 7 days cure period as compared to non-cured values.

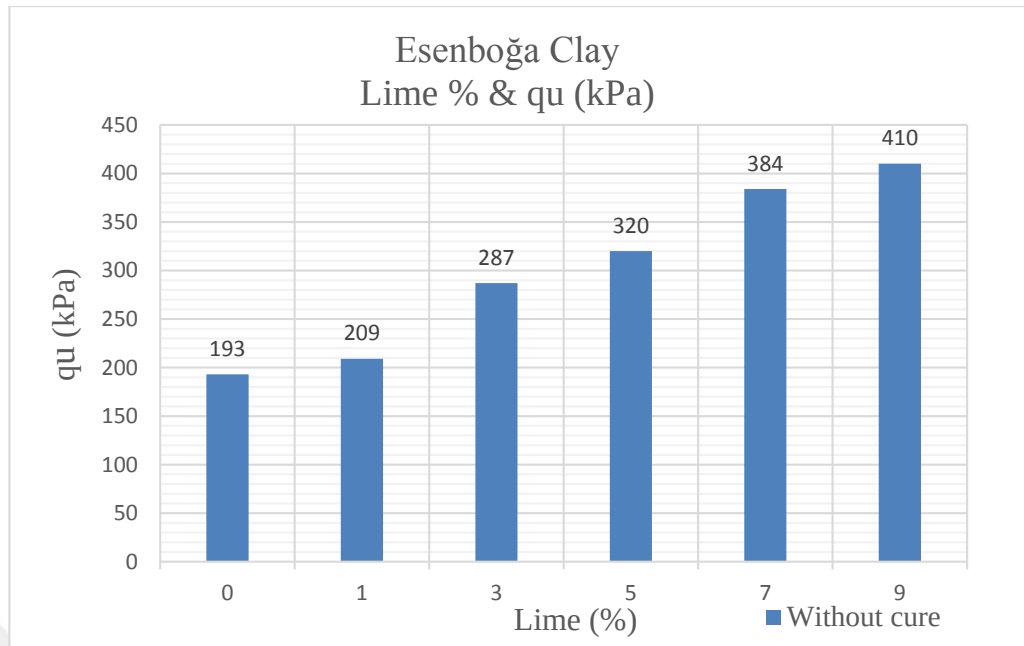


Figure 4.26 : Variation of unconfined compressive strength (q_u non-soaked) with lime content without cure.

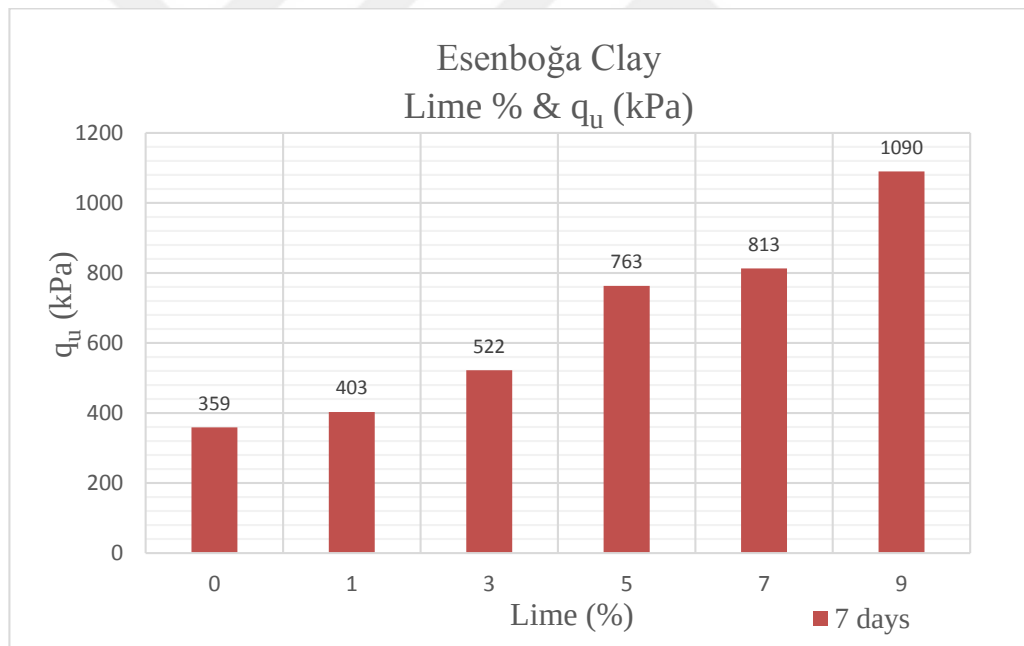


Figure 4.27 : Variation of unconfined compressive strength (q_u non-soaked) with lime content for 7 days cure period.

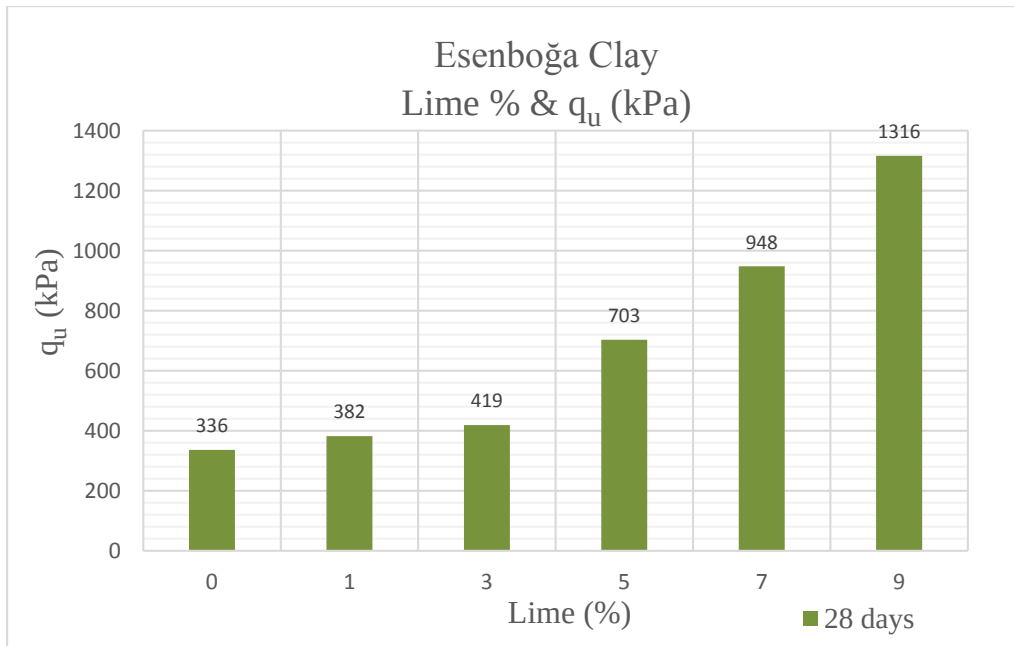


Figure 4.28 : Variation of unconfined compressive strength (q_u non-soaked) with lime content for 28 days cure period.

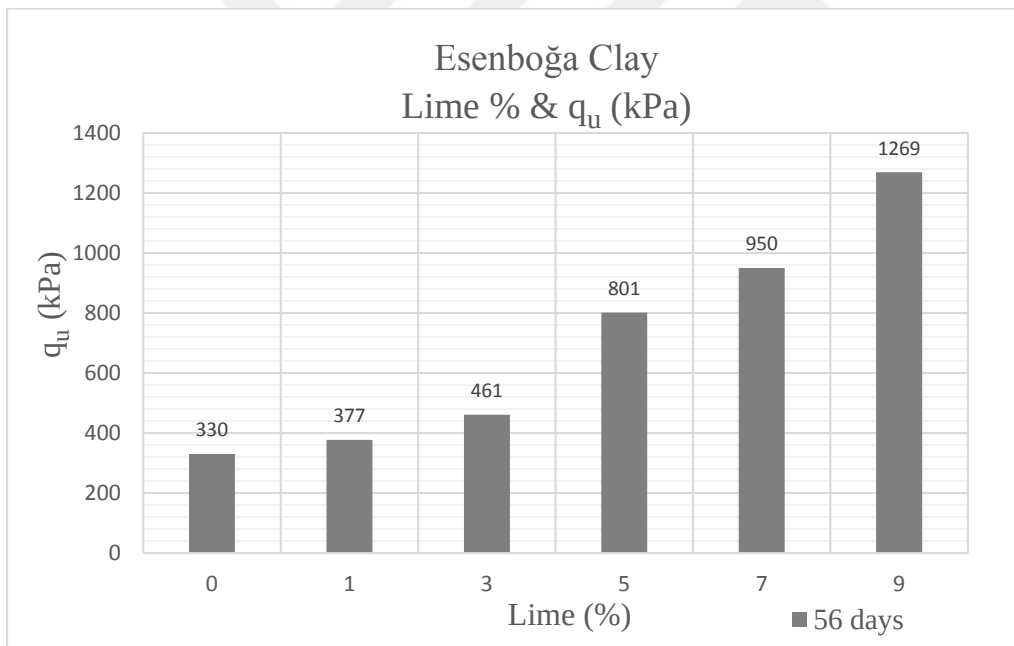


Figure 4.29 : Variation of unconfined compressive strength (q_u non-soaked) with lime content for 56 days cure period.

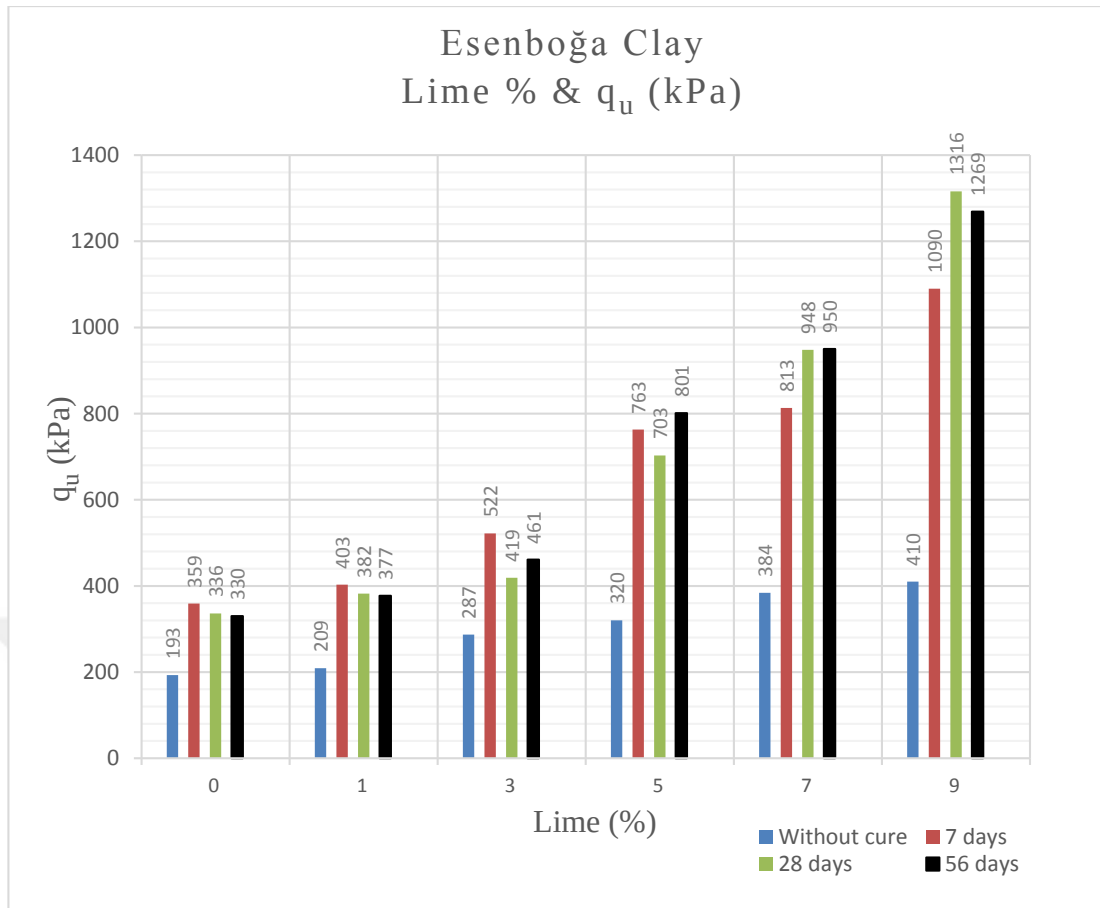


Figure 4.30 : Variation of unconfined compressive strength (q_u non-soaked) with lime content for 7, 28 and 56 days cure periods.

Figure 4.31 below shows the variation of unconfined compressive strength depending on the lime content for the non-cured samples but for soaked condition in this case. It is seen that 1% lime content is ineffective in strength increase. There are significant increases in unconfined compressive strength in each lime content increase starting from 1% up to 7% lime content. The results of 9% lime content are similar to 7% lime content.

The unconfined compressive strengths are given for different curing periods in Figure 4.32, Figure 4.33 and Figure 4.34 for soaked condition. The strength increase behavior is similar to non-cured condition above. 1% lime content seems to be ineffective for 7, 28 and 56 days cure periods. Starting from 5% lime content the strength increases for each lime content increase are quite remarkable. It is observed that the unconfined compressive strengths of soaked samples are always lower than the ones in non-soaked condition in all lime contents for curing periods of 7, 28 and 56 days. This result was not valid for non-cured condition.

All the results for soaked condition are given together in Figure 4.35.

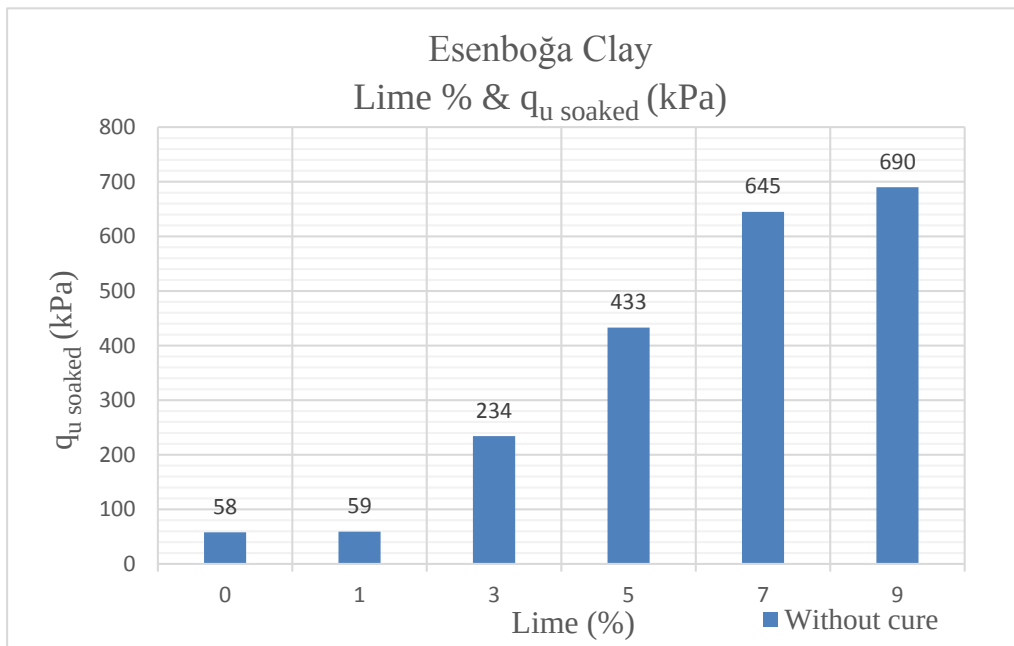


Figure 4.31 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content without cure.

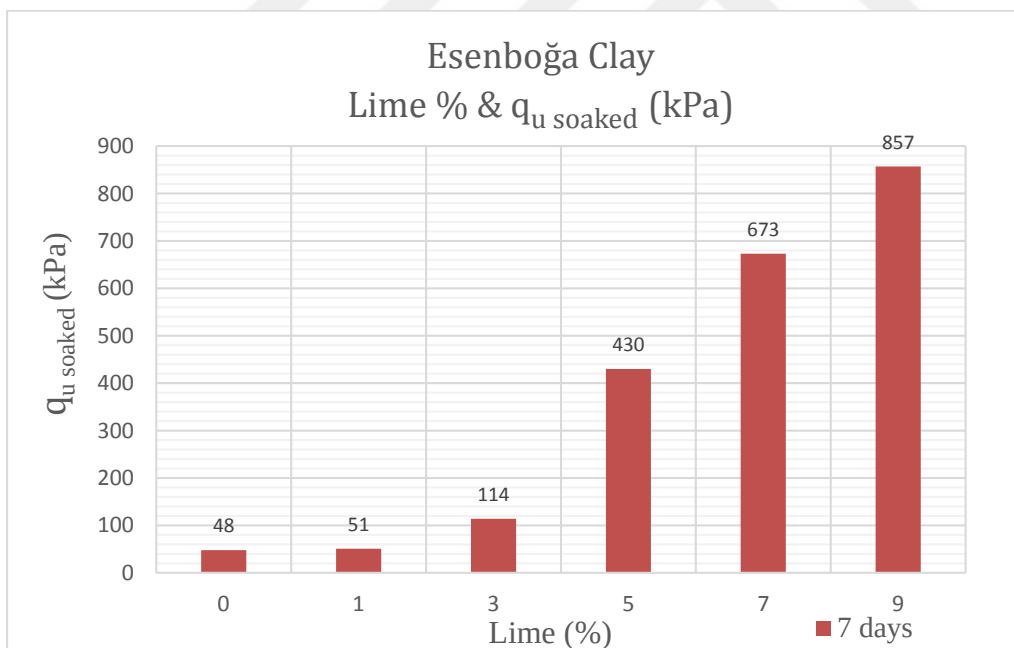


Figure 4.32 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 7 days cure period.

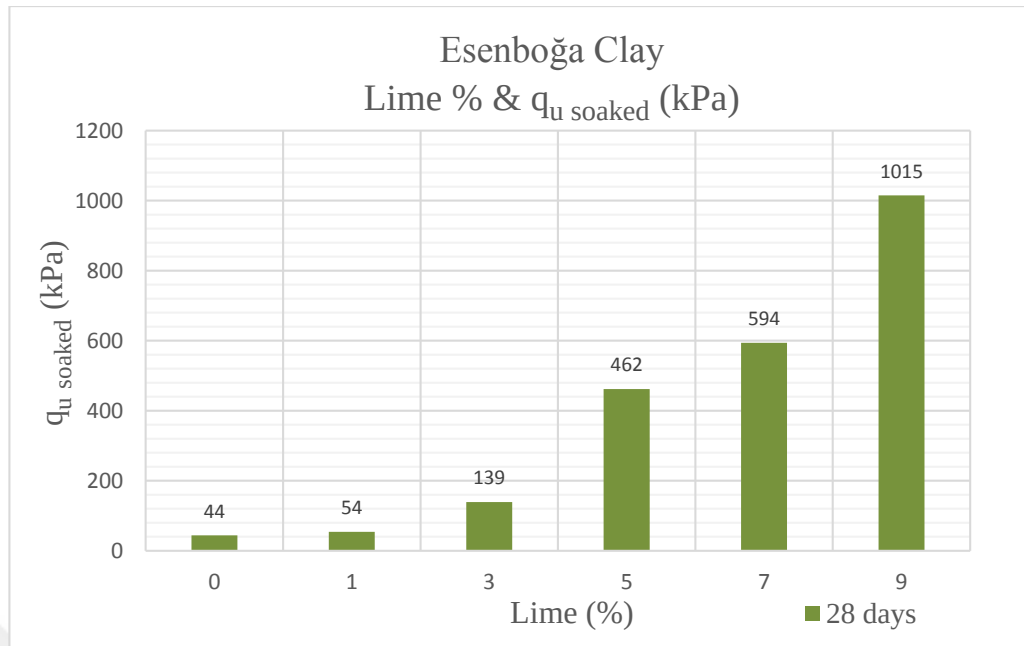


Figure 4.33 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 28 days cure period.

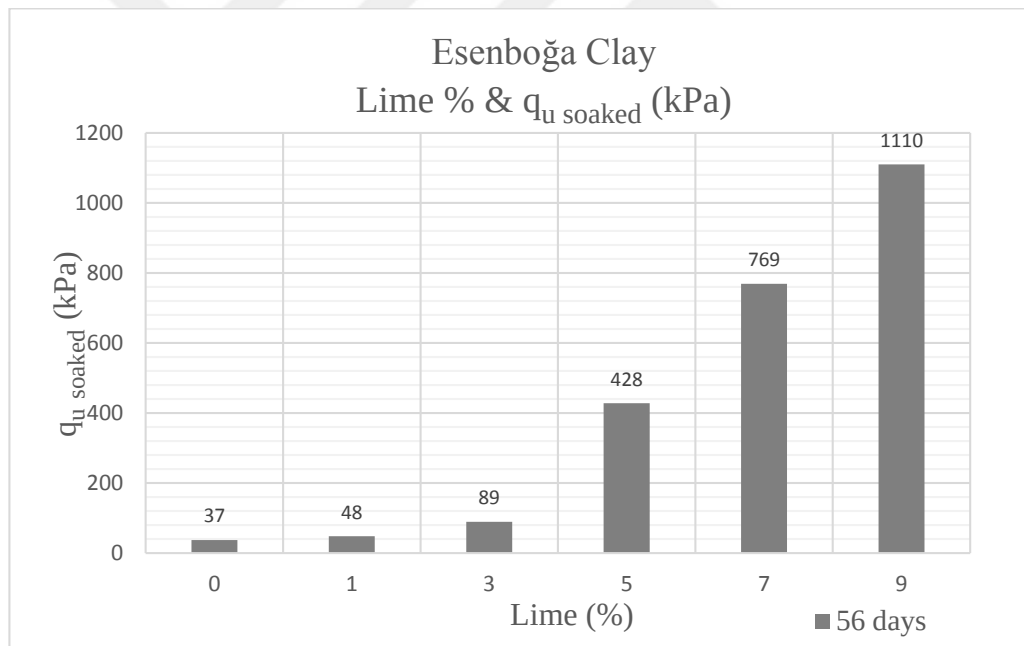


Figure 4.34 : Variation of unconfined compressive strength of soaked samples ($q_{u \text{ soaked}}$) with lime content for 56 days cure period.

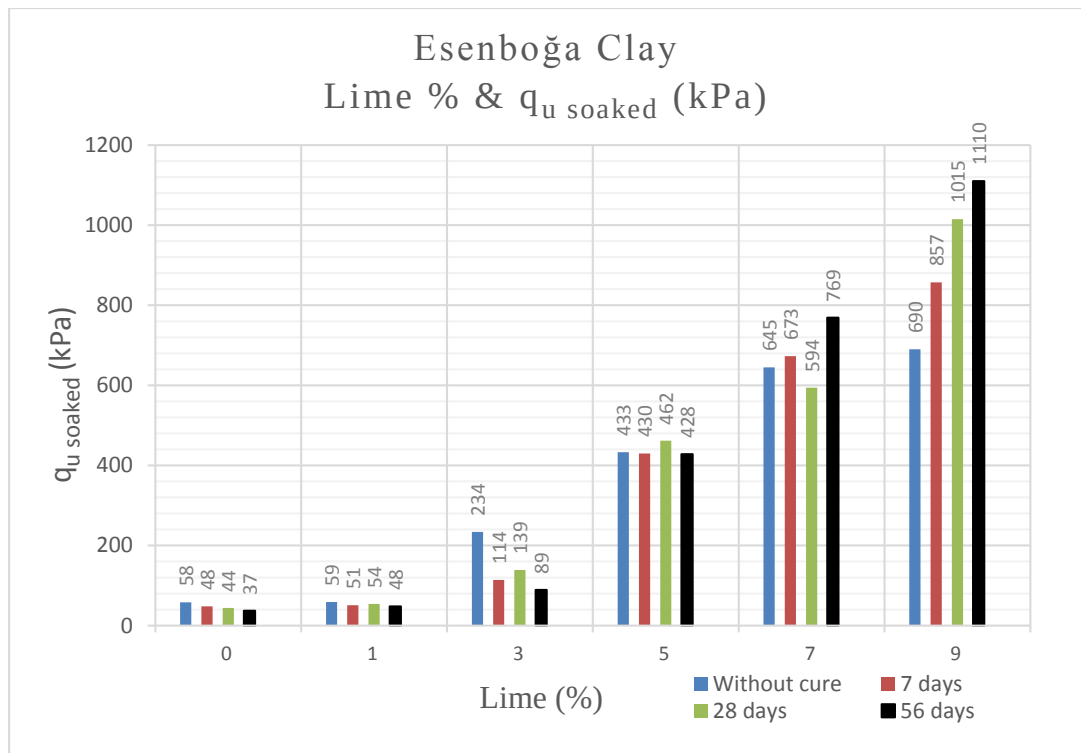


Figure 4.35 : Variation of unconfined compressive strength of soaked samples (q_u soaked) with lime content for 7, 28 and 56 days cure periods.

4.3.2 Bilkent Clay

A very highly plastic clay type of soil has been mixed with hydrated lime at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%). Samples have been prepared at optimum water content and maximum dry density by means of Harvard miniature compaction test (Wilson, 1950, ASTM D698-07).

Grain size distribution, Atterberg limits, strength and swelling properties of the lime treated clay samples have been determined at each lime content which is increased step by step. The effect of lime stabilization on Atterberg limits, strength and swelling characteristics are given in Figure 4.36 to Figure 4.46.

Atterberg limits, unconfined compression tests and oedometer tests have been conducted on the soil mixtures. The tests have been performed according to the ASTM standards (ASTM 427, 1998; ASTM 2166, 2013; ASTM 4318, 2010; ASTM 4546, 2008).

4.3.2.1 Effects of stabilization on index parameters

When the Figure 4.36 is examined, the effective lime content to decrease the liquid limit and plasticity index can be considered as 4 to 5%. But this effectiveness does not guarantee the allowable swell percentages as will be explained below.

Shrinkage limit increases as the lime content increases as given in Figure 4.36. The soil exhibits the highest value of shrinkage limit at 9% lime content. Up to 7% of lime content, the average increment for shrinkage limit is 10% for each lime content increment. Whereas this increment is doubled (i.e. about 20%) when the lime content increases from 7 to 9%.

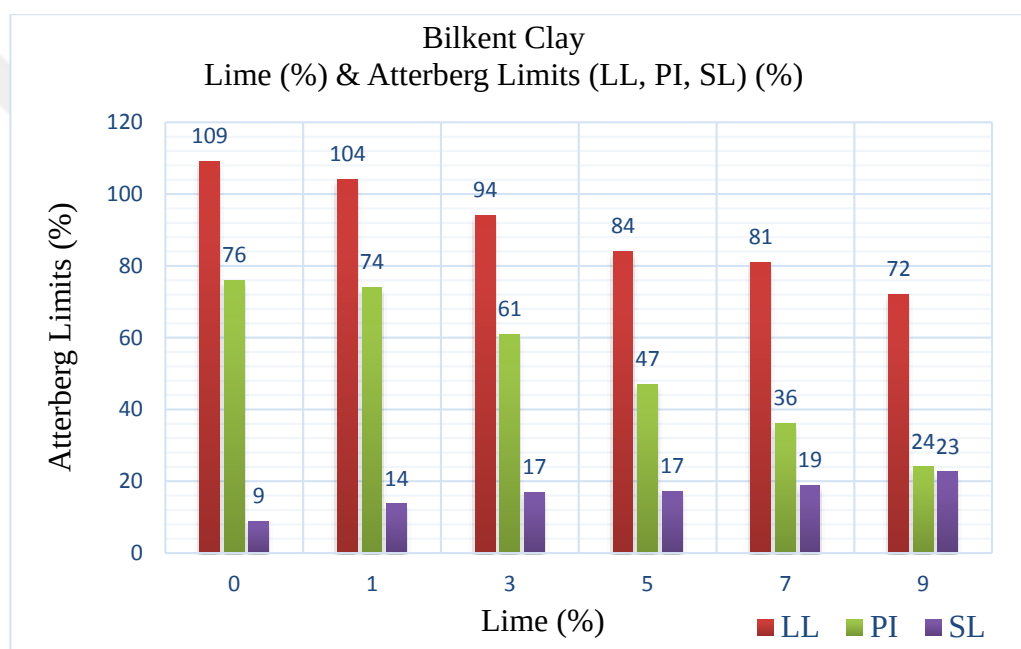


Figure 4.36 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with lime content.

Proctor test results for stabilized samples using lime are given in Table 4.4.

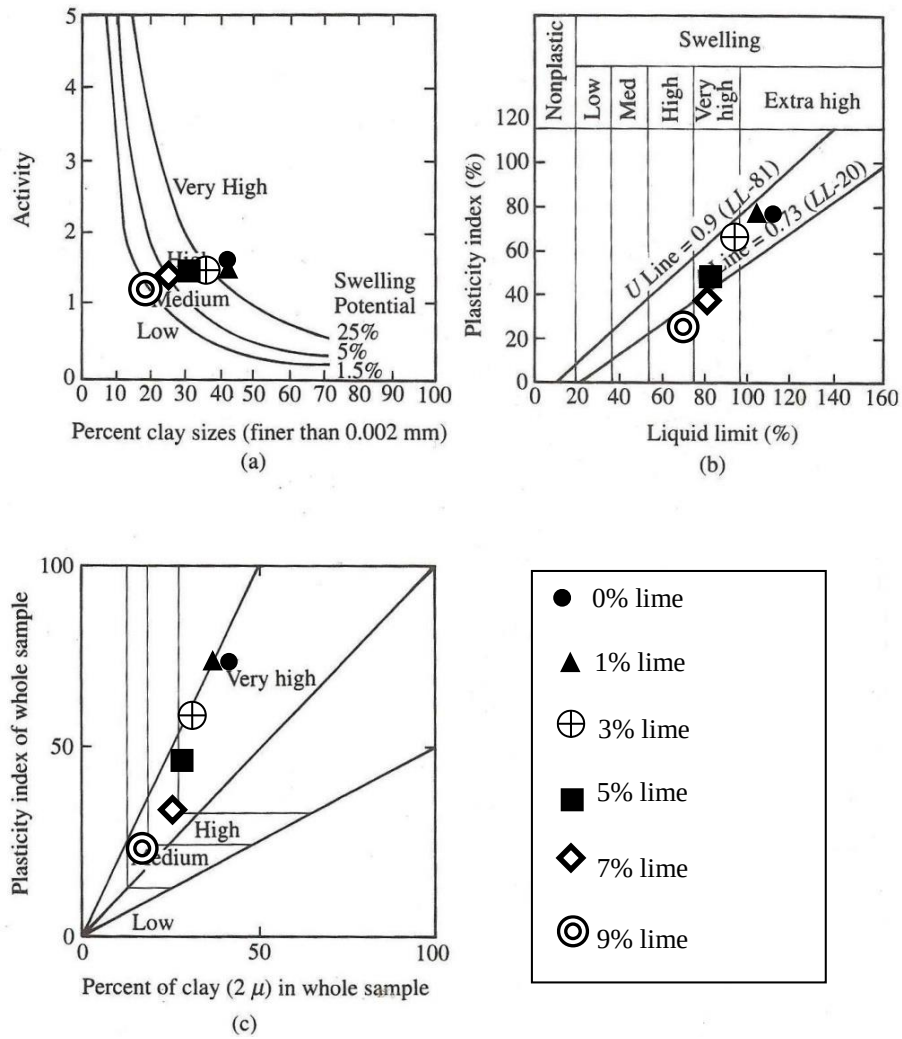
Table 4.4 : Proctor test results for treated samples with lime for Bilkent Clay.

Lime Content (%)	0 (Natural)	1	3	5	7	9
γ_{drymax} (kN/m ³)	13.69	13.15	13.27	13.33	13.44	13.75
W_{opt} (%)	29	27.5	29.2	29.2	29	30.6

4.3.2.2 Effects of stabilization on swell parameters

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on lime treated samples are marked on the charts in Figure 4.37, the corresponding swelling potentials can be specified. According to Seed et al. (1962) and Van der Merwe (1964) approaches, the swelling potential of the sample containing 9% lime, decreases to “low”. However, Dakshanamurthy and Raman’s (1973) chart still gives “high” swell potentials for 9% lime content. The results of oedometer swell tests in this study show that Dakshanamurthy and Raman’s (1973) method, which only takes into account liquid limit, is quite conservative. The methods in which the clay fractions are taken into consideration seems more reliable. As a result, Seed et al. (1962) and Van der Merwe (1964) approaches are more compatible with the results of oedometer swell tests performed in this study (Figure 4.37).

It has been observed that the activity values are generally in a trend of decrease in lime-treated samples. This is because the rate of decrease of plasticity index values is generally larger than the rate of decrease of clay size fraction values. Çokça (2001) also found in a laboratory experiment program, in which an artificial expansive soil (kaolinite+bentonite) was treated with lime and fly ash, that the activity decreased with increasing percent stabilizer.



- (a) Seed et al. (1962)
 (b) Dakshanamurthy and Raman (1973)
 (c) Van der Merwe (1964)

Figure 4.37 : Criteria for determining swell potential (the marks in the figures are for Bilkent Clay) (Das, 1999).

The swell percentages found in laboratory experiments are given in Figure 4.38 for different lime contents. It is seen that the swell percentages are decreasing parabolically with lime content. The swell percentage value has been found under 1% when the lime content reaches about 7% to 9%.

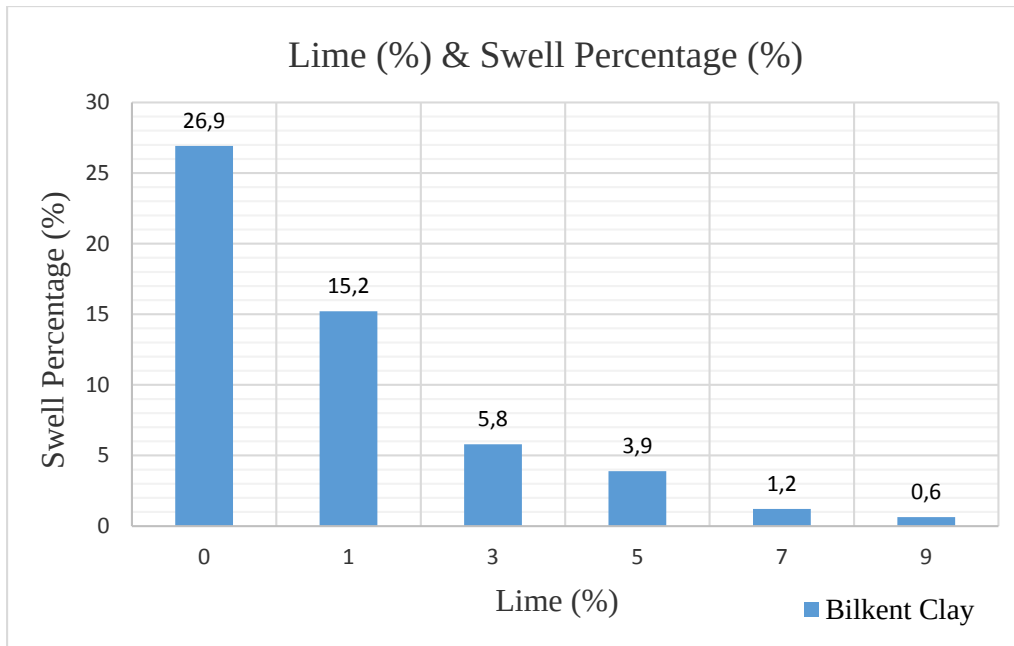


Figure 4.38 : Variation of swell percentage with lime content.

Figure 4.39 below shows the net swell percentages with increasing lime content. This chart has been prepared based on a theoretical calculation in which net swell percentages are calculated with proportional to natural clay swell percentage (i.e. natural clay is composed of 100% clay and its swell percentage has been measured as 26.9% in experiment, then with 1% lime, the clay content is 99%, thus the net swell percentage is calculated as 26.7%).

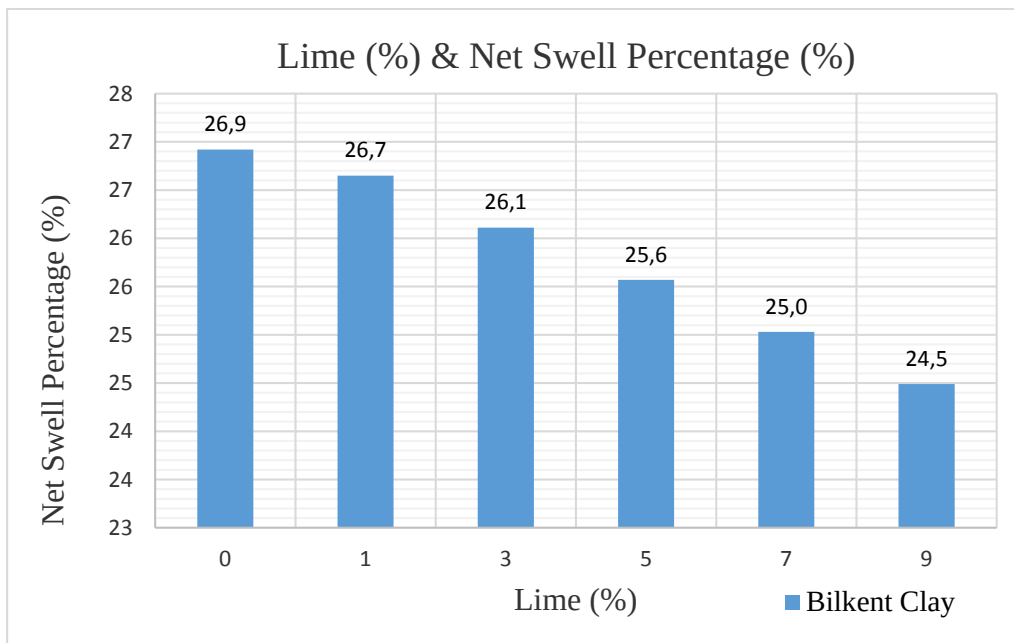


Figure 4.39 : Variation of net swell percentage with lime content.

Figure 4.40 shows the effect of lime on swell percentage as lime content increases. This chart is simply the difference between the charts given in Figure 4.38 and 4.39. It should be noted here that the swell percentage decrease is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between lime and clay. Figure 4.40 shows only the latter effect. Thus, the effect of lime on swell percentage diminishes after 7% lime content as observed in Figure 4.40. The optimum lime content in Figure 4.40 is about 3% as far as the swelling is concerned. In a design of sub-base of transportation structures (i.e. highways, railways), this optimum lime content can be taken as a reference. Swell tests should be executed starting with this reference value. In case of not satisfying the design criteria, this optimum lime content should be increased. For instance, the allowable swell percentage for transportation structures is generally accepted as 0.5% as given in Turkish Highway Standards, KGM, 2005. This criterion is compatible with the Seed et al.'s (1962) categorization in which the degree of expansion is “low” for potential swell 0 to 1.5%. It is inferred from Figure 4.38 that about 9% to 10% lime content is necessary in Bilkent Clay for 0.5% swelling, for example. Thus, lime contents larger than 3% optimum value may especially be necessary to satisfy the design criteria in the very highly plastic clays such as encountered in Bilkent district of Ankara.

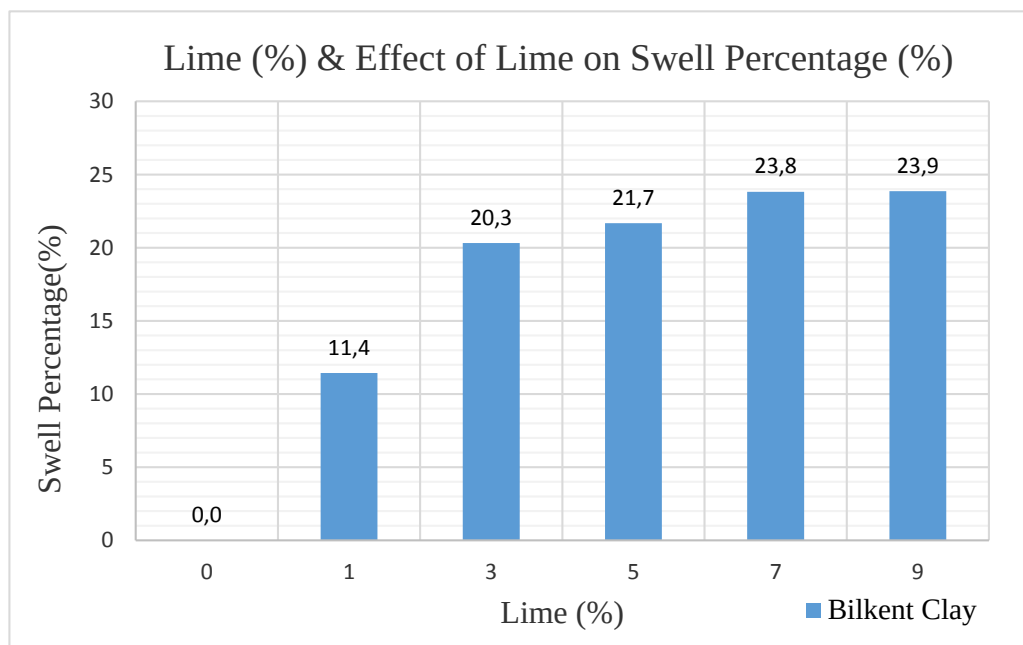


Figure 4.40 : Effect of lime content on swell percentage.

The swell pressures have been found in laboratory experiments with different percentages of the lime agent. The results are given in Figure 4.41 below. The swell pressure decreased considerably at 7% lime content as given in this figure in accordance with the above evaluations.

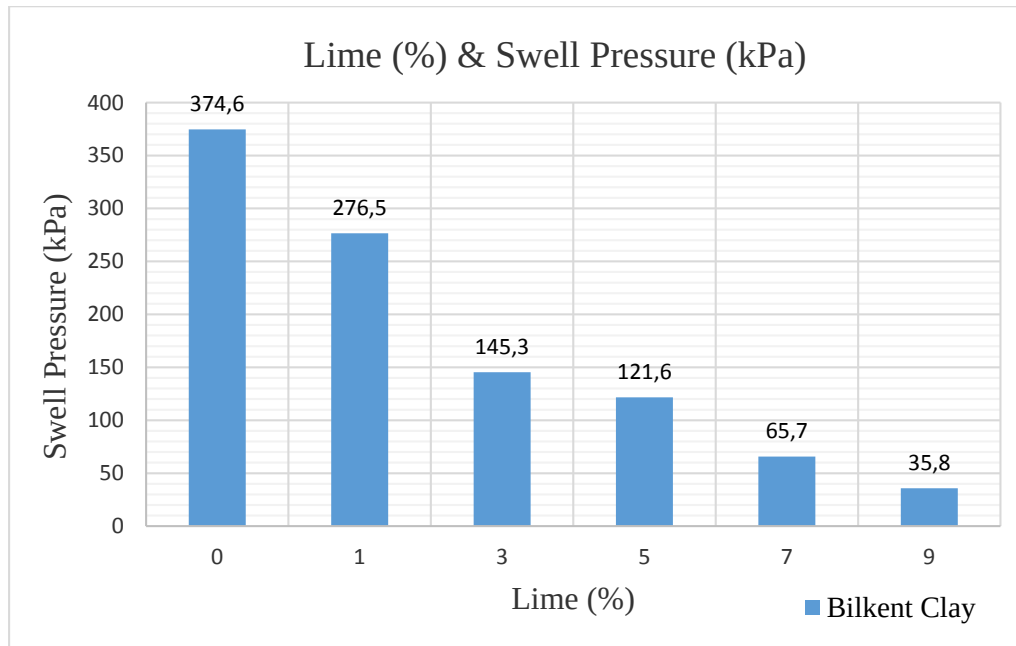


Figure 4.41 : Variation of swell pressure with lime content.

Figure 4.42 and Figure 4.43 show the net swell pressure and the effect of lime on swell pressure with increasing lime content respectively. The explanations are similar to ones given in swell percentage subject above. Therefore, the effect of lime on swell pressure is maximum in lime content increase from 1% to 3%. This is compatible with the result observed in swell percentage subject.

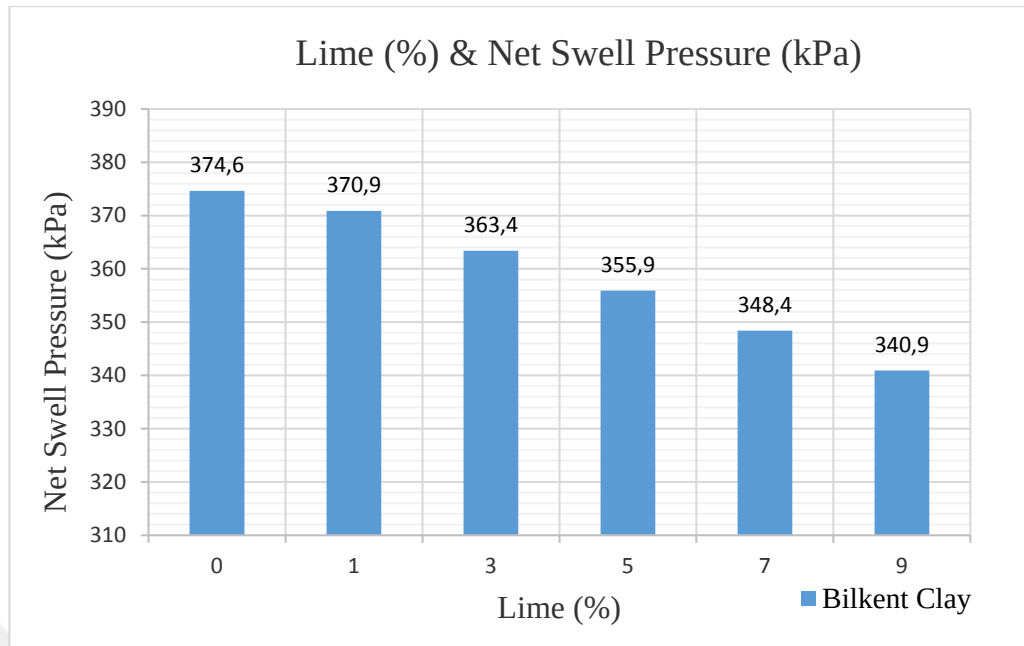


Figure 4.42 : Variation of net swell pressure with lime content.

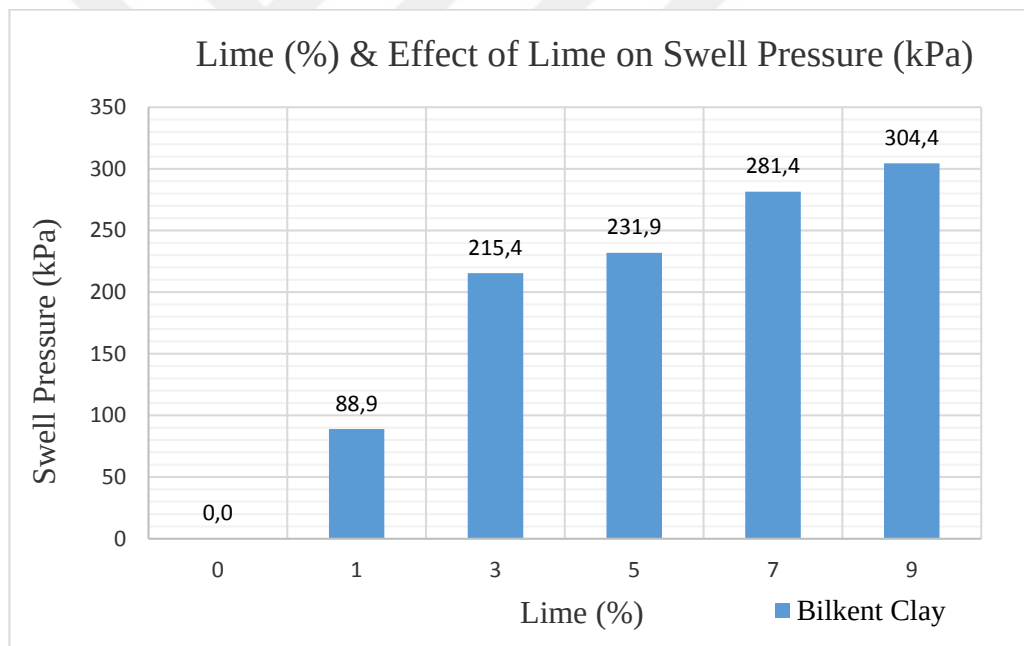


Figure 4.43 : Effect of lime content on swell pressure.

4.3.2.3 Effects of stabilization on strength parameters

Figure 4.44 shows the variation of unconfined compressive strength with increasing lime content for non-soaked and soaked conditions. It should be noticed here that perforated moulds have been used to inundate the soil sample to prepare soaked samples. It is inferred from the results that the unconfined compressive strengths are greater in non-soaked condition up to 7% lime content. Starting from 7%, the strengths

are greater in soaked condition due to pozzolanic reactions with the effect of water. It can be accepted that strength values in soaked and non-soaked conditions are practically same at about 6% lime content (Figure 4.44).

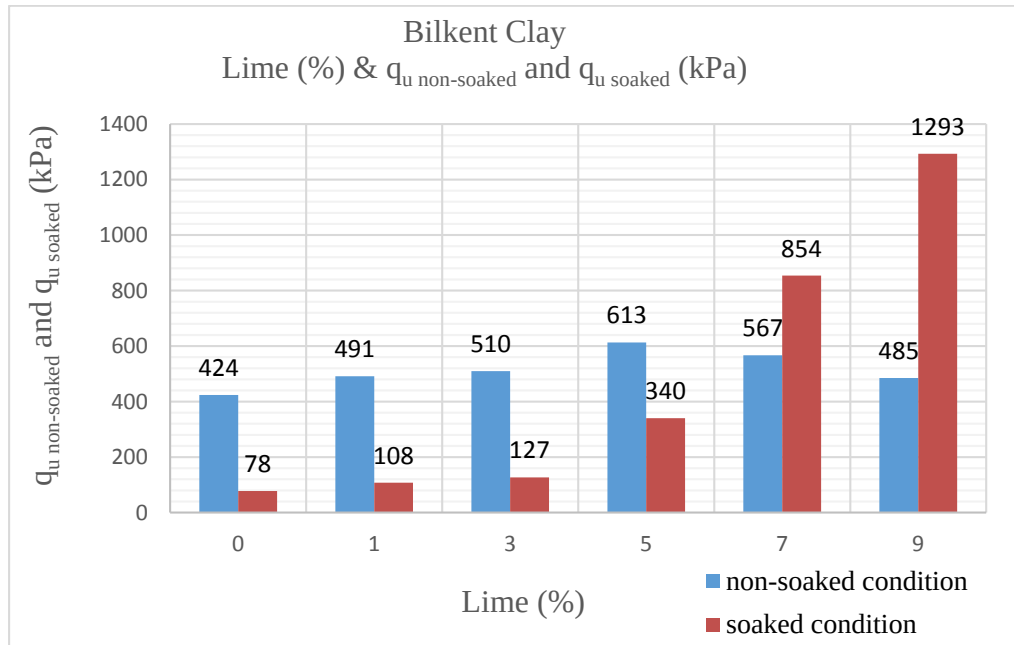


Figure 4.44 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with lime content.

4.3.2.4 Effects of stabilization on compression parameters

In this part, effect of lime stabilization on compression parameters for Bilkent clay is explained. Using the results of odometer tests, void ratio (e) versus effective stress (σ') graphs are prepared first. Void ratio (e) versus $\log(\sigma')$ graphs can be drawn then so that the graphs can be approximated to straight lines. Both compression and expansion part of the test can be monitored to get compression and expansion indices from the slopes of the straight lines. C_c is the slope of the virgin compression line of the test. High value of C_c means high compressibility. The expansion index (C_e) is the slope of the unloading part of the test plot. In the same way, the higher the C_e value, the higher the expansion capacity of the clay. The compressibility and expansion potential of highly plastic clays cause problems in foundation engineering applications as stated before. Lime stabilization effect in this context is to be evaluated below for Bilkent Clay.

e - σ' and e - $\log \sigma'$ graphs are given in Figure 4.45 and Figure 4.46 respectively. The variations of compression index (C_c) and expansion index (C_e) values with lime

content are given in Figure 4.47 and Figure 4.48 respectively. It is noticed that the clay sample is quite compressible in natural state. Both compression and expansion (recompression) curves are steep in natural state of the soil.

The preconsolidation pressure of the natural Bilkent Clay can be determined as $\sigma_c \approx 4 \text{ kg/cm}^2$ from Figure 4.46. Since the depth of sampling is about 5m for Bilkent Clay, the over-consolidation ratio (OCR) for it can be calculated as about OCR=4.

The slope of compression and recompression curves are not so different from the natural soil curves in 1% and 3% lime contents. Starting from 5% lime content the picture is different. The compressibility and swelling properties are improved starting from 5% lime content as noticed in Figure 4.46. This behavior can be observed more clearly in Figure 4.47 and Figure 4.48. It is realized, on the other hand, that void ratio values are about 10% larger in 7% lime content than the ones in 5% lime content. This may be due to flocculation process in lime stabilized samples. Thus, the optimum lime content can be accepted as 5% for Bilkent Clay after curing.

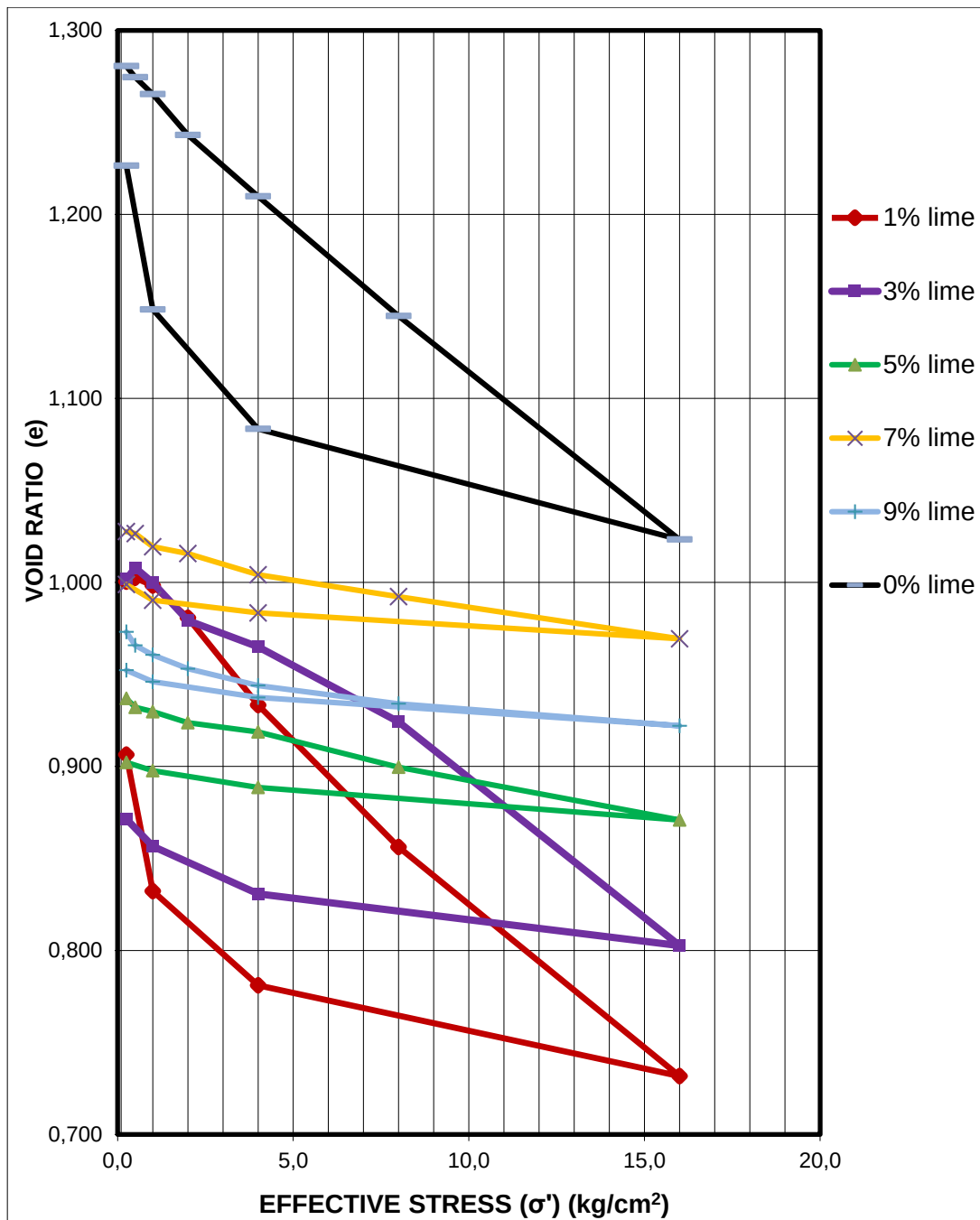


Figure 4.45 : e - σ' relationship for natural and lime-treated soil samples for Bilkent Clay.

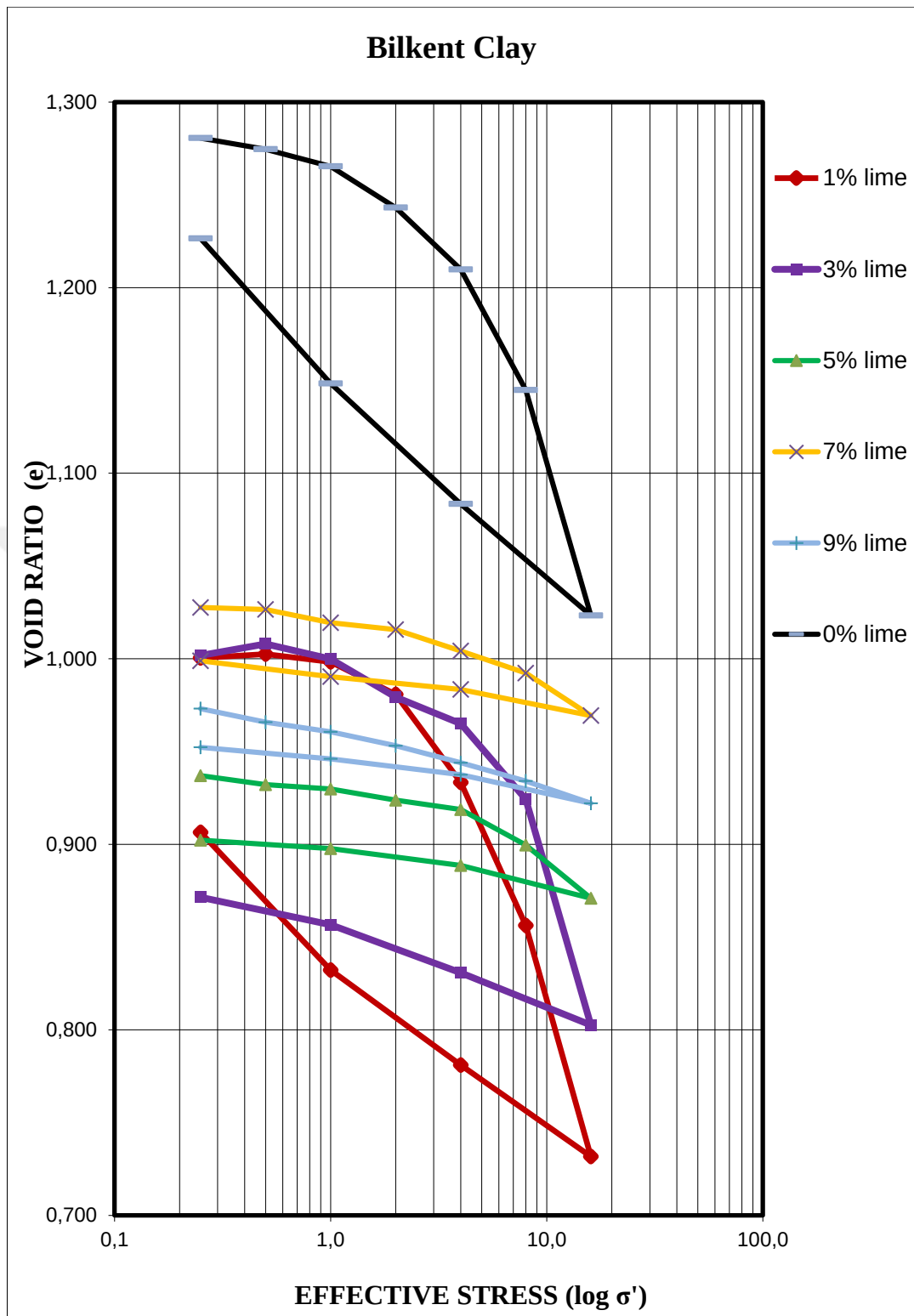


Figure 4.46 : e-log σ' relationship for natural and lime-treated soil samples for Bilkent Clay.

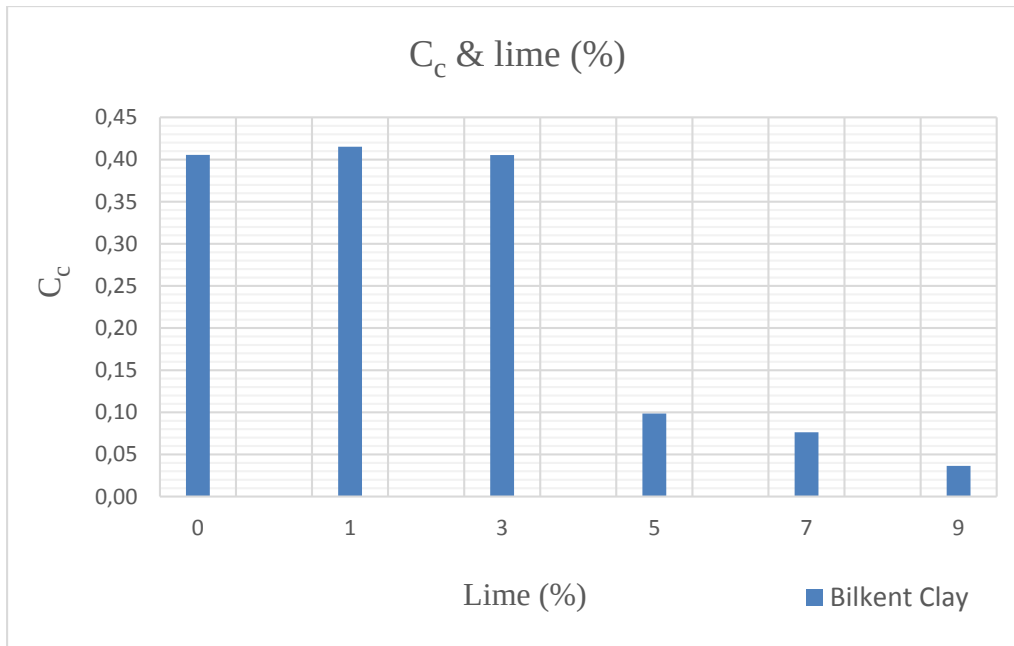


Figure 4.47 : Compression index (C_c) values with increasing lime content for Bilkent Clay.

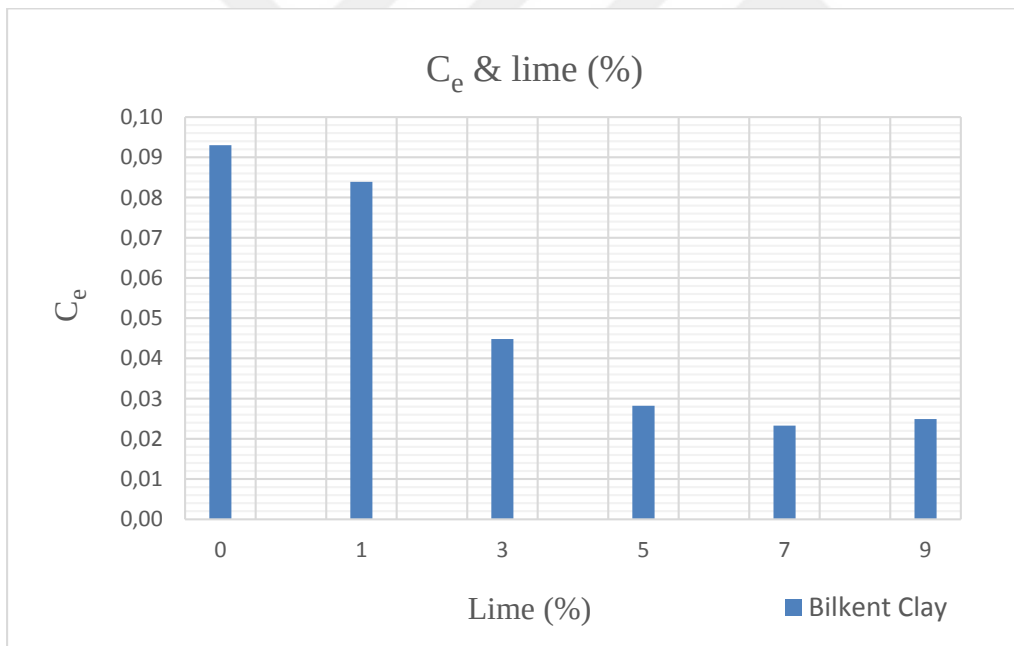


Figure 4.48 : Expansion index (C_e) values with increasing lime content for Bilkent Clay.

4.4 Fly Ash Stabilization

The stabilization effect of fly ash stabilization on both clays have been examined and the results are presented in this section. Once again, the index and swelling properties, strength values for both soaked and non-soaked conditions, resilient modulus and

methylene blue values and compressibility characteristics have all been investigated. In addition to above, time effect on the resilient modulus values has been tested. The results are as follows:

4.4.1 Esenboğa Clay

The Esenboğa clay this time has been mixed with fly ash at different percentages of dry weight of the soil (5%, 10%, 15%, 20% and 25%). Atterberg limits, strength and swelling properties of the treated soil samples have been examined on samples prepared for each mix ratios given above. The effect of fly ash stabilization on index and engineering properties of the plastic clay are given in Figure 4.49 to Figure 4.59.

Atterberg Limits tests, unconfined compression tests and oedometer tests have been conducted on the soil mixtures. Tests have been performed according to the ASTM Standards.

4.4.1.1 Effects of stabilization on index parameters

When Figure 4.49 is evaluated, it is realized that liquid limit and plasticity index values are increased significantly by addition of 5% fly ash. These increased liquid limit and plasticity index values cannot be decreased to untreated soil values up to about 15% to 25% fly ash contents. This means that fly ash treatment in this extremely highly plastic clay is not effective up to about 20% as an average as far as the liquid limit and plasticity index values are concerned.

Shrinkage limit increases as the fly ash content increases as given in Figure 4.49. The soil exhibits the highest value of shrinkage limit at 25% fly ash content. The average increment for shrinkage limit is about 11% for each fly ash content increment of 5%.

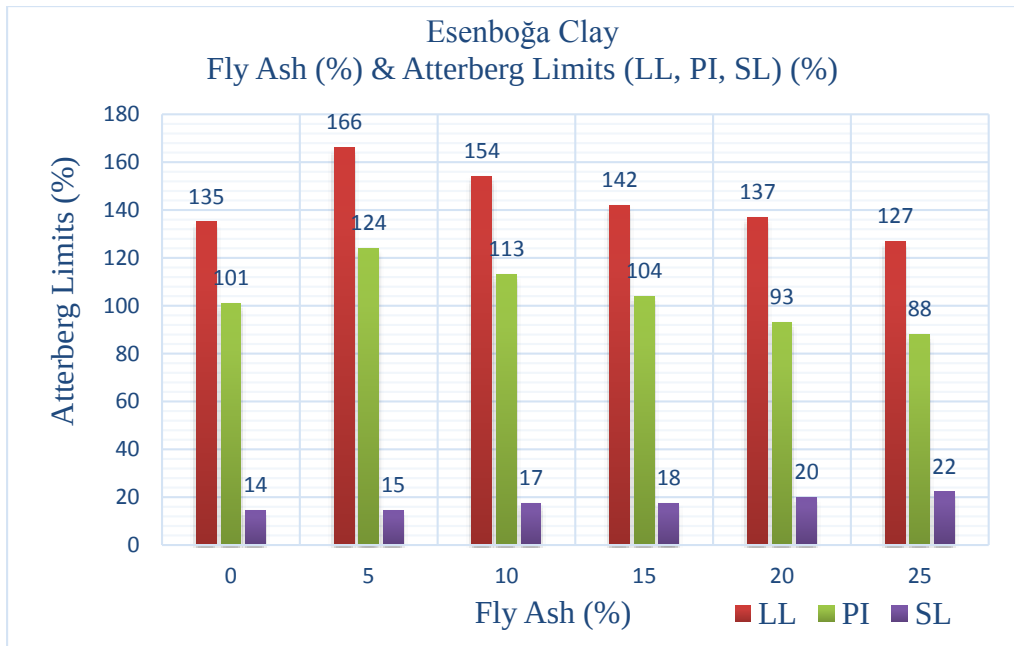


Figure 4.49 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with fly ash content.

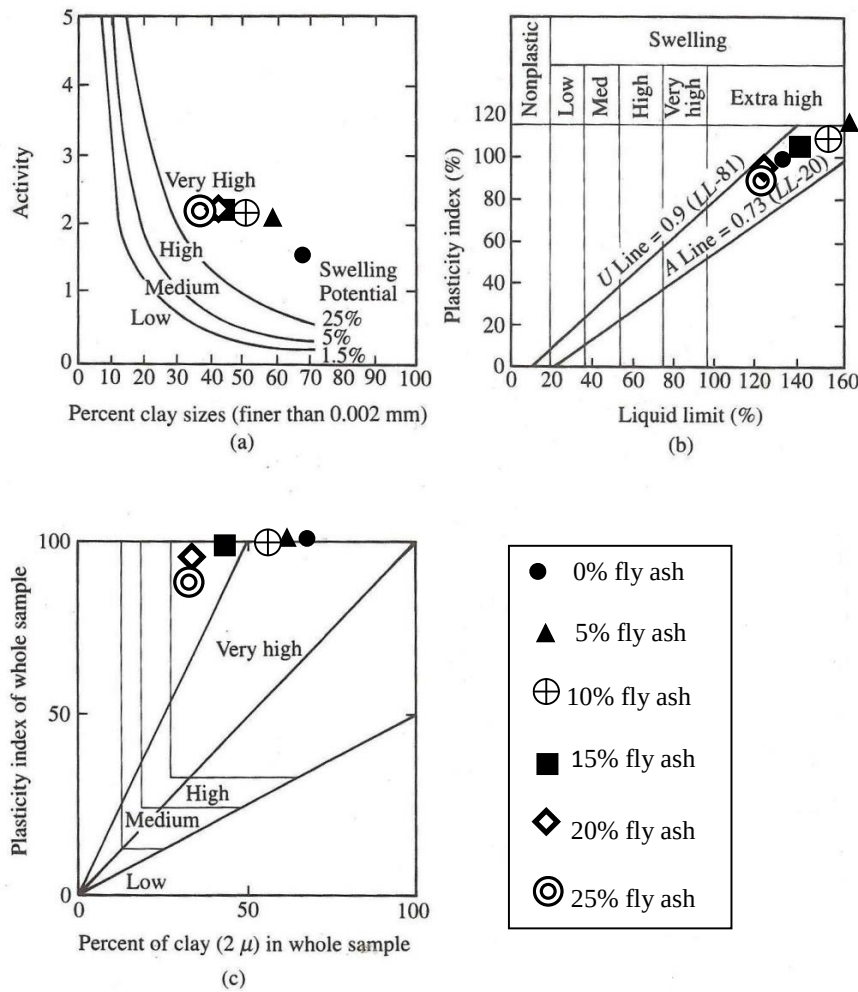
Proctor test results for stabilized samples using fly ash are given in Table 4.5.

Table 4.5 : Proctor test results for treated samples with fly ash for Esenboğa Clay.

Fly Ash Content (%)	0 (Natural)	5	10	15	20	25
γ_{drymax} (kN/m ³)	11.60	12.84	13.14	13.08	11.95	11.91
W_{opt} (%)	39.5	33.5	35.5	36.5	35.0	33.5

4.4.1.2 Effects of stabilization on swell parameters

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on fly ash treated samples are marked on the charts in Figure 4.50, swell potentials can be evaluated. The results show that all fly ash-treated samples still have “very high” swell potentials. It is concluded according to these charts that fly ash agent is not effective in this clay to decrease swell potential.



(a) Seed et al. (1962)
(b) Dakshanamurthy and Raman (1973)
(c) Van der Merwe (1964)

Figure 4.50 : Criteria for determining swell potential (the marks in the figures are for Esenboğa Clay) (Das, 1999).

Variation of swell percentage with lime content found in the laboratory experiments is given in Figure 4.51 below. It can be inferred from this figure that the swell percentage values are still high (i.e. about 8 to 10%) for fly ash amounts of 20 to 25%.

For instance, the allowable swell percentage for transportation structures is generally accepted as 0.5% as given in Turkish Highway Standards, KGM, 2005. This criterion is compatible with the Seed et al.'s (1962) categorization in which the degree of expansion is "low" for potential swell 0 to 1.5%. The maximum fly ash content in this study (25%) is not adequate to obtain allowable swell percentages for transportation structures, as shown in Figure 4.51. There are reductions in swell percentages as the fly ash content increases but at the highest amount of fly ash (25%) the swell

percentage is still 8.61% which is quite high. It can be stated here that fly ash improvement of an extremely highly plastic clay is not possible within reasonable fly ash content limits as far as the swell percentage criterion is concerned.

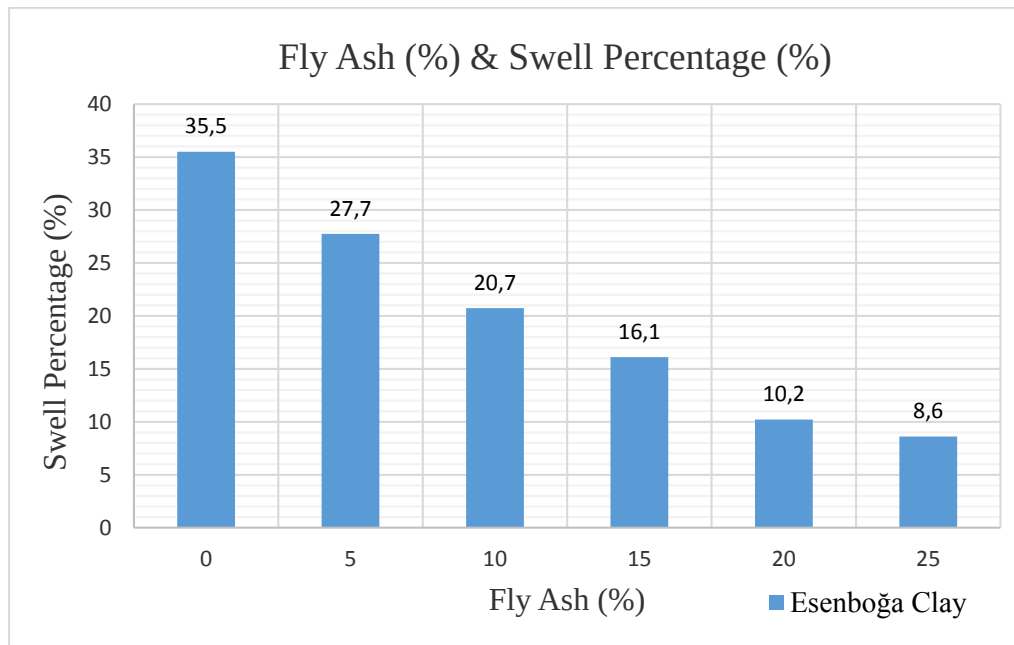


Figure 4.51 : Variation of swell percentage with fly ash content.

Figure 4.52 shows the effect of fly ash content on net swell percentage. This chart has been prepared based on a theoretical calculation in which net swell percentages are calculated with proportional to measured natural clay swell percentage (i.e. natural clay is composed of 100% clay and its swell percentage has been measured as 35.5% in experiment; then with 5% fly ash, the clay content is 95%, thus the net swell percentage is calculated as 33.7%, the calculations are similar with other fly ash contents).

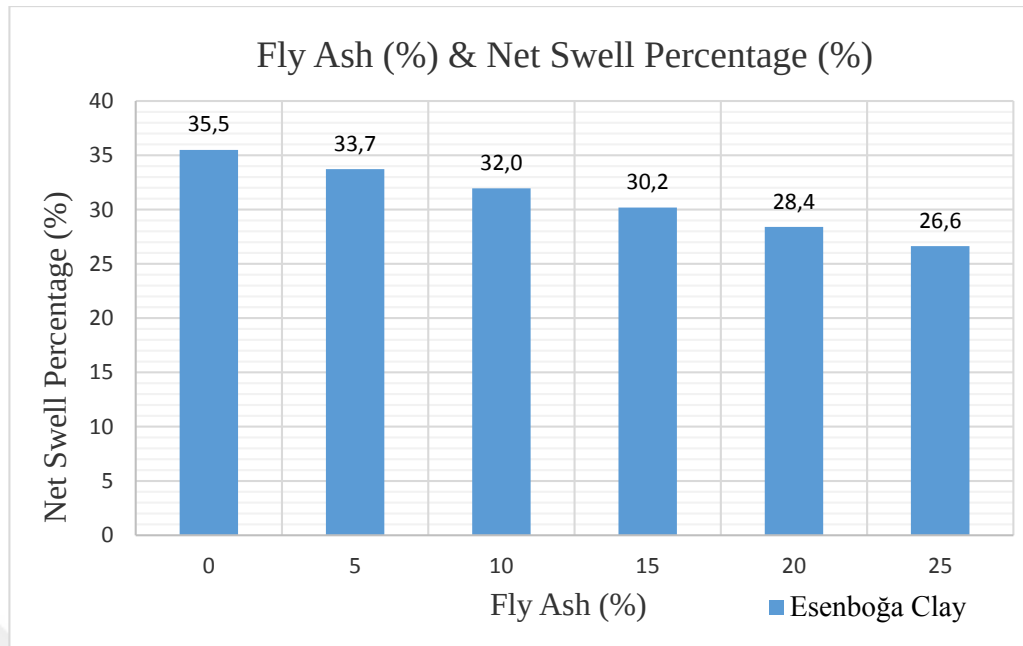


Figure 4.52 : Variation of net swell percentage with fly ash content.

Figure 4.53 shows the effect of fly ash on swell percentage as fly ash content increases. This chart is simply the difference between the swell percentages found in the laboratory experiments shown in Figure 4.51 and the values given in Figure 4.52. It should be noted here that a decrease in swell percentage is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between fly ash and clay. Figure 4.53 shows only the latter effect. Thus, the effect of fly ash on swell percentage diminishes after 20% fly ash content as observed in Figure 4.53. It is also noticed that the swell percentages found in laboratory experiments (Figure 4.51) are still 8.5% to 10% for fly ash contents 20% to 25% respectively. These swell percentages are generally not acceptable in a transportation sub-structure design. This result is compatible with the result mentioned above in preliminary design charts (Figure 4.50). Thus, it can be concluded that the fly ash agent alone may not be sufficient for swelling design criteria in such type of extremely highly plastic clays.

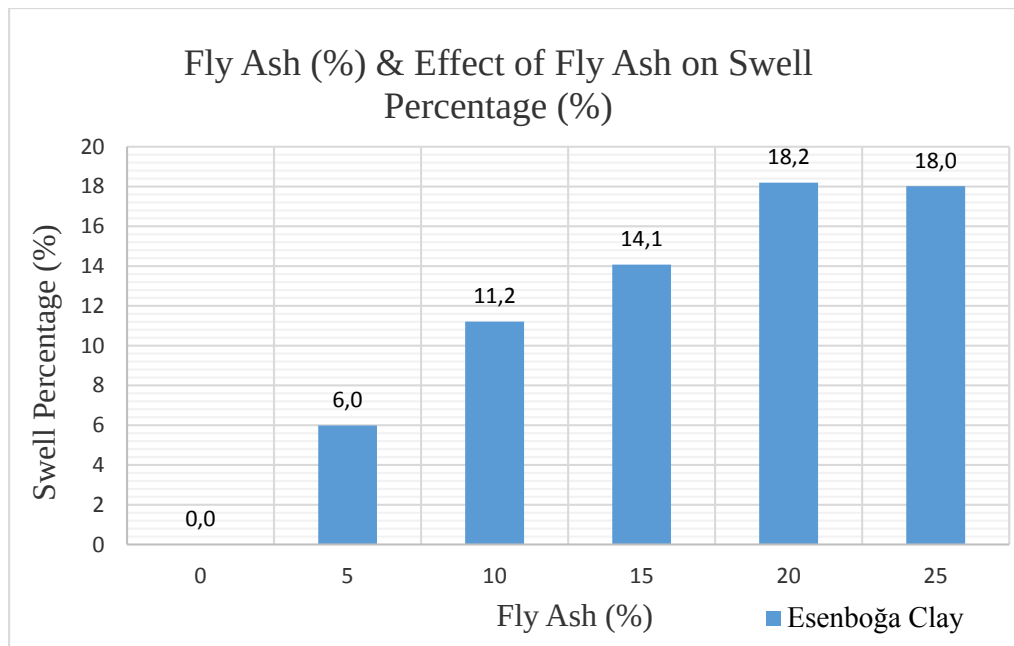


Figure 4.53 : Effect of fly ash content on swell percentage.

Figure 4.54 gives the swell pressures with fly ash content found in laboratory experiments. Swell pressures are decreased continuously with increasing fly ash content as given in this figure. This decrease is about 7.5% as an average for each fly ash content increment up to 15% (i.e. for 5%, 10% and 15%). Swell pressures are decreased more significantly after 15% fly ash content. The swell pressure is decreased by %23 from 15% to 20% and by 15% from 20% to 25% fly ash contents (Figure 4.54).

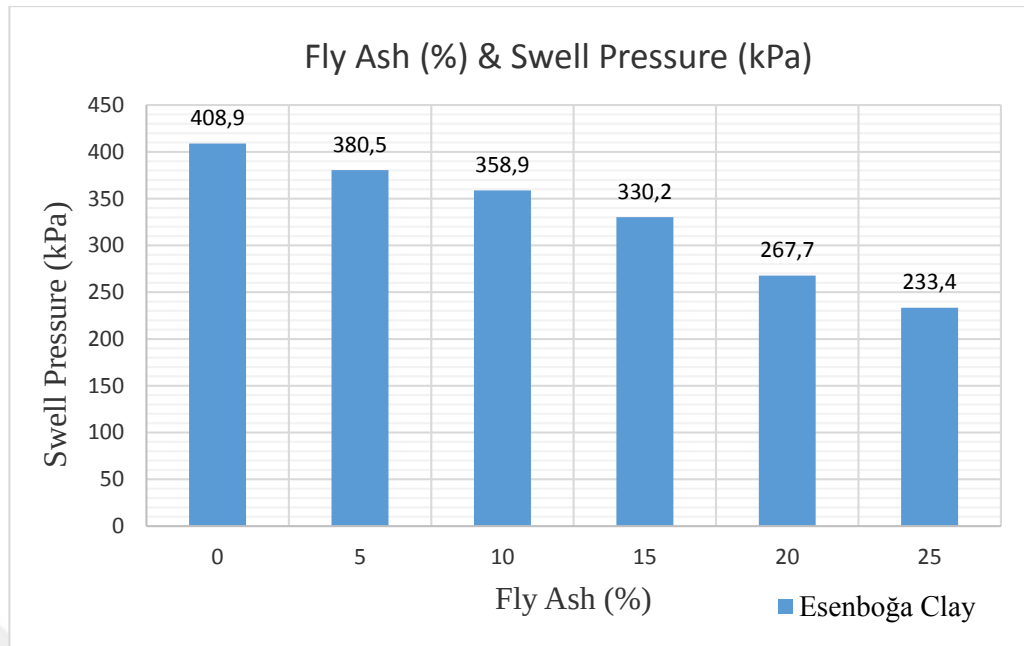


Figure 4.54 : Variation of swell pressure with fly ash content.

Figure 4.55 and Figure 4.56 show the net swell pressure and the effect of fly ash on swell pressure with increasing fly ash content respectively. The explanations are similar to ones given in swell percentage subject above. Therefore, the effect of fly ash on swell pressure is maximum in fly as content increase from 15% to 20%. This is compatible with the result observed in swell percentage subject.

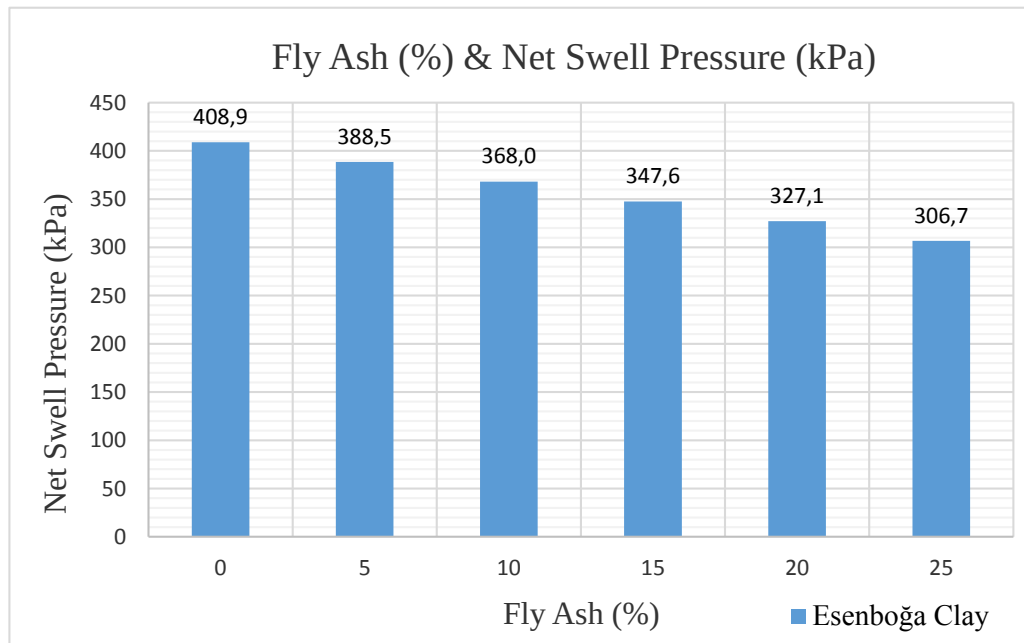


Figure 4.55 : Variation of net swell pressure with fly ash content.

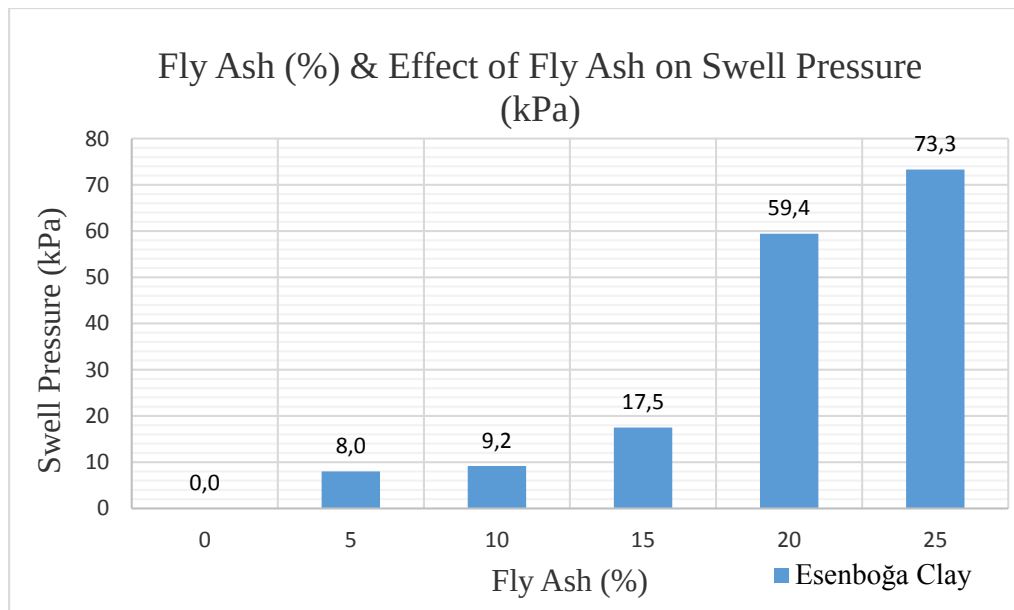


Figure 4.56 : Effect of fly ash content on swell pressure.

4.4.1.3 Effects of stabilization on strength parameters

Figure 4.57 gives the variation of unconfined compressive strengths with fly ash content for non-soaked and soaked conditions. The unconfined compressive strengths are always greater in non-soaked condition as expected. The strengths increase by adding fly ash agent up to 15% content and then decrease for both non-soaked and soaked conditions. Therefore 15% fly ash content can be considered as optimum when the unconfined compressive strength is taken into account.

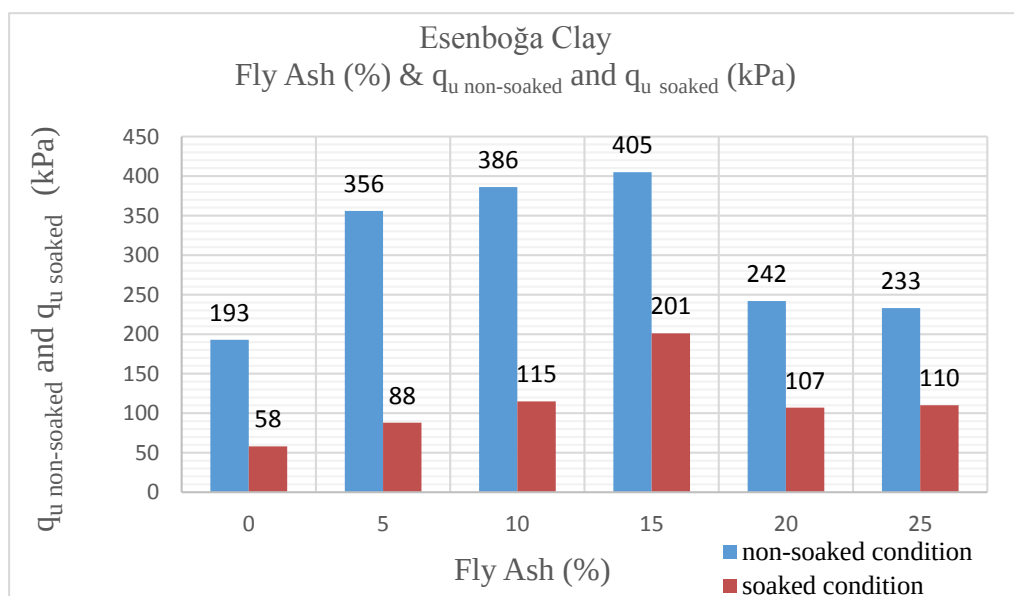


Figure 4.57: Variation of unconfined compressive strength of non-soaked (q_u non-soaked) and soaked (q_u soaked) samples with fly ash content.

4.4.1.4 Effects of stabilization on compression parameters

In this part, effect of fly ash stabilization on compression parameters for Esenboğa Clay is explained. Using the results of odometer tests, void ratio (e) versus effective stress (σ') graphs are prepared first. Void ratio (e) versus $\log(\sigma')$ graphs can be drawn then so that the graphs can be approximated to straight lines. Both compression and expansion part of the test can be monitored to get compression and expansion indices from the slopes of the straight lines. C_c is the slope of the virgin compression line of the test. High value of C_c means high compressibility. The expansion index (C_e) is the slope of the unloading part of the test plot. In the same way, the higher the C_e value, the higher the expansion capacity of the clay. The compressibility and expansion potential of highly plastic clays cause problems in foundation engineering applications as stated before. Fly ash stabilization effect in this context is to be evaluated below for Esenboğa Clay.

e - σ' and e - $\log \sigma'$ graphs are given in Figure 4.58 and Figure 4.59 respectively. The variations of compression index (C_c) and expansion index (C_e) values with fly ash content are given in Figure 4.60 and Figure 4.61 respectively. It is noticed that the clay sample is quite compressible in natural state. Both compression and expansion (recompression) curves are steep in natural state of the soil. The compressibility and expansion properties are improved slightly starting from 10% fly ash content as noticed in Figure 4.60. It is interesting to note that compressibility of the fly ash-treated samples starts to increase again in 20% fly ash content. This means that the amount of fly ash has a negative effect at about 20% contents (Figure 4.60). It is noticed, on the other hand, the expansion index has started to decrease in 10% fly ash content and remained almost unchanged afterwards (Figure 4.61). As a result, it can be said that the effect of fly ash on the compressibility characteristics of Esenboğa Clay is quite unstable.

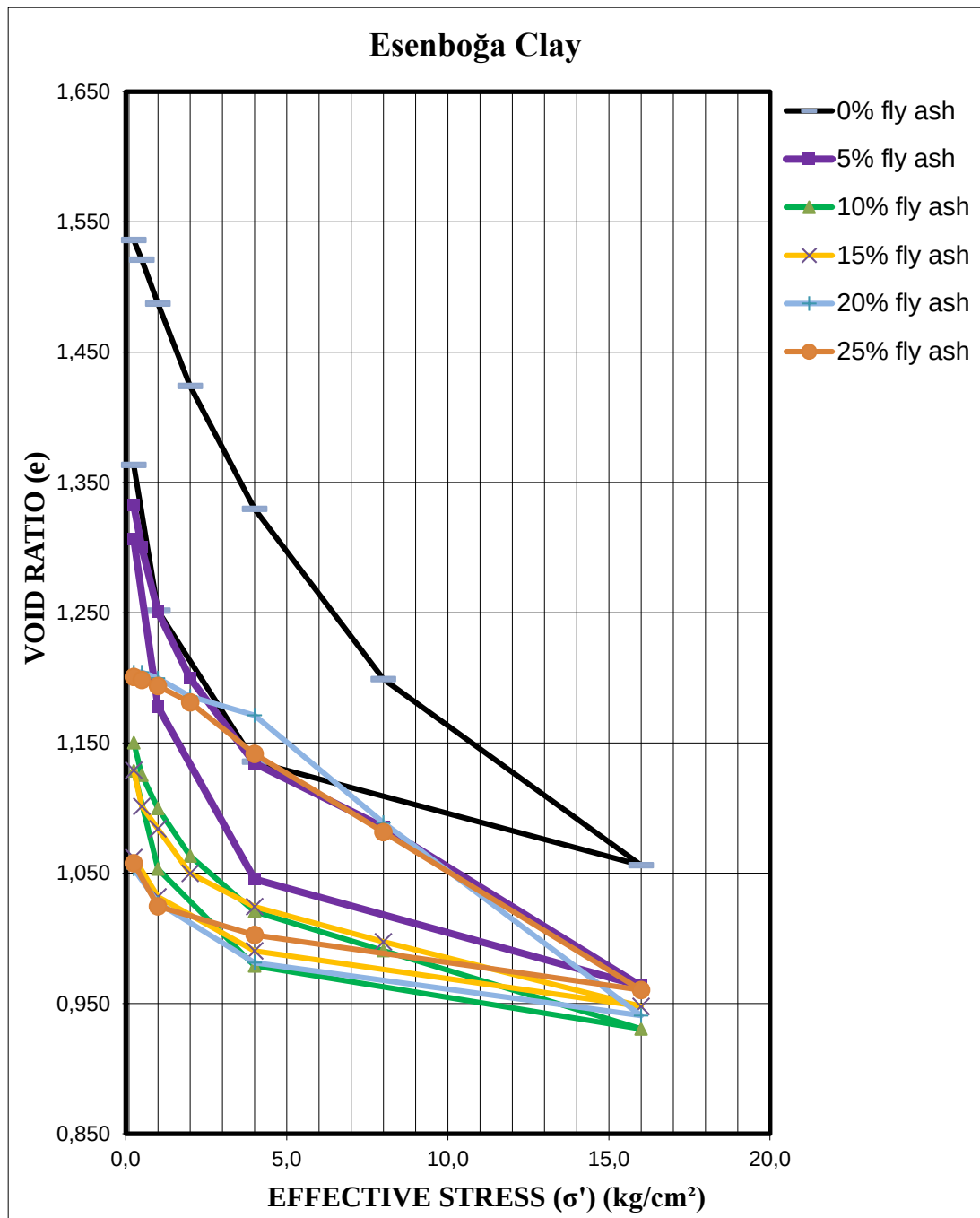


Figure 4.58 : e - σ' relationship for natural and fly ash-treated soil samples for Esenboğa Clay.

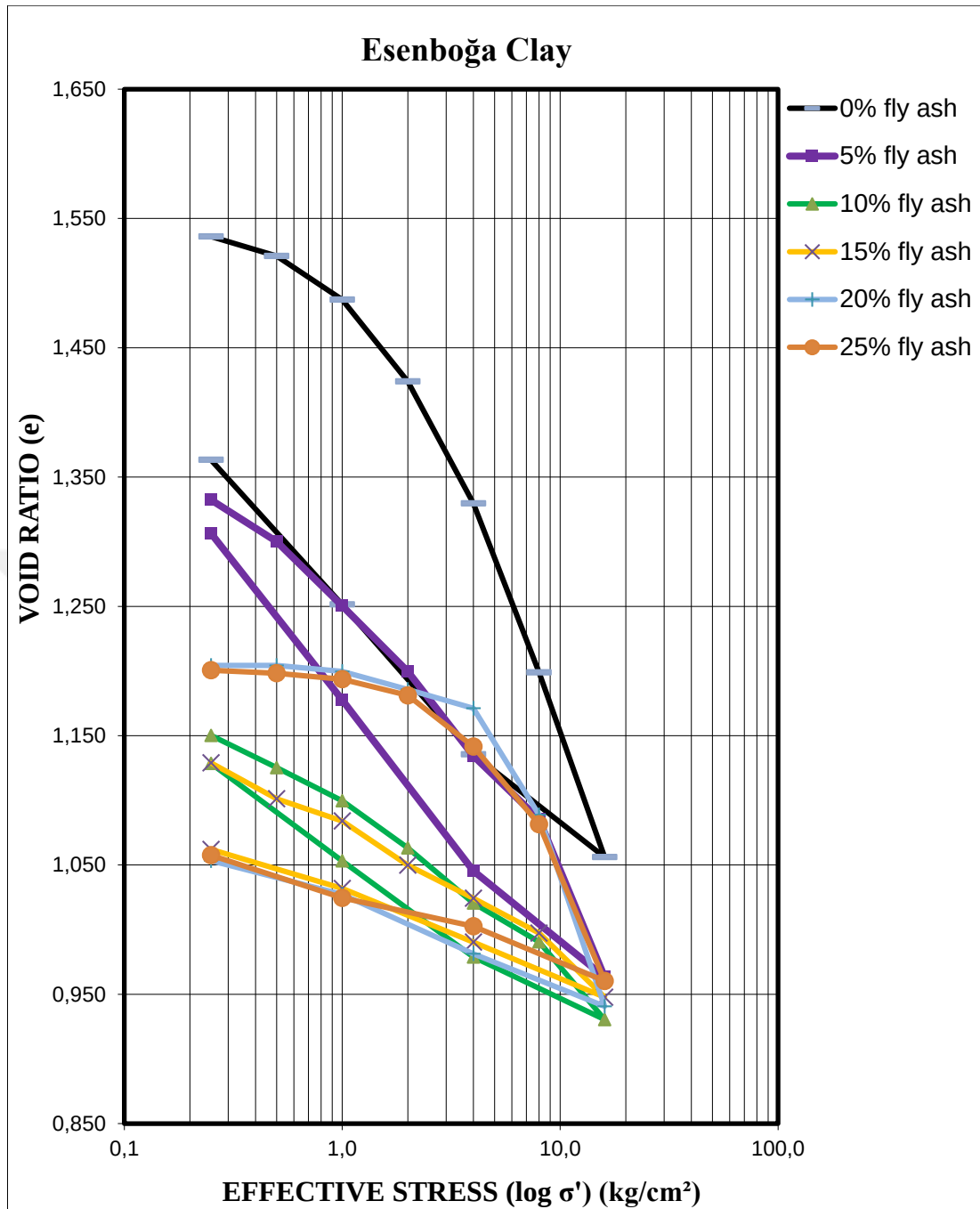


Figure 4.59 : e-log σ' relationship for natural and fly ash-treated soil samples for Esenboğa Clay.

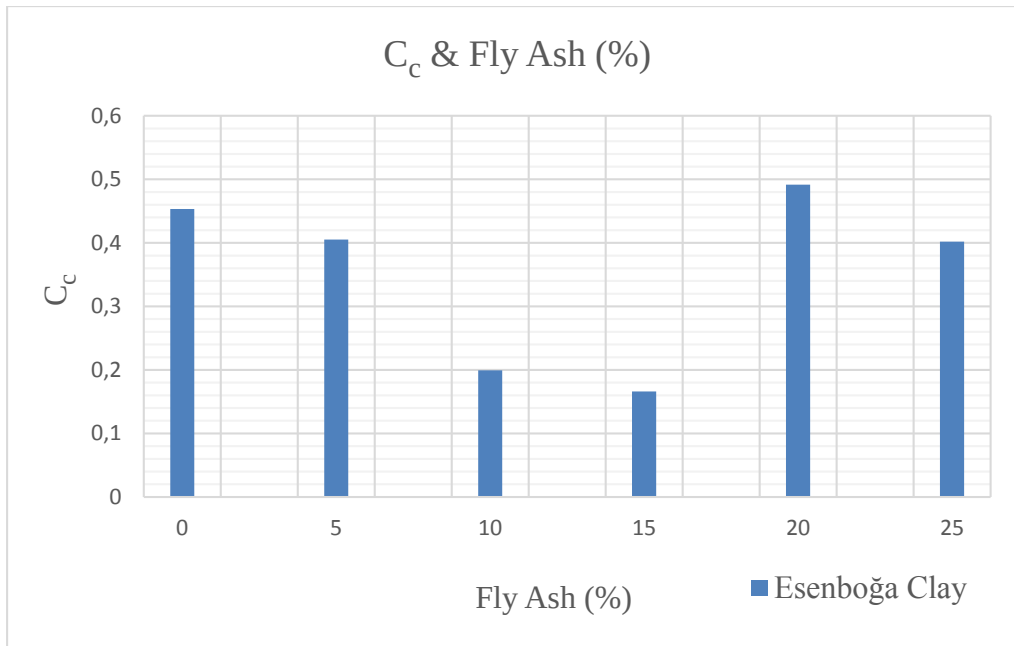


Figure 4.60 : Compression index (C_c) values with increasing fly ash content for Esenboğa Clay.

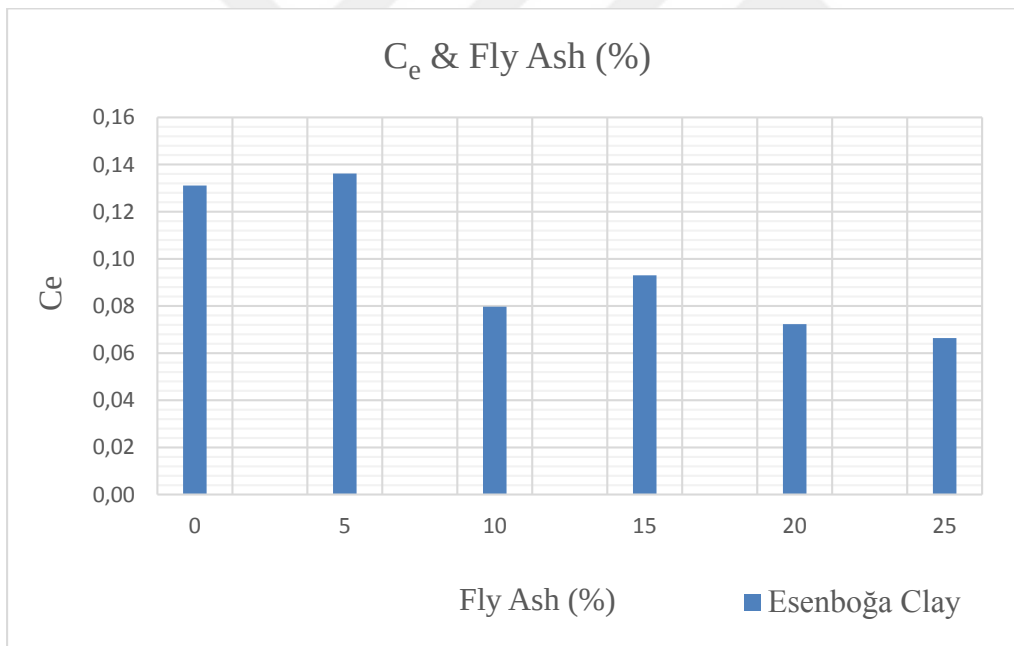


Figure 4.61 : Expansion index (C_e) values with increasing fly ash content for Esenboğa Clay.

4.4.2 Bilkent Clay

The Bilkent Clay this time has been mixed with fly ash at different percentages of dry weight of the clay (5%, 10%, 15%, 20% and 25%). Atterberg limits, strength and swelling properties of the treated clay samples have been examined on samples

prepared for each mix ratios given above. The effect of fly ash stabilization on index and engineering properties of the plastic clay are given in Figure 4.62 to Figure 4.72.

Atterberg limits tests, unconfined compression tests and oedometer tests have been conducted on the soil mixtures. Tests have been performed according to the ASTM Standards.

4.4.2.1 Effects of stabilization on index parameters

When the Figure 4.62 is evaluated, it is realized that liquid limit and plasticity index values are decreased continuously in a stable manner as fly ash content is increased. In each fly ash increment of 5% the decrease in liquid limit is between 2% and 9%. For plasticity index, this decrease is between 3% and 13%.

Shrinkage limit increases as the fly ash content increases as given in Figure 4.62. The first jump in shrinkage limit is quite high from natural soil to 5% fly ash content (i.e. about 44%). Increases in shrinkage limits are moderate in later fly ash contents (i.e. shrinkage limit increases by 3.6% to 18%).

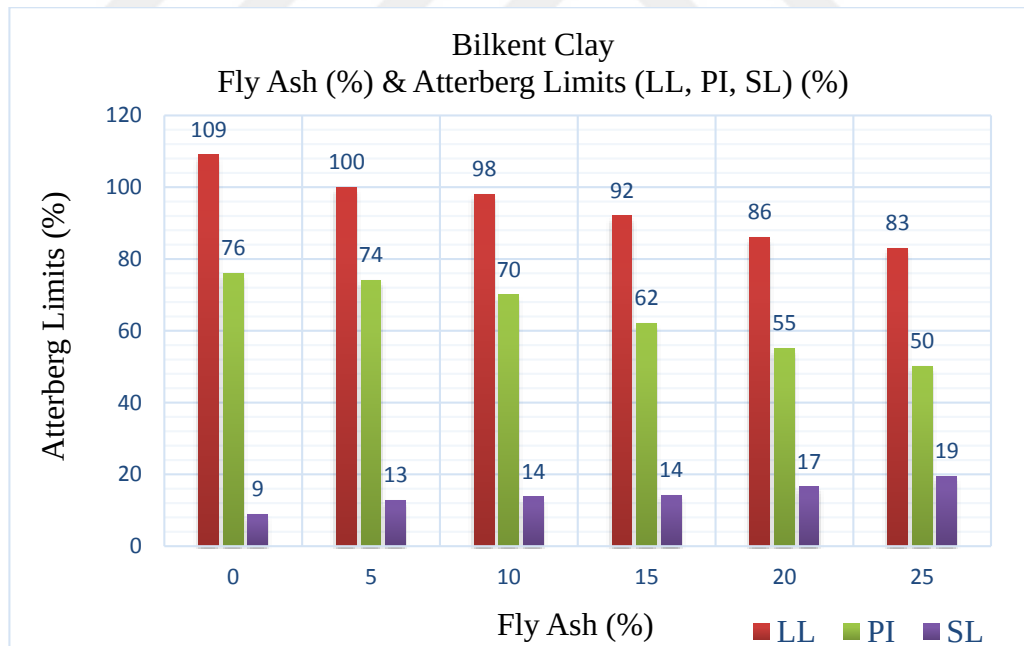


Figure 4.62 : Variation of Atterberg limits; liquid limit (LL), plasticity index (PI) and shrinkage limit (SL) with fly ash content.

Proctor test results for stabilized samples using fly ash are given in Table 4.6.

Table 4.6 : Proctor test results for treated samples with fly ash for Bilkent Clay.

Fly Ash Content (%)	0 (Natural)	5	10	15	20	25
γ_{drymax} (kN/m ³)	13.69	13.23	13.16	13.20	13.52	13.70
W_{opt} (%)	29	27.7	28	29.4	28.5	27.3

4.4.2.2 Effects of stabilization on swell parameters

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on fly ash treated samples are marked on the charts in Figure 4.63, swell potentials can be evaluated. The swell potentials are still “high” to “very high” for 5% and 10% fly ash amounts. For the 15%, 20% and 25% fly ash amounts on the other hand, the swell potentials are “low” to “medium” in Seed et al. (1962) and Van der Merwe (1964) approaches. Dakshanamurthy and Raman (1973) approach gives “very high” swell potential for all fly ash contents. These results show that after a preliminary guess with these charts site-specific oedometer swelling tests are needed for a definite description of swell potential.

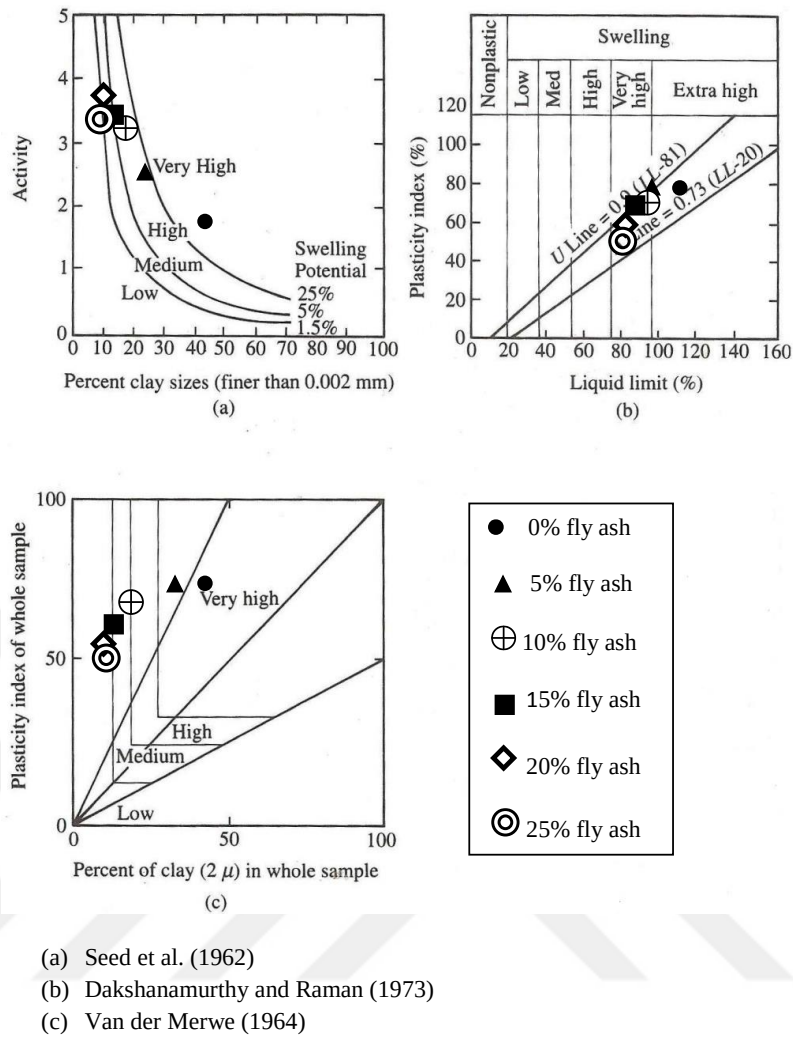


Figure 4.63 : Criteria for determining swell potential (the marks in the figures are for Bilkent Clay) (Das, 1999).

The swell percentages for each fly ash content found in laboratory experiments are given in Figure 4.64. The swell percentages decrease continuously as fly ash content increases. However, it is understood that the swelling amount is still quite high even for fly ash content of 25%.

The allowable swell percentage for transportation structures is generally accepted as 0.5% as given in Highway Standards (KGM, 2005) as mentioned before. However, the maximum fly ash content in this study (25%) is not adequate to obtain allowable swell percentages for transportation structures, as shown in Figure 4.64. There are reductions in swell percentages as the fly ash content increases but at the highest amount of fly ash (i.e. 25%) the swell percentage is still 4.53% which is inadequate. It can be stated here that fly ash improvement of a very highly plastic clay may not be possible within reasonable fly ash contents as far as the swell percentage criterion is concerned.

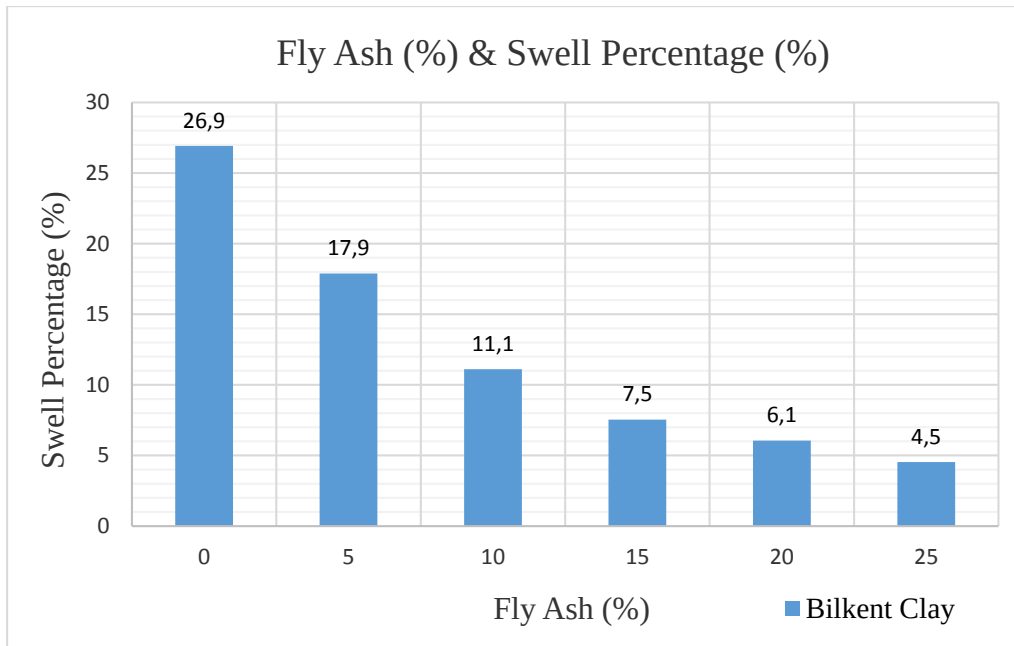


Figure 4.64 : Variation of swell percentage with fly ash content.

Figure 4.65 below shows the net swell percentages with increasing fly ash content. This chart has been prepared based on a theoretical calculation in which net swell percentages are calculated with proportional to natural clay swell percentage (i.e. natural clay is composed of 100% clay and its swell percentage has been measured as 26.9% in experiment, then with 5% fly ash, the clay content is 95%, thus the net swell percentage is calculated as 25.6%, the calculations are similar with other fly ash contents).

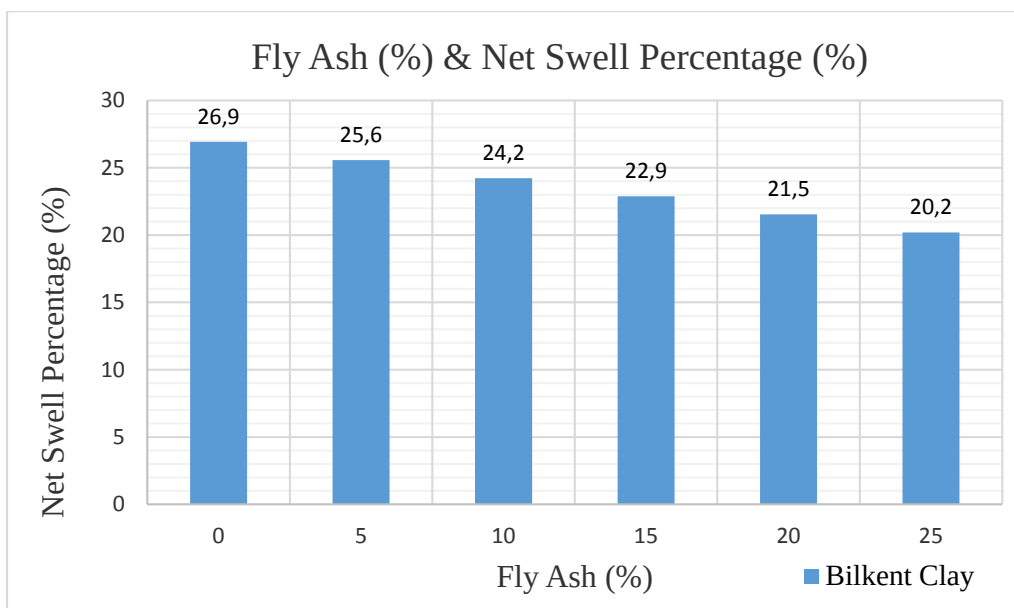


Figure 4.65: Variation of net swell percentage with fly ash content.

Figure 4.66 shows the effect of fly ash on swell percentage as fly ash content increases. This chart is simply the difference between the charts given in Figure 4.64 and 4.65. It should be noted here that the swell percentage decrease is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between fly ash and clay. Figure 4.66 shows only the latter effect. The effect of fly ash is maximum at early stages (i.e. 5% to 10% contents). The effect of fly ash on swell percentage, on the other hand, diminishes after 15% fly ash content as observed in Figure 4.66.

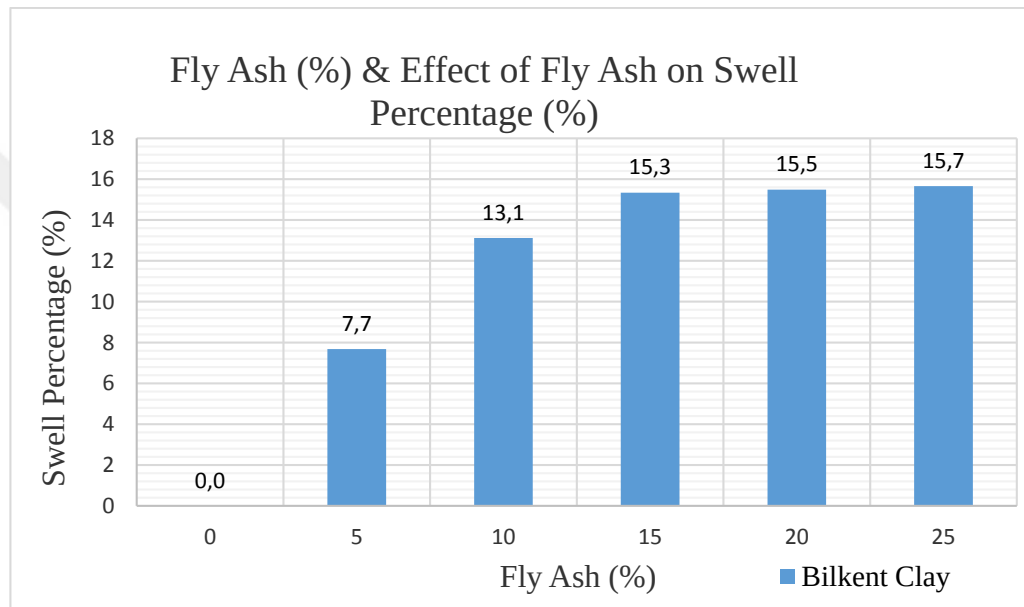


Figure 4.66 : Effect of fly ash content on swell percentage.

It is observed that the swell percentages found in laboratory experiments are still “high” even for high contents of fly ash (i.e. 4.5% swelling as given in Figure 4.64). Thus, it can be concluded that the fly ash agent may not be sufficient alone for swelling design criteria in such type of very highly plastic clays.

Figure 4.67 gives the variation of swell pressure with fly ash content found in laboratory. As understood, swell pressure remains almost same for 5% fly ash amount. Then, swell pressure is decreased quite significantly from 5% to 10% fly ash content (about 72%). These decreases are moderate in later fly ash amounts.

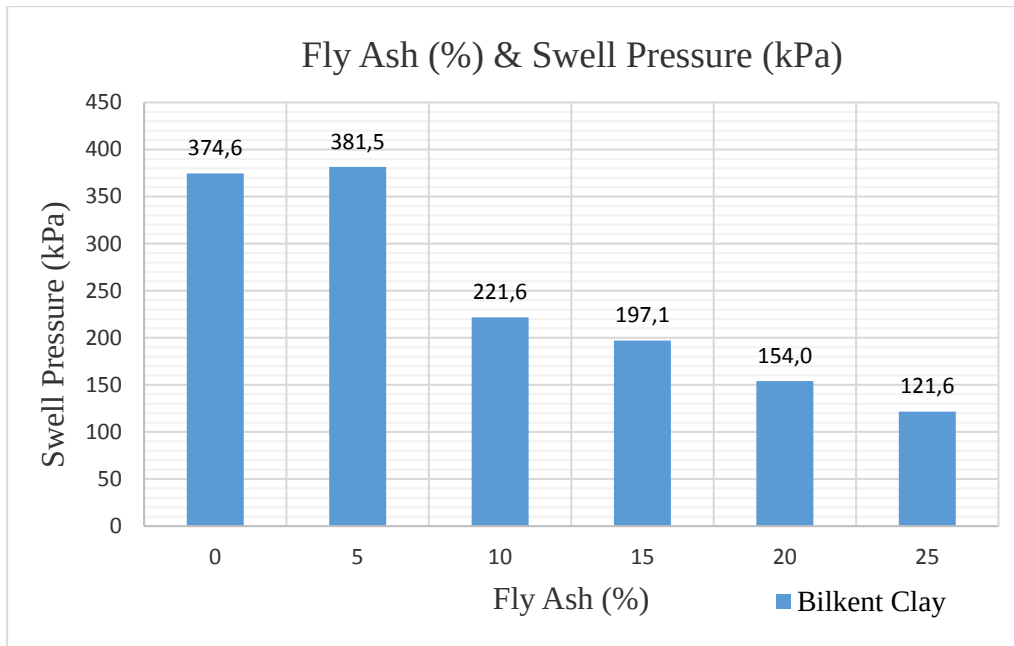


Figure 4.67 : Variation of swell pressure with fly ash content.

Figure 4.68 and Figure 4.69 show the net swell pressure and the effect of fly ash on swell pressure with increasing fly ash content respectively. The explanations are similar to ones given in swell percentage subject above. Therefore, the effect of fly ash on swell pressure is maximum in fly as content increase from 5% to 10%. This is compatible with the result observed in swell percentage subject.

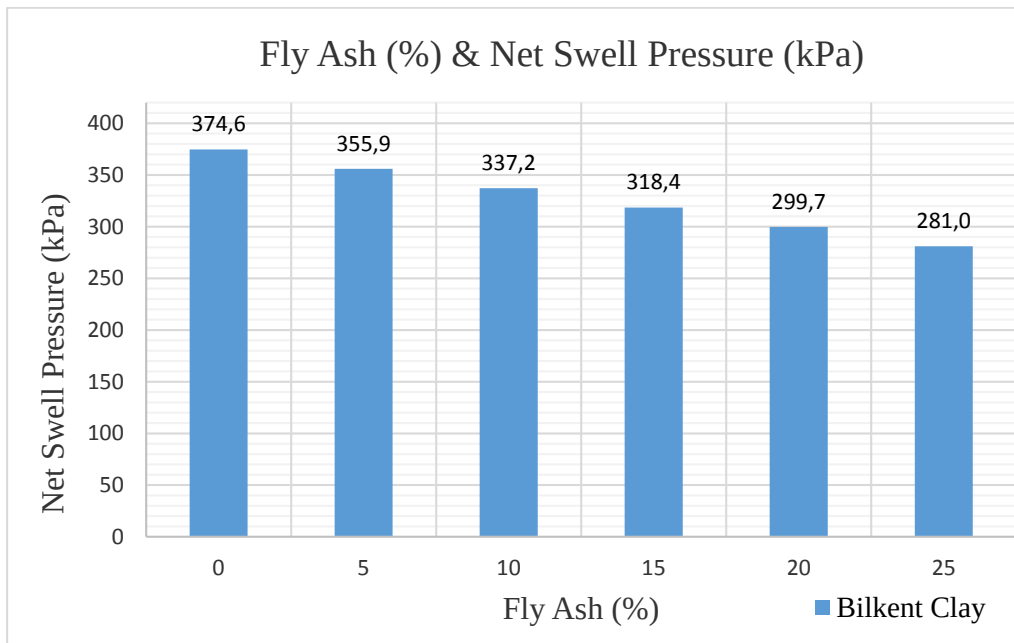


Figure 4.68 : Variation of net swell pressure with fly ash content.

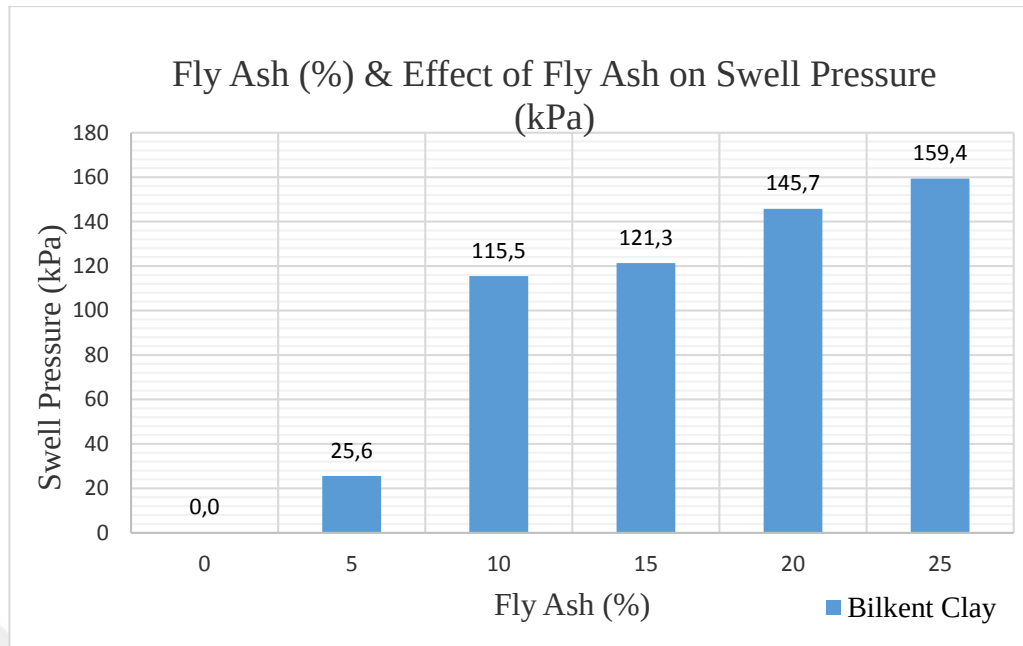


Figure 4.69 : Effect of fly ash content on swell pressure.

4.4.2.3 Effects of stabilization on strength parameters

Figure 4.70 gives the variation of unconfined compressive strengths with fly ash content for non-soaked and soaked conditions. The unconfined compressive strengths are always greater in non-soaked condition as expected. The strength first decreases by adding 5% fly ash and then increases steadily up to 25% fly ash as seen in Figure 4.70 for non-soaked condition. The same behavior is also valid in soaked condition. It is inferred from the results that the non-soaked strengths are at least two times more than soaked strengths. This result can be attributed to the lack of pozzolanic reactions which are quite effective in lime stabilization.

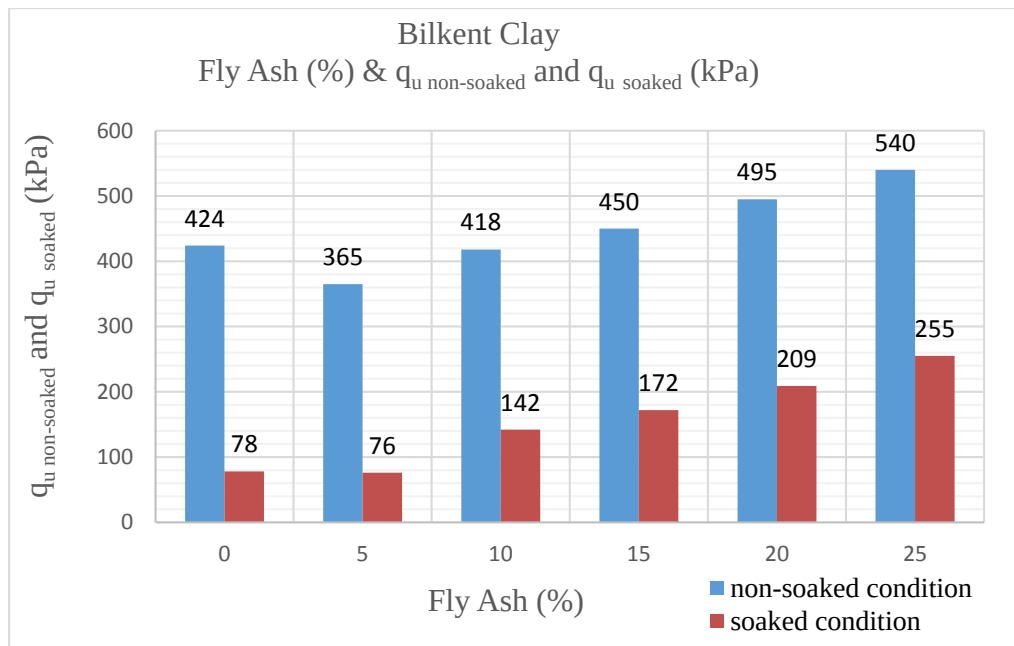


Figure 4.70 : Variation of unconfined compressive strength of non-soaked ($q_{u \text{ non-soaked}}$) and soaked ($q_{u \text{ soaked}}$) samples with fly ash content.

4.4.2.4 Effects of stabilization on compression parameters

In this part, effect of fly ash stabilization on compression parameters for Bilkent Clay is explained. Using the results of odometer tests, void ratio (e) versus effective stress (σ') graphs are prepared first. Void ratio (e) versus $\log(\sigma')$ graphs can be drawn then so that the graphs can be approximated to straight lines. Both compression and expansion part of the test can be monitored to get compression and expansion indices from the slopes of the straight lines. C_c is the slope of the virgin compression line of the test. High value of C_c means high compressibility. The expansion index (C_e) is the slope of the unloading part of the test plot. In the same way, the higher the C_e value, the higher the expansion capacity of the clay. The compressibility and expansion potential of highly plastic clays cause problems in foundation engineering applications as stated before. Fly ash stabilization effect in this context is to be evaluated below for Bilkent Clay.

e - σ' and e - $\log \sigma'$ graphs are given in Figure 4.71 and Figure 4.72 respectively. The variations of compression index (C_c) and expansion index (C_e) values with fly ash content are given in Figure 4.73 and Figure 4.74 respectively. It is noticed that the clay sample is quite compressible in natural state. Both compression and expansion (recompression) curves are steep in natural state of the soil. The effect of fly ash on the compression index values is completely unstable as understood in Figure 4.73. The

expansion index values, on the contrary, decrease steadily up to 20% fly ash content (Figure 4.74).

As a result, the effect of fly ash on the compressibility characteristics of both Esenboğa and Bilkent Clays which are extremely and very highly plastic is unstable and can be accepted as ineffective.



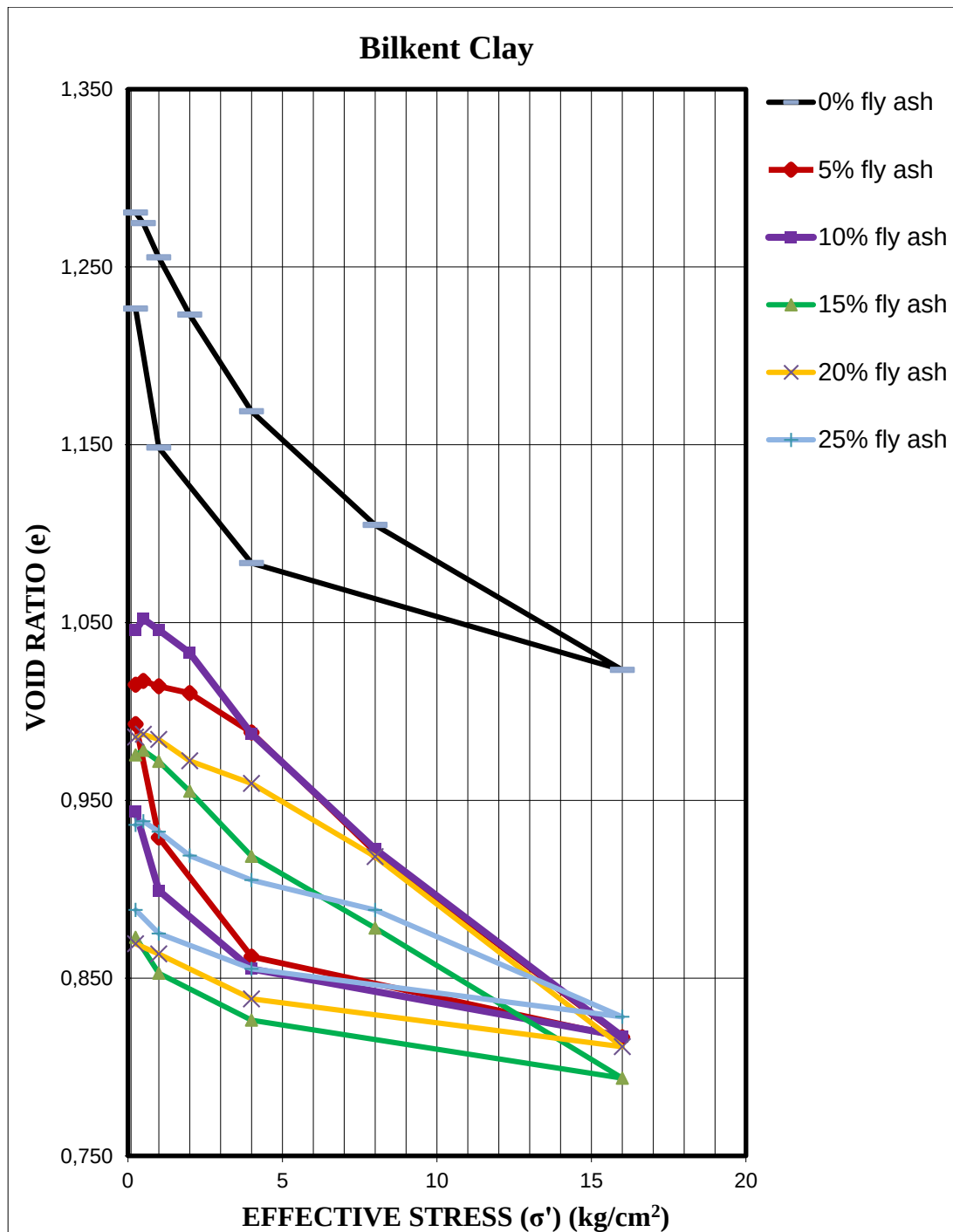


Figure 4.71 : e - σ' relationship for natural and fly ash-treated soil samples for Bilkent Clay.

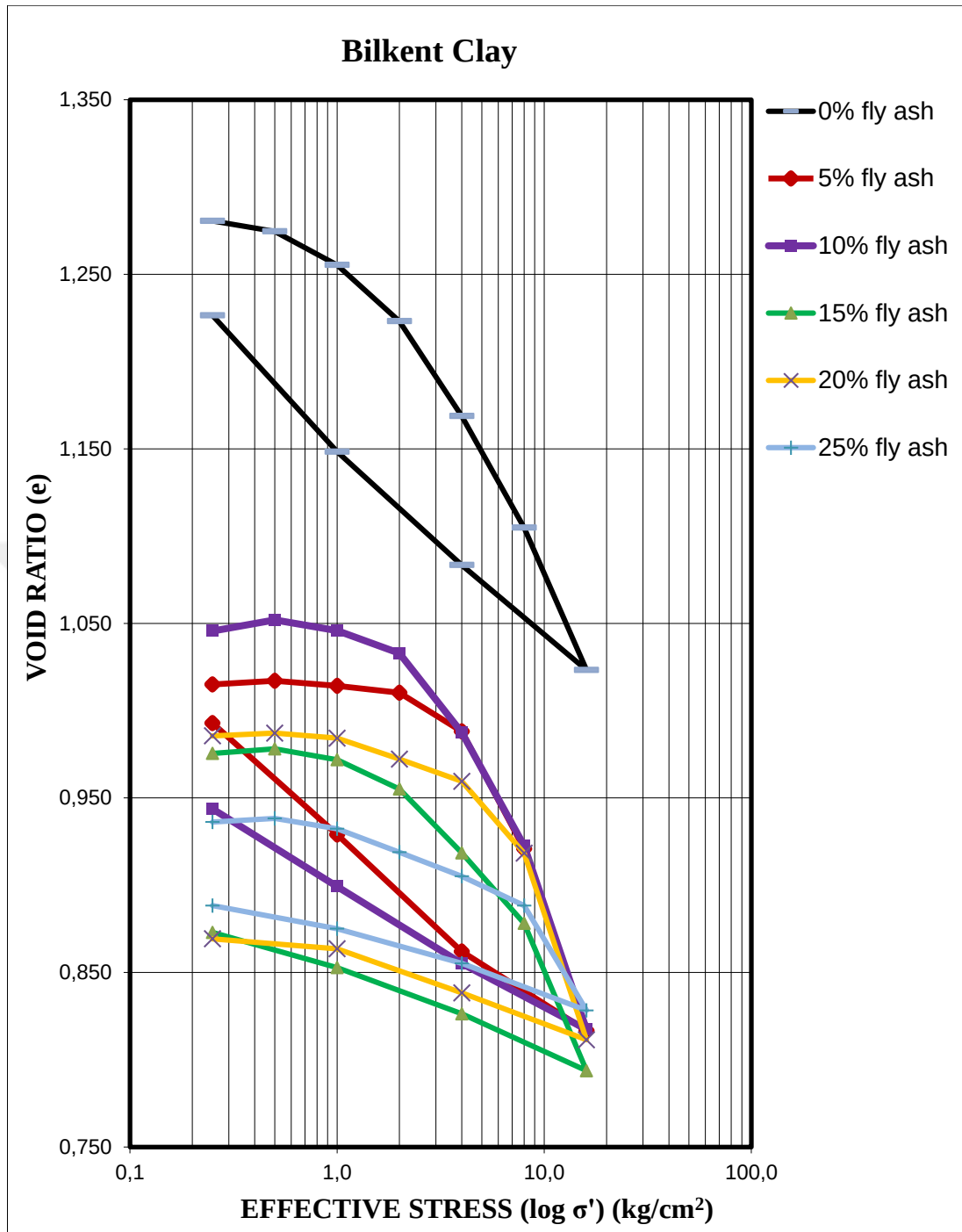


Figure 4.72 : e - $\log \sigma'$ relationship for natural and fly ash-treated soil samples for Bilkent Clay.

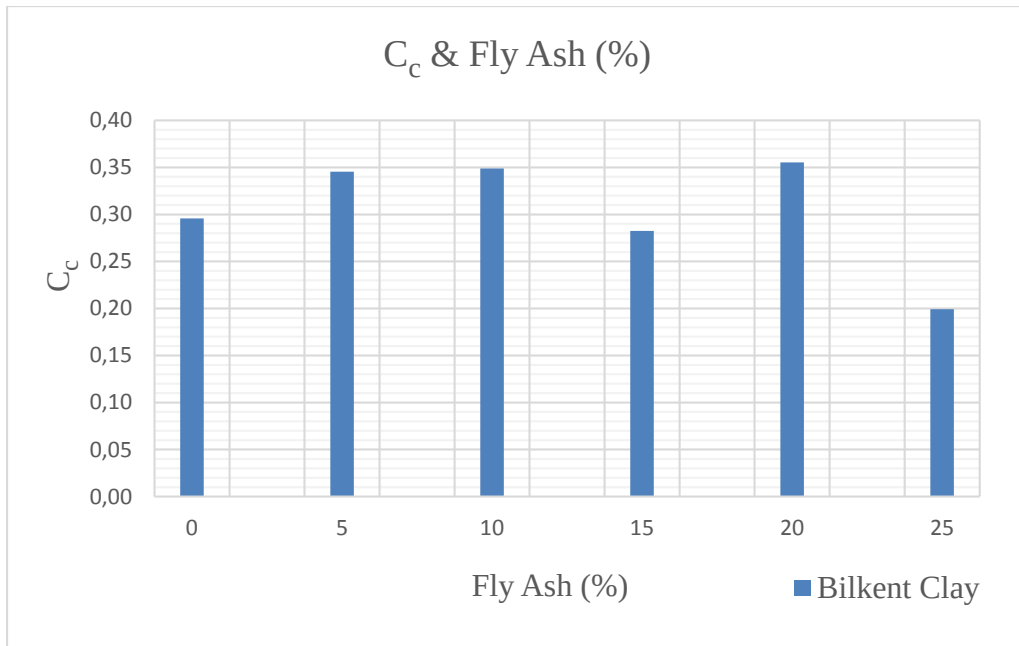


Figure 4.73 : Compression index (C_c) values with increasing fly ash content for Bilkent Clay.

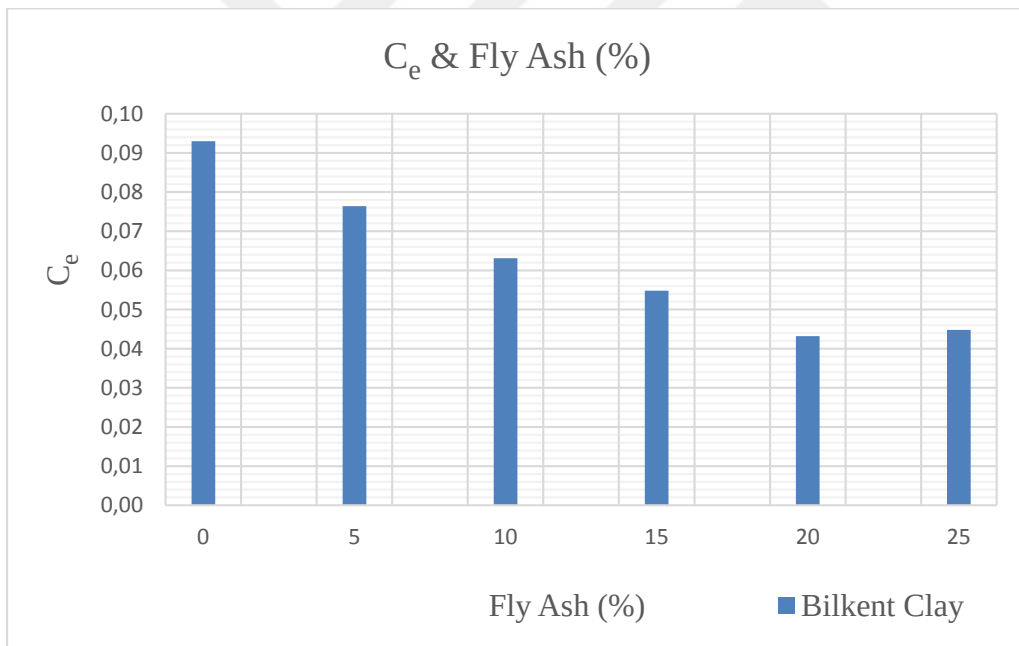


Figure 4.74 : Expansion index (C_e) values with increasing fly ash content for Bilkent Clay.

4.5 Resilient Modulus Tests

Resilient modulus tests for both Esenboğa and Bilkent clays with only lime agent were performed without cure and with 7, 28, 56 and 90 days cure periods for each lime percentages (1%, 3%, 5%, 7% and 9%). Resilient modulus tests were also conducted on natural clays.

4.5.1 Esenboğa Clay

4.5.1.1 Effects of time on lime-treated Esenboğa Clay

Mean resilient modulus values with increasing lime content are given in Figure 4.75 for Esenboğa Clay without cure. There is a substantial increase in mean modulus value when the lime content is increased from 1% to 3%.

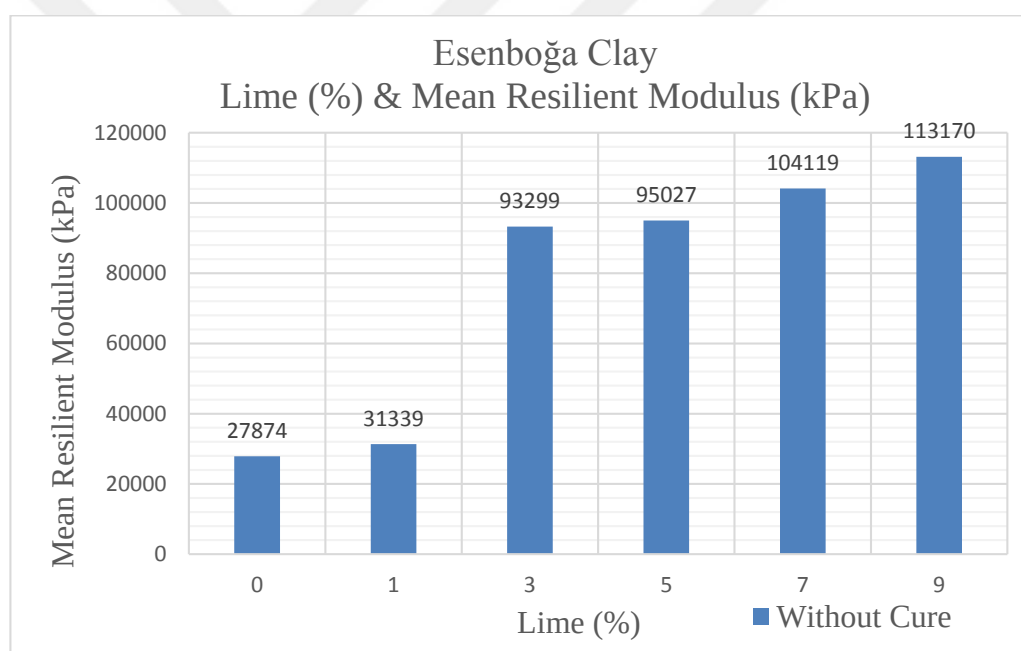


Figure 4.75 : Mean resilient modulus (M_r) values with lime content without cure for Esenboğa Clay.

When the same relationship is examined for 7-days cure period (Figure 4.76), it is seen that there is a stable increase in mean resilient modulus value as lime content increases. The increases at 3% and 9% lime contents, on the other hand, are somewhat more remarkable.

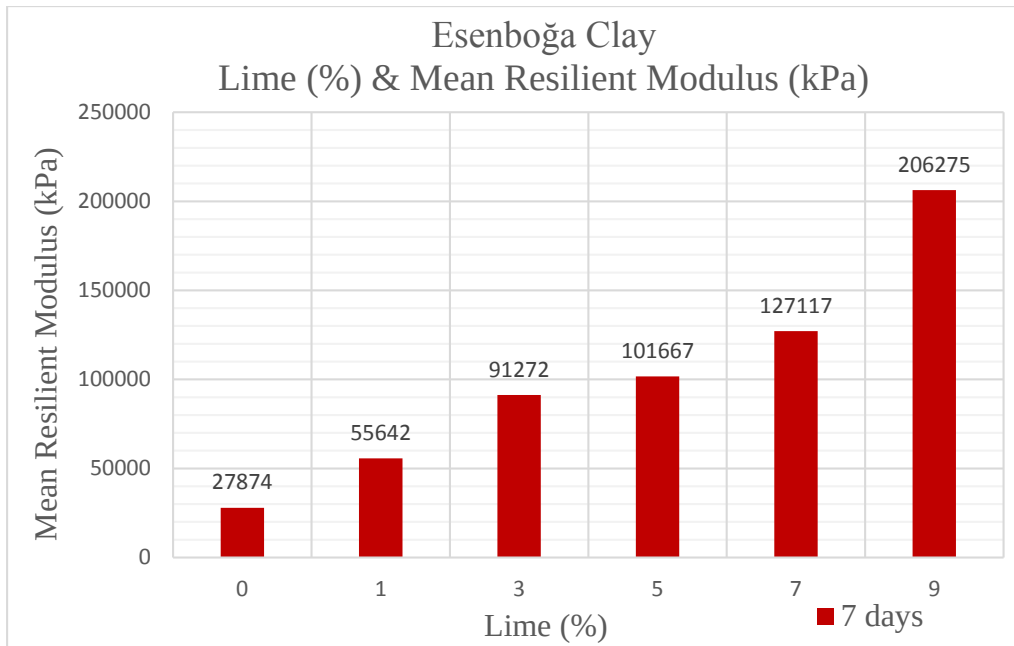


Figure 4.76 : Mean resilient modulus (M_r) values with lime content for 7 days cure period for Esenboğa Clay.

The above evaluations are also valid for 28-days, 56-days and 90-days cure periods as noticed in Figure 4.77, Figure 4.78 and Figure 4.79, respectively below.

Therefore, 3% lime content as a general can be accepted as an effective stage when the mean resilient modulus increases are taken into account.

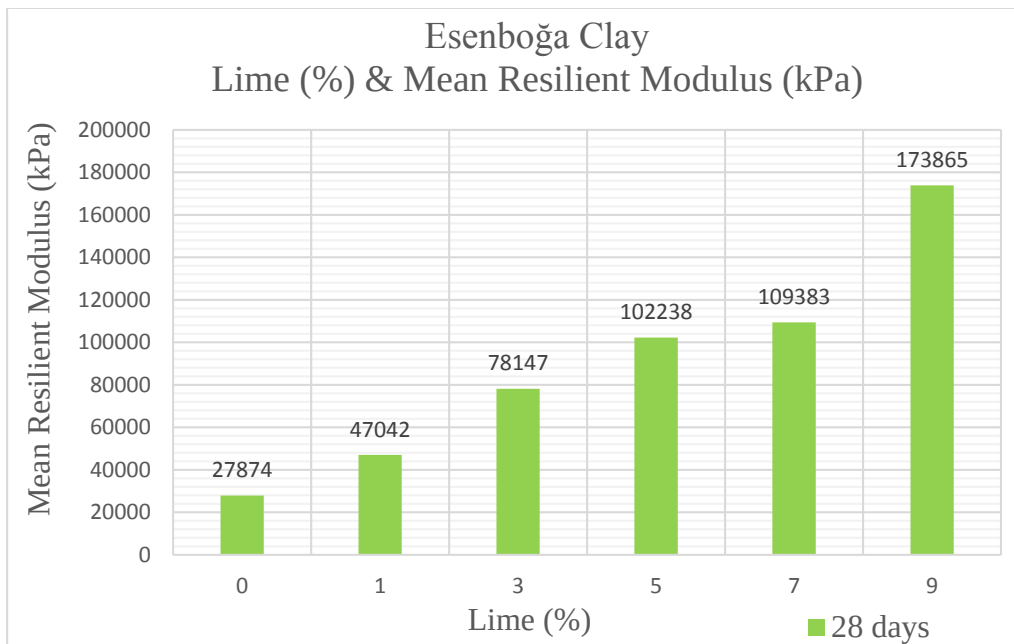


Figure 4.77 : Mean resilient modulus (M_r) values with lime content for 28 days cure period for Esenboğa Clay.

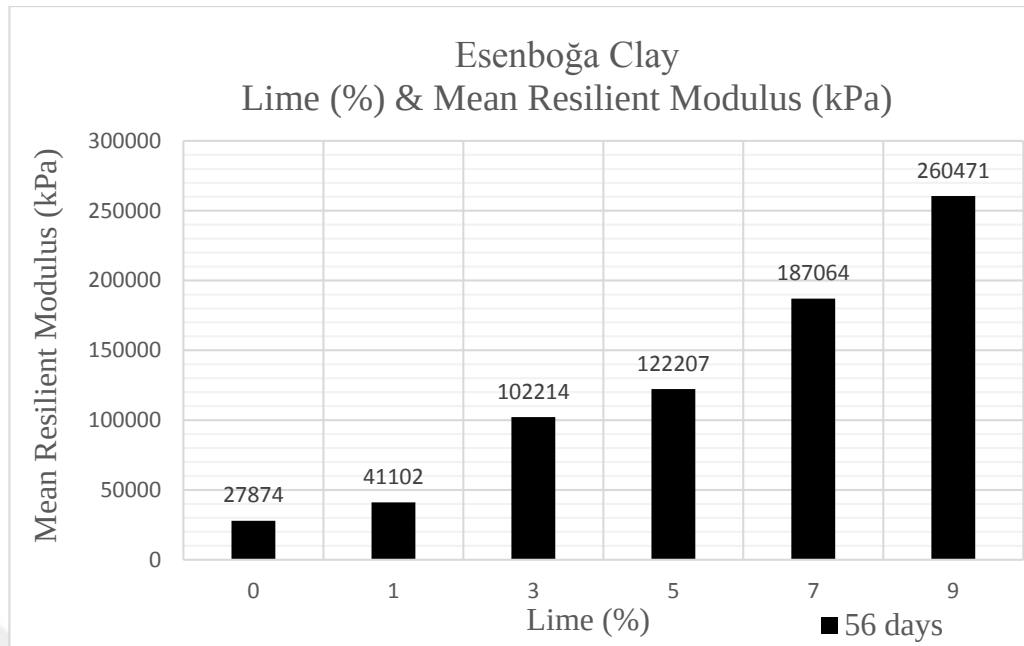


Figure 4.78 : Mean resilient modulus (M_r) values with lime content for 56 days cure period for Esenboğa Clay.

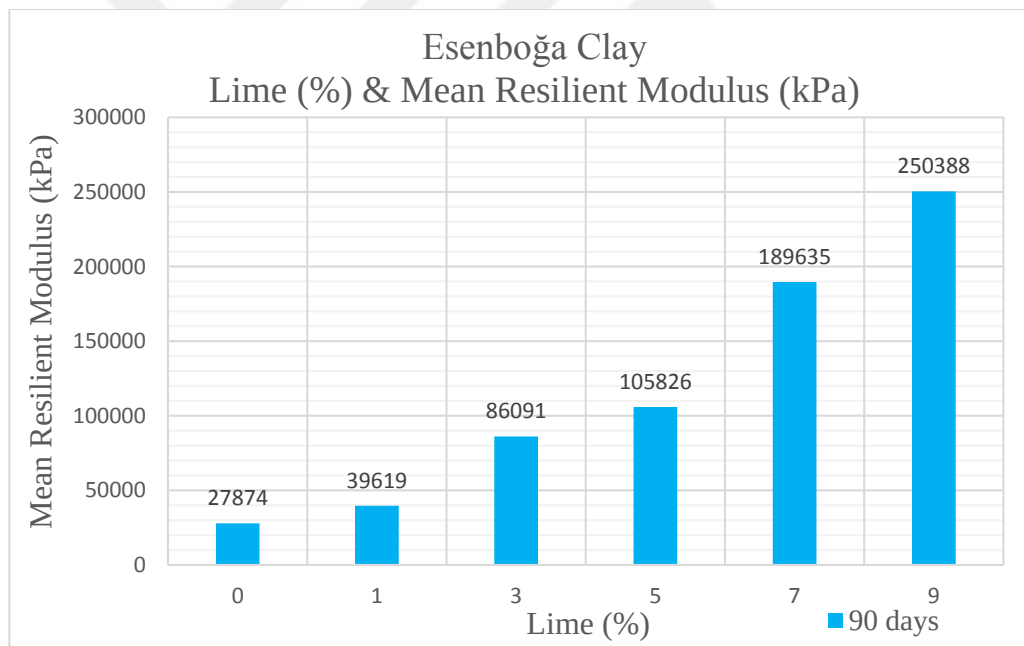


Figure 4.79 : Mean resilient modulus (M_r) values with lime content for 90 days cure period for Esenboğa Clay.

Mean resilient modulus values are given for natural clay (without cure) and for all other cure periods together with increasing lime content in Figure 4.80. The mean resilient modulus values are not stable within a lime content as cure period increases as noticed. But, it is worthy of note that the mean modulus values are always maximum

at 56-days cure period starting from 3% lime content. Therefore, 90-days cure period can be accepted ineffective when the resilient modulus values are taken into account.

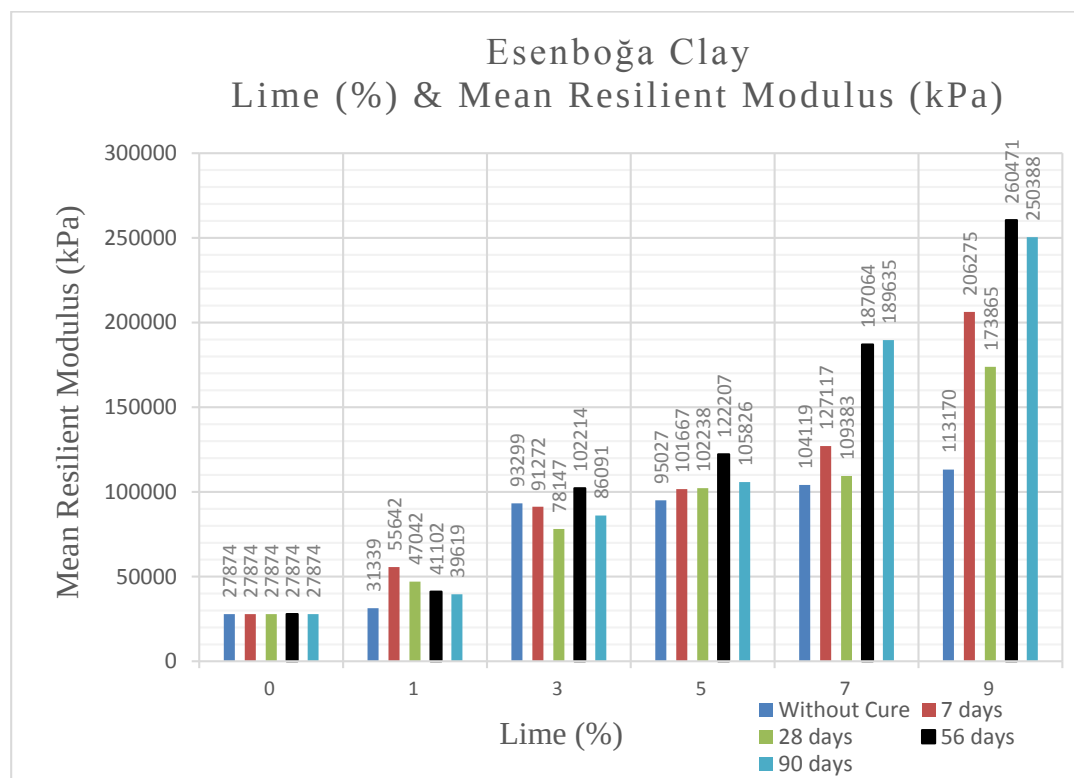


Figure 4.80 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.

As explained in Chapter 2, the minimum value of M_r for the subgrade layer in highways, according to Turkish Highway Standard's minimum CBR value of 8, is 66.000 kPa (9500 psi) according to Christopher et al.'s (2006) correlation (See Figure 2.16). Otherwise, soil improvement is needed. It is observed in Figure 4.80 that the above condition is satisfied at 3% lime content in all cured samples and without cure condition. In Standard, it is stated that there is no need for subbase design if the M_r value of the subbase material exceeds 138.000 kPa (20.000 psi). The M_r values at about 5% to 6% lime content for 56 days curing period even reach these values (Figure 4.80).

4.5.2 Bilkent Clay

4.5.2.1 Effects of time on lime-treated Bilkent Clay

Mean resilient modulus values with increasing lime content are given in Figure 4.81 for Esenboğa Clay without cure. The mean modulus values increase almost quadratically with increasing lime content. At the beginning, the increase rate in the

mean resilient modulus is between 75-80% (i.e. between 0%, 1% and 3% lime contents). Later, these increase rates are about 50%, 40% and 30% in the other lime contents of 5%, 7% and 9% respectively.

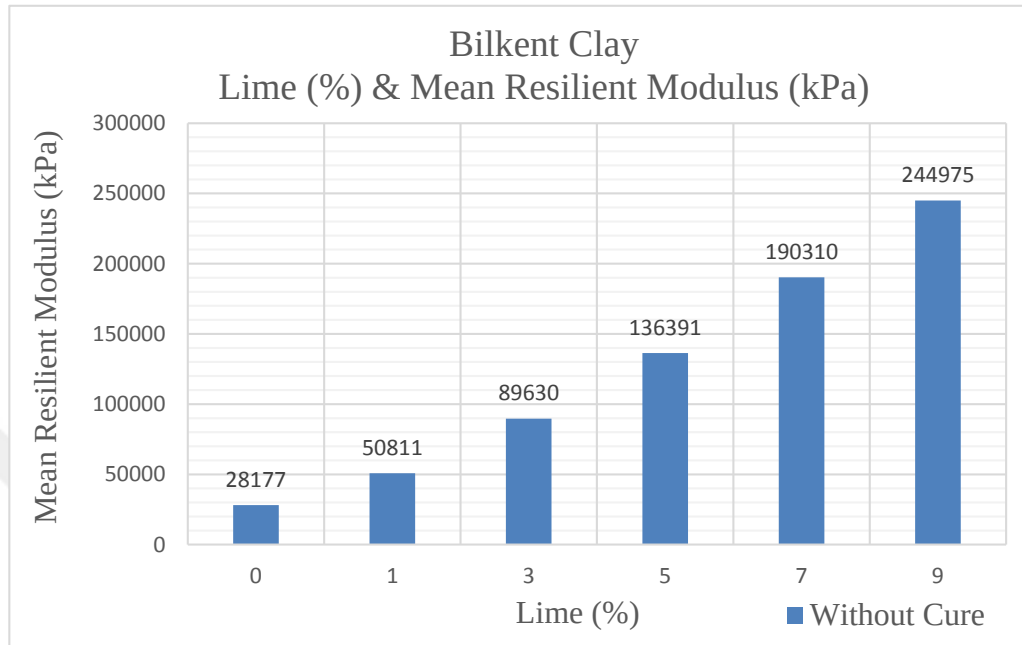


Figure 4.81 : Mean resilient modulus (M_r) values with lime content without cure for Bilkent Clay.

When the same relationship is examined for 7-days cure period (Figure 4.82), it is seen that there is a stable increase in mean resilient modulus value as lime content increases. But, the increase in the mean resilient modulus between the 1% and 3% lime contents is quite substantial.

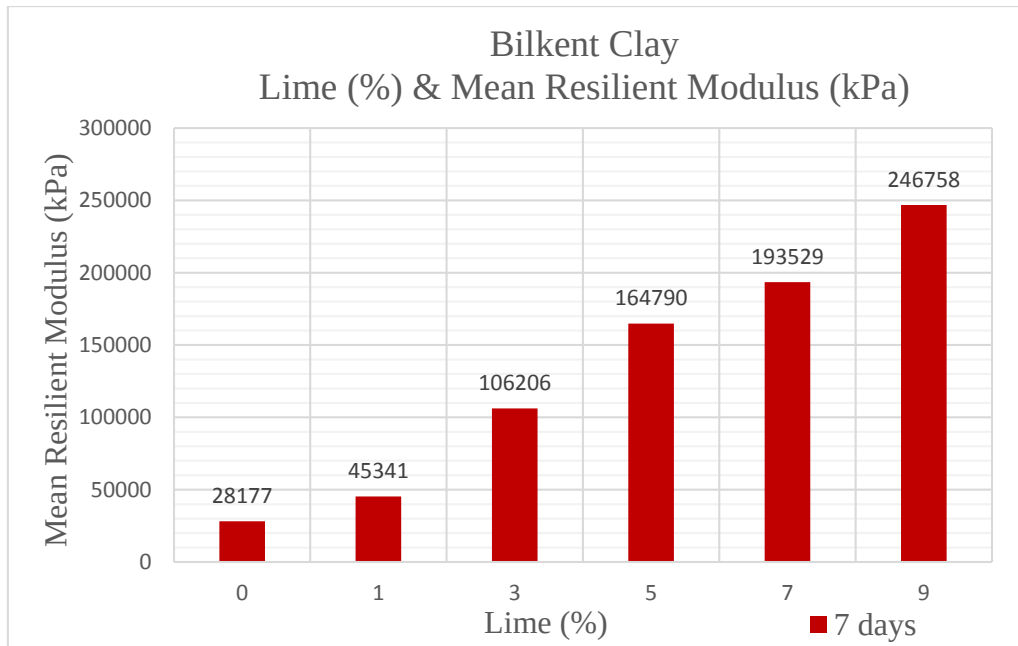


Figure 4.82 : Mean resilient modulus (M_r) values with lime content for 7 days cure period for Bilkent Clay.

When the same graphs are examined for the other cure periods (i.e. 28, 56 and 90 days), it can be concluded that the mean resilient modulus values increase erratically with increasing lime content (Figure 4.83, Figure 4.84 and Figure 4.85). However, the increases between the 5% and 7% lime contents are quite satisfactory. Therefore 7% lime content can be considered as an effective stage for Bilkent Clay.

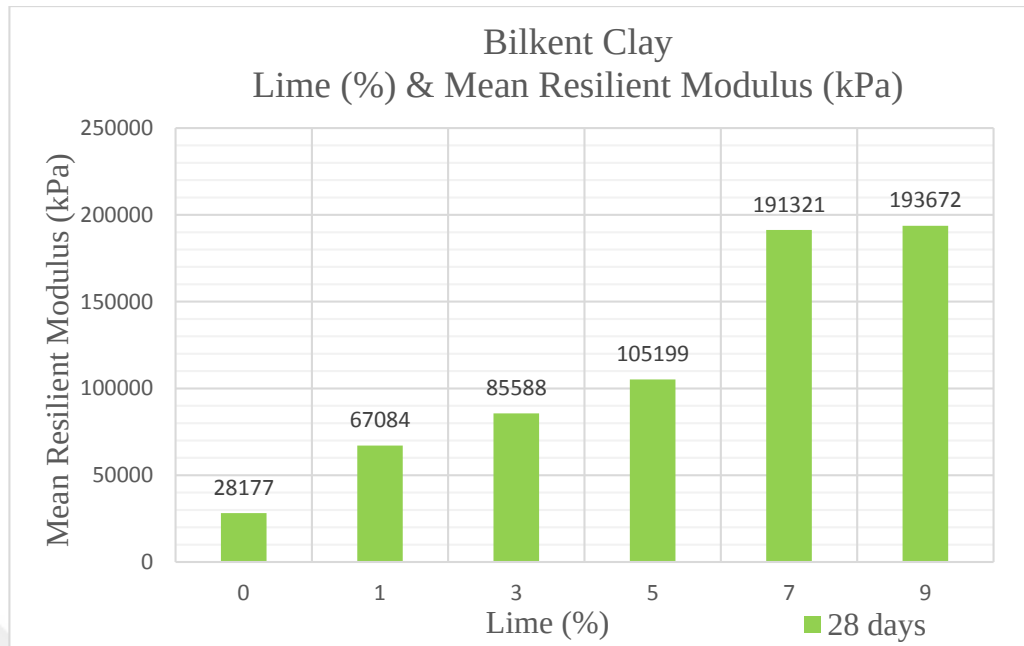


Figure 4.83 : Mean resilient modulus (M_r) values with lime content for 28 days cure period for Bilkent Clay.

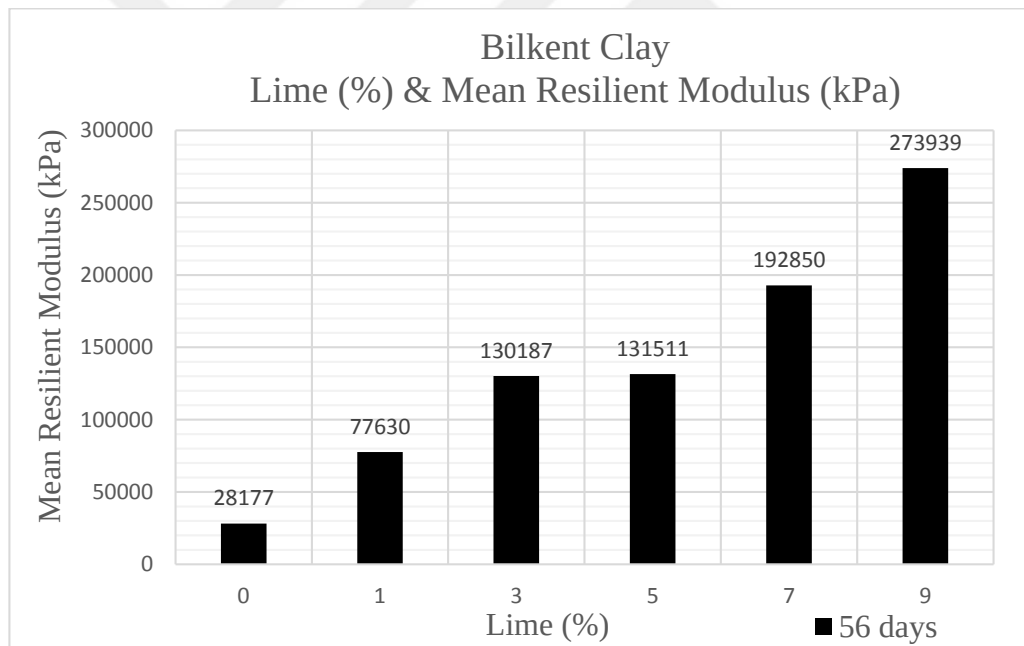


Figure 4.84 : Mean resilient modulus (M_r) values with lime content for 56 days cure period for Bilkent Clay.

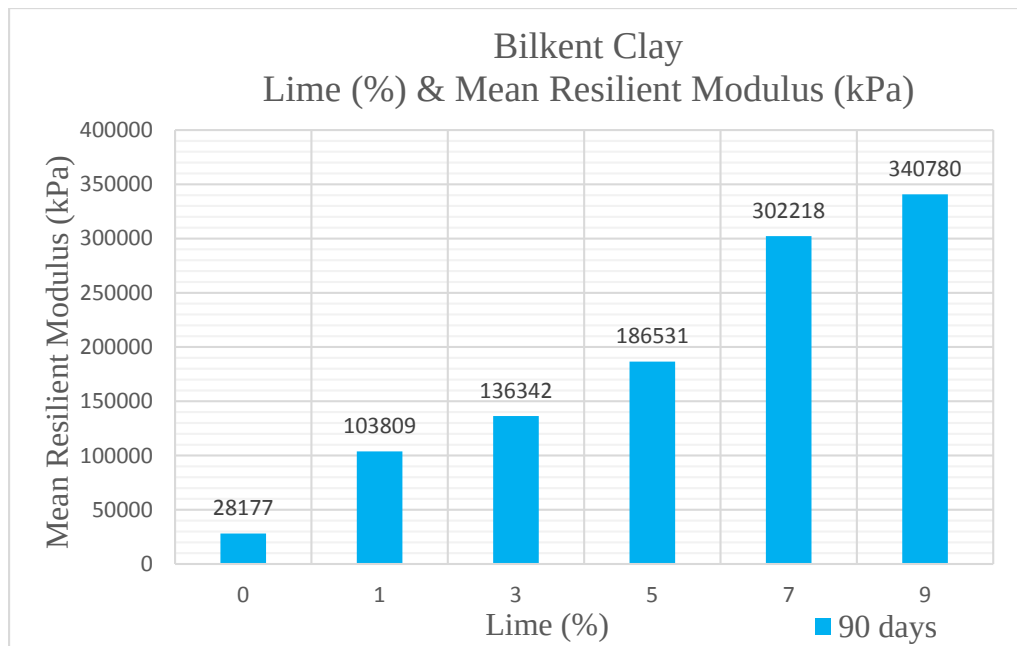


Figure 4.85 : Mean resilient modulus (M_r) values with lime content for 90 days cure period for Bilkent Clay.

Mean resilient modulus values are given for natural clay (without cure) and for all other cure periods together with increasing lime content in Figure 4.86. The mean resilient modulus values are not stable within a lime content as cure period increases as in Esenboğa case. It should be pointed out, on the other hand, that 90 days-cure period gives the highest mean resilient modulus value in all lime contents.

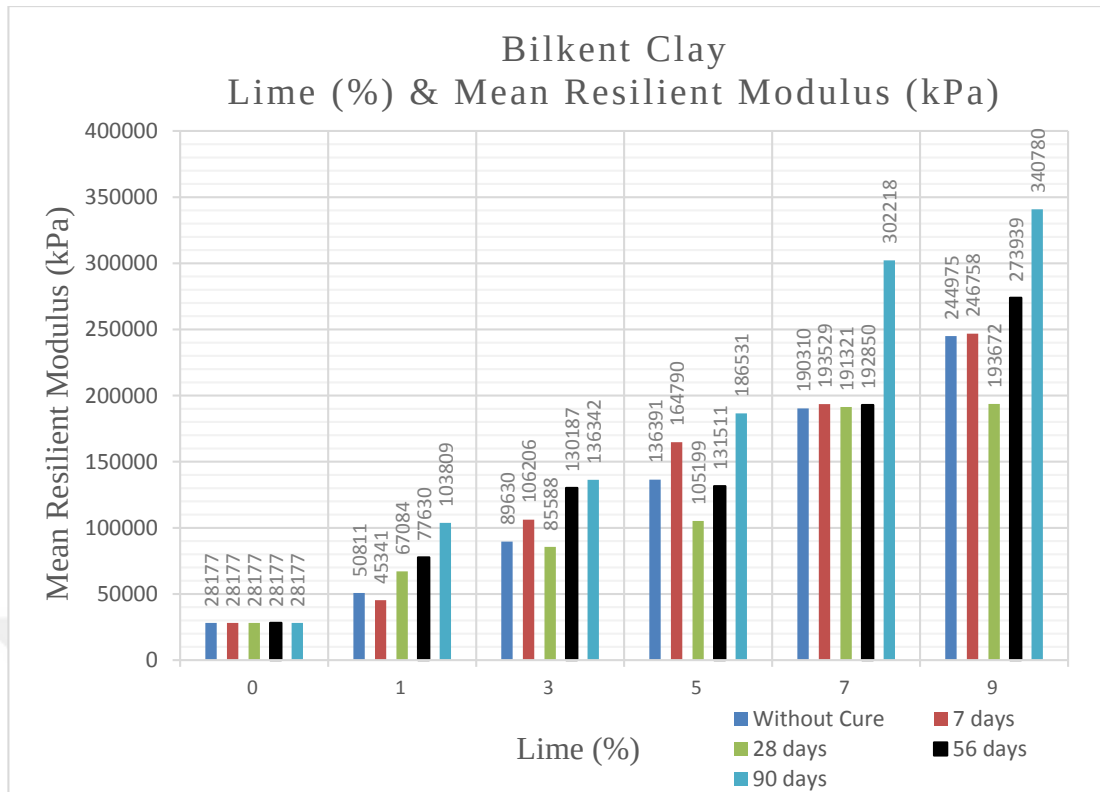


Figure 4.86 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Bilkent Clay.

The similar results of Esenboğa Clay case for the criteria of resilient modulus (M_r) value stated in Turkish Highway Standards are valid for Bilkent Clay also. 3% lime content is enough for the minimum $M_r=66.000$ kPa which is equivalent to minimum CBR=8% as observed in Figure 4.86 (for CBR<8%, soil improvement is necessary). Moreover, the M_r values exceed 138.000 kPa (20.000 psi) in all curing times and also without cure condition at about 6% lime content (20.000 psi limit is for subbase material so that there is no need for subbase). All these results prove the efficiency of lime agent in the extremely highly plastic soils like Esenboğa and Bilkent Clays.

4.5.3 Effect of stress state on resilient modulus values

The effect of stress state can be revealed by observing the relationship between resilient modulus values and deviator stresses used in the tests. Figure 4.87 shows this relationship for natural Esenboğa Clay with the confining stresses of 14 kPa, 28 kPa and 42 kPa used in the tests. It is noticed that there is generally a tendency of softening behaviour in terms of resilient modulus as the deviator stress increases for untreated soil.

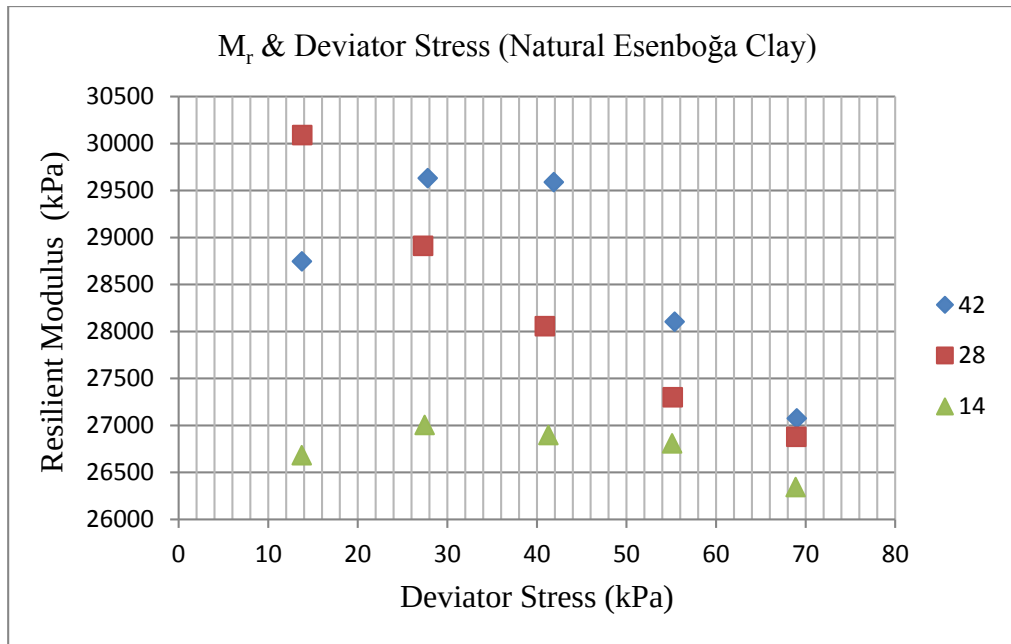


Figure 4.87 : Resilient modulus (M_r) versus deviator stress with 14 kPa, 28 kPa and 42 kPa confining stresses (Natural Esenboğa Clay).

The state of stress effect on resilient modulus values for lime-stabilized Esenboğa Clay (7% lime content) can be seen in Figure 4.88. It is understood that the softening behaviour observed in natural clay is not valid in this case. The resilient modulus values can be accepted not changing at the beginning. It is observed that the resilient modulus values increase generally for relatively higher deviator stresses (Figure 4.88).

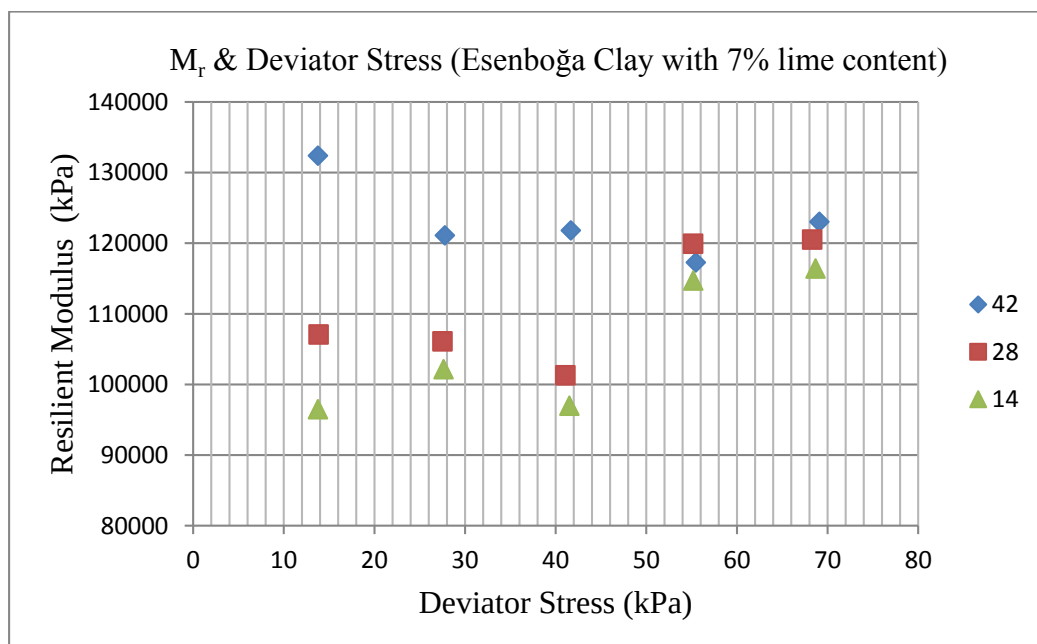


Figure 4.88 : Resilient modulus (M_r) versus deviator stress with 14 kPa, 28 kPa and 42 kPa confining stresses (Esenboğa Clay with 7% lime content)

4.6 Methylene Blue Test (MBT)

Methylene blue tests have been performed on untreated and lime-treated Esenboğa and Bilkent Clays for each percentage in order to determine the activity of the clay and also to have an idea about the swell potential, index properties and specific surface area.

4.6.1 MBT for Esenboğa Clay

Lime content versus methylene blue value (MBV) relationship is given in Figure 4.89. The variation of activity with lime content is given in Figure 4.90. It has been observed that the activity values decrease generally with the increase in lime content (Figure 4.90). As a result of decrease in activity with lime content, the surface area of clay particles decreases due to flocculation. Therefore, MBV decreases as expected (Figure 4.89). The decreases in MBV are remarkable between 3% to 7%. The same behaviour can be accepted valid for the relationship of lime content versus activity value as given in Figure 4.90. In other words, 7% lime content is quite effective for the decrease of both MBV and activity.

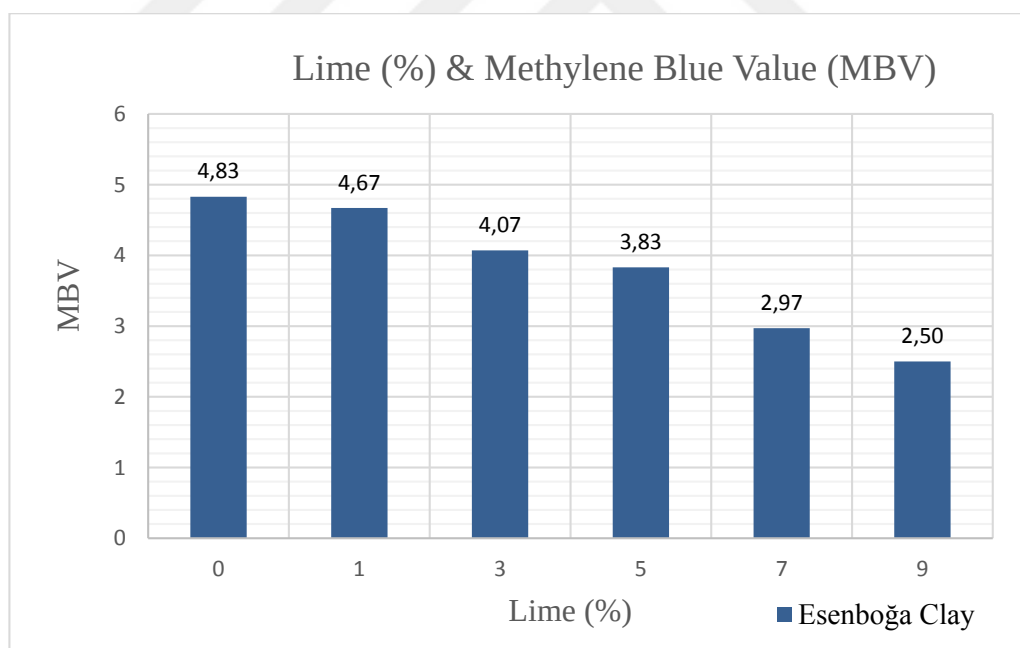


Figure 4.89 : Variation of methylene blue value (MBV) with lime content for Esenboğa Clay.

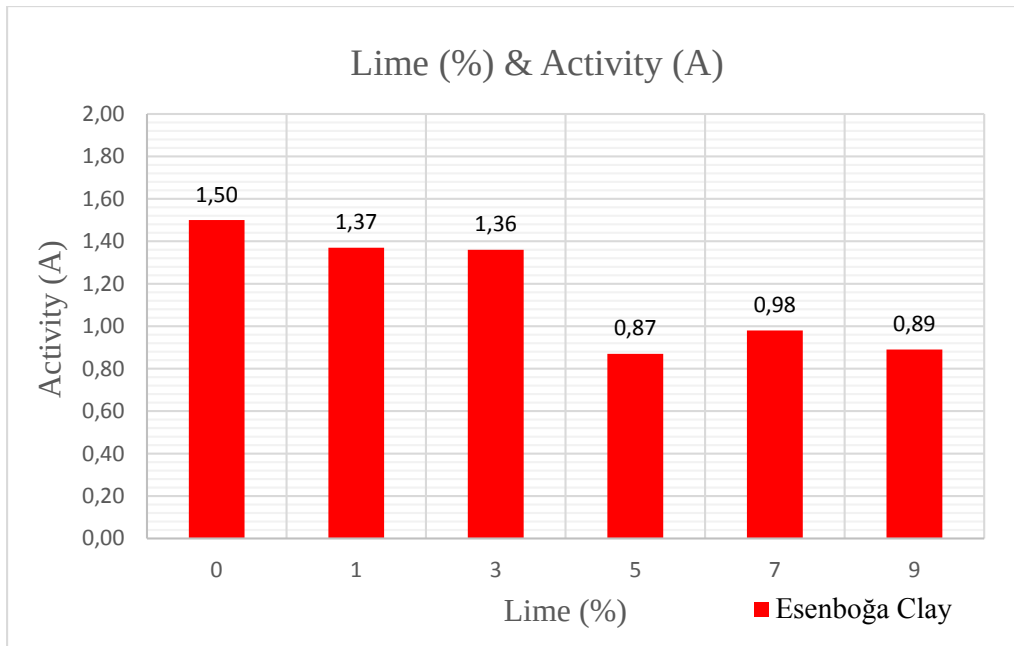


Figure 4.90 : Variation of activity (A) with lime content for Esenboğa Clay.

4.6.2 MBT for Bilkent Clay

The relationship of methylene blue value (MBV) and lime content is given in Figure 4.91. There is a stable decrease in MBV with lime content as understood from this figure. It is observed that MBV decreases to quite low ranges at 3% to 7% lime contents (i.e. MBV is about 0.65 at these ranges of lime content).

The activity values are given in Figure 4.92 with lime content. The rate of decrease in activity for Bilkent Clay is mild as compared with the MBV decrease. The activity value is still 1.51 at 5% lime content. It is seen that the activity value decreases to 1.20, which may be a value for an inactive soil, at 9% lime content.

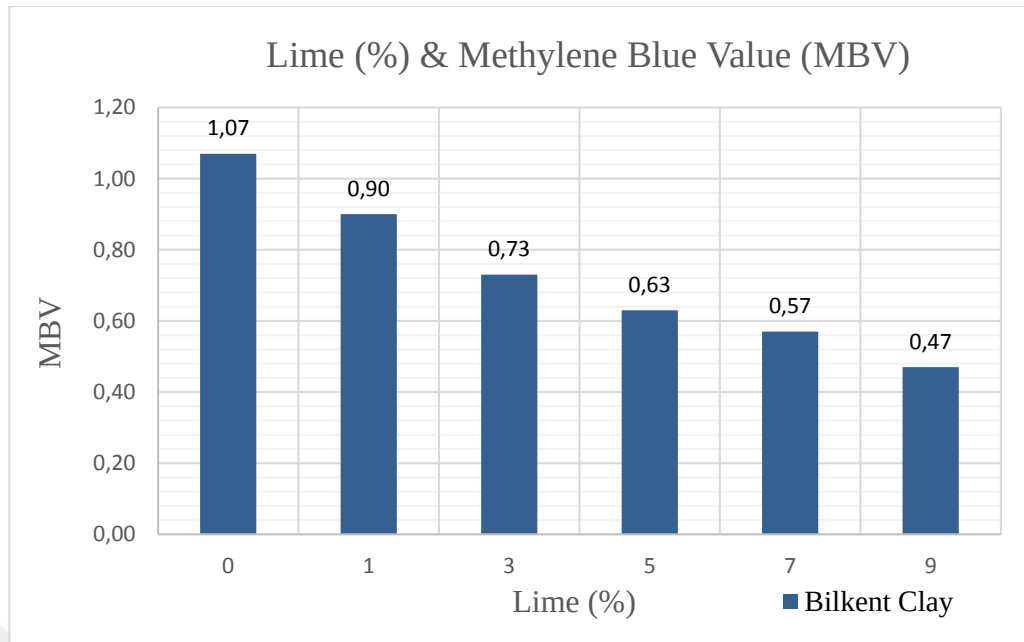


Figure 4.91 : Variation of methylene blue value (MBV) with lime content for Bilkent Clay.

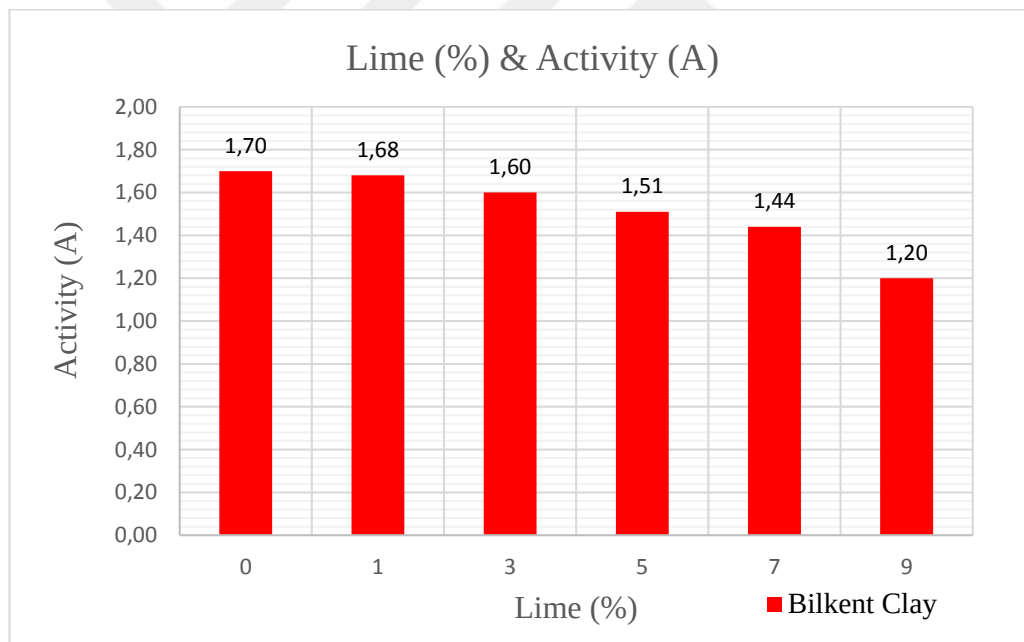


Figure 4.92 : Variation of activity (A) with lime content for Bilkent Clay.

4.7 Tests Conducted after Resilient Modulus Test on RM Test Specimens

4.7.1 Unconfined compression strength test

The unconfined compressive strengths (q_u) of the specimens after resilient modulus test have been determined. The diameter and the height of resilient modulus samples were 10 cm and 20 cm respectively. The same size specimens have also been used for

the determination of unconfined compressive strengths (q_u) in non-soaked condition for all cure periods. The results for each clay are as follows:

4.7.1.1 Esenboğa Clay

The q_u values for Esenboğa Clay are given in the following figures. Figure 4.93 gives the strengths of the samples without cure. It is noticed that 7% lime content is the optimum stage. The q_u value decreases at 9% lime content.

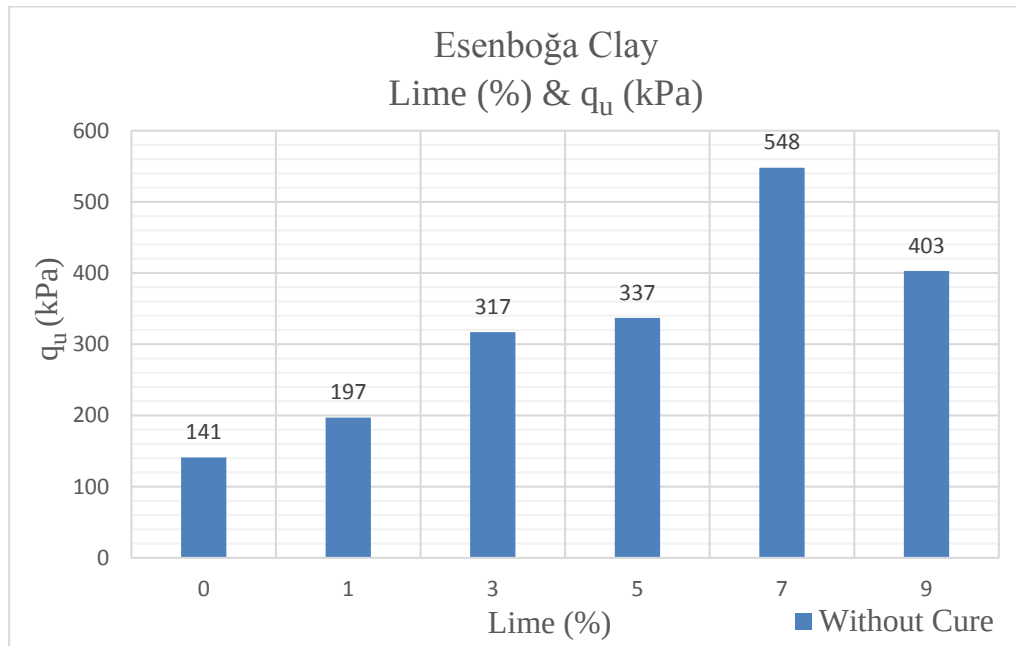


Figure 4.93 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content without cure for Esenboğa Clay.

The q_u values of the samples which have different cure periods (i.e. 7, 28, 56 and 90 days) are given in Figure 4.94 to Figure 4.97. As understood from these figures, there are sharp increases in strength values just starting from 1% lime contents. There is also an optimum stage, which is about 5% to 7%, for 7-days cure period. The strengths increase with lime content in a stable manner for the other cure periods. The strength values, on the other hand, can be said to increase satisfactorily at about 3% lime content. Namely, the q_u values at 3% lime content for the cure periods of 28, 56 and 90 days are about 3 to 3.7 times bigger than the strength values of untreated samples.

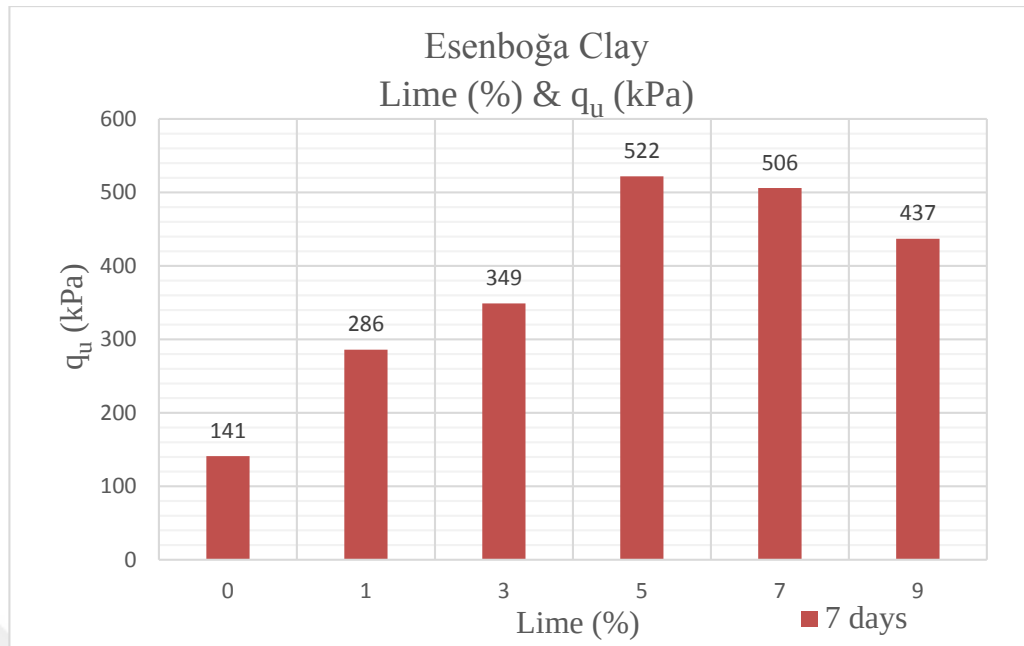


Figure 4.94 : Variation of unconfined compressive strength (q_u non-soaked) after resilient modulus test with lime content for 7 days cure period for Esenboğa Clay.

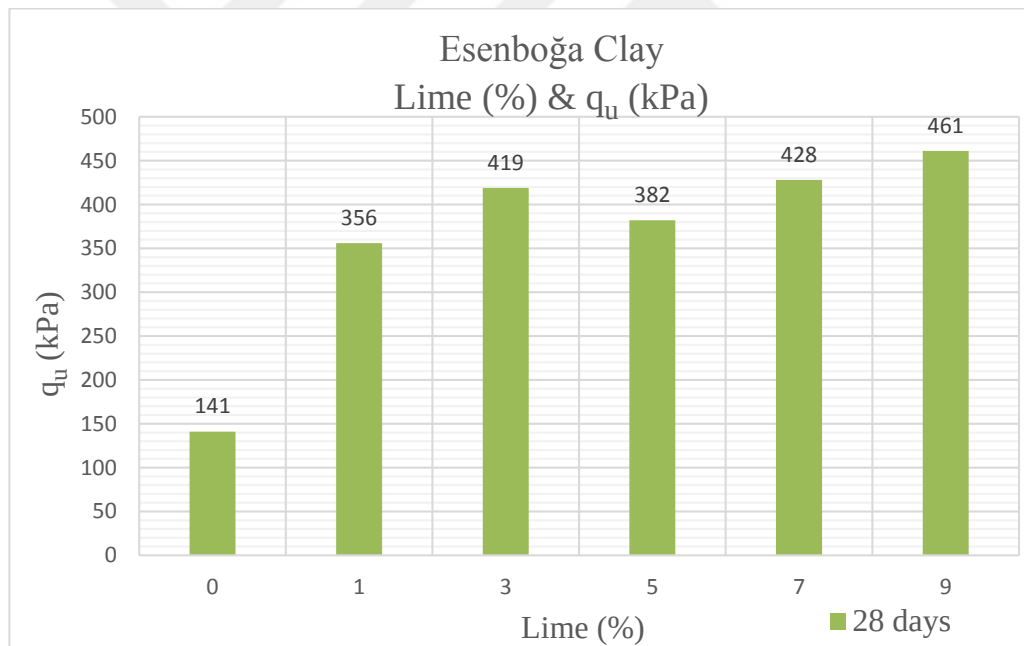


Figure 4.95 : Variation of unconfined compressive strength (q_u non-soaked) after resilient modulus test with lime content for 28 days cure period for Esenboğa Clay.

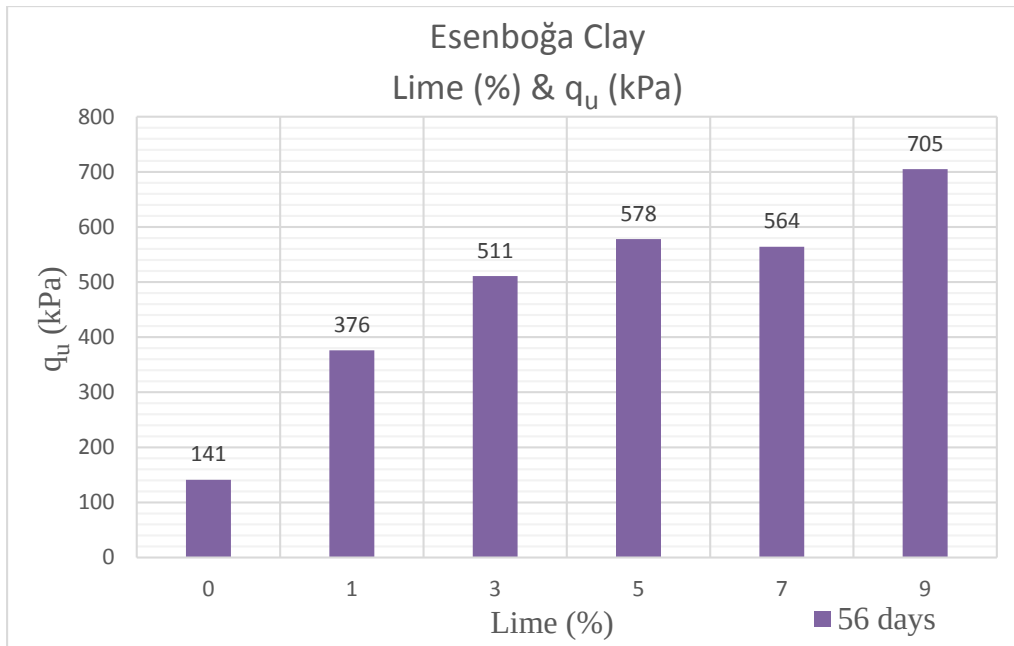


Figure 4.96 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 56 days cure period for Esenboğa Clay.

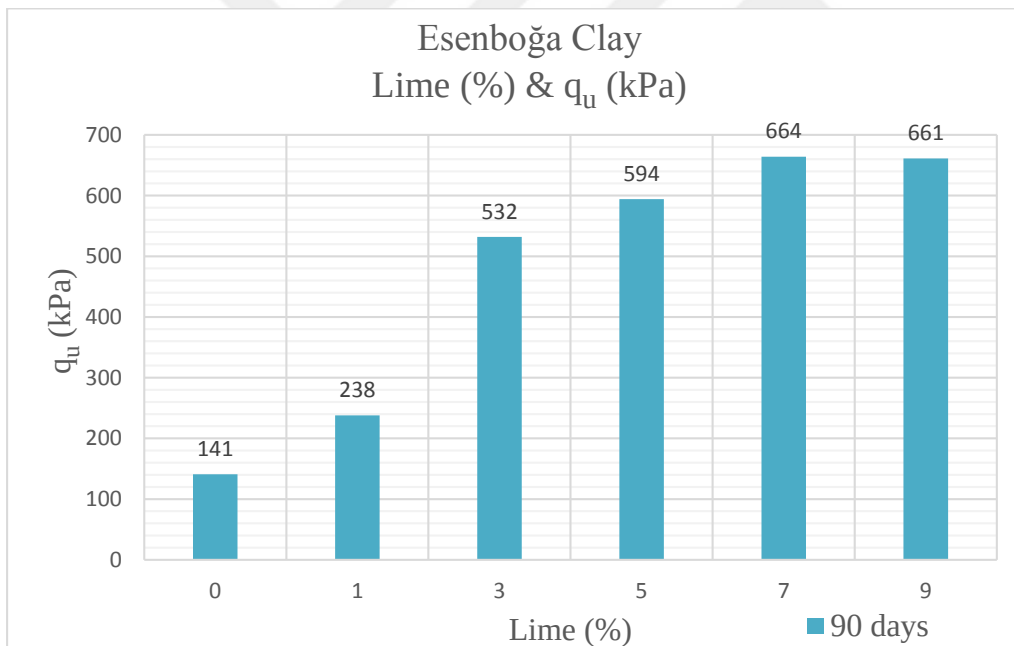


Figure 4.97 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 90 days cure period for Esenboğa Clay.

All results of Esenboğa Clay are given in the same figure below (Figure 4.98). The effect of curing can be observed in this figure with some exceptions.

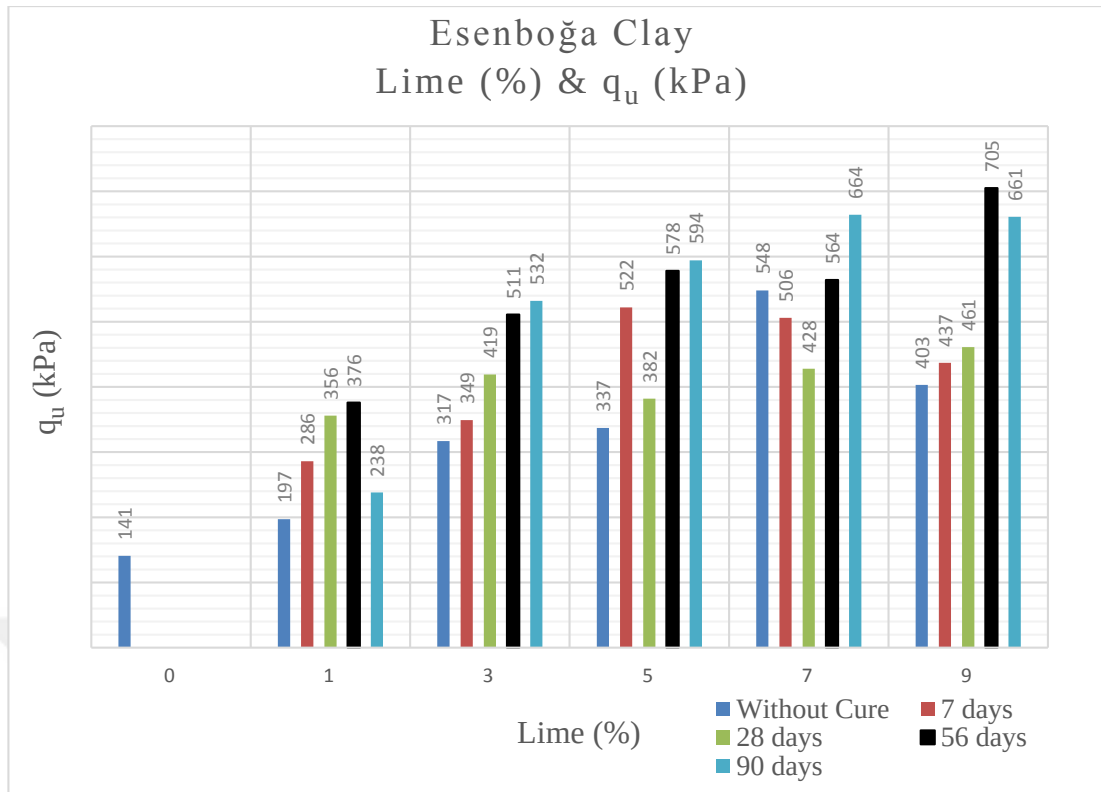


Figure 4.98 : Variation of unconfined compressive strength (q_u non-soaked) after resilient modulus test with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.

4.7.1.2 Bilkent Clay

The q_u values for Bilkent Clay without cure are given Figure 4.99 below. It is noticed that 7% lime content is the optimum stage as in Esenboğa case. The q_u value decreases at 9% lime content even less than the value of 5% lime content.

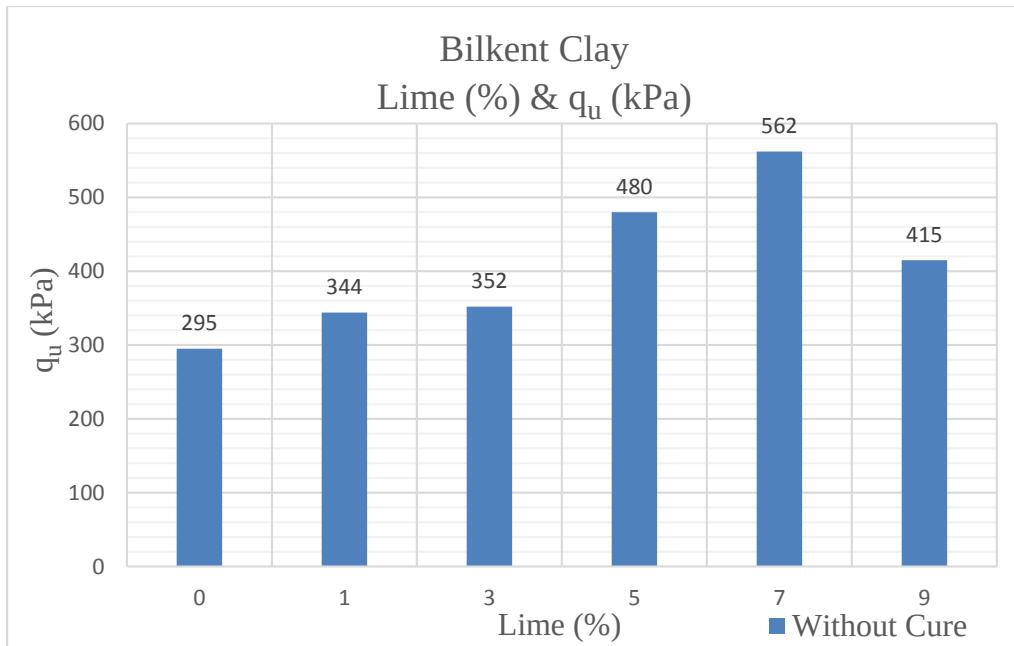


Figure 4.99 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content without cure for Bilkent Clay.

The q_u values of the samples which have different cure periods (i.e. 7, 28, 56 and 90 days) are given in Figure 4.100 to Figure 4.103. As seen in these figures, there are mild and stable increases in strength values with lime content in Bilkent Clay. The increases at 7% lime content are more remarkable (sometimes 5%). However, the results are not as satisfactory as in the Esenboğa case. That is to say, the q_u values at 7% lime content for the cure periods of 7, 28, 56 and 90 days are about 3 times bigger than the strength values of untreated samples. Similar result was about at 3% lime content in Esenboğa case.

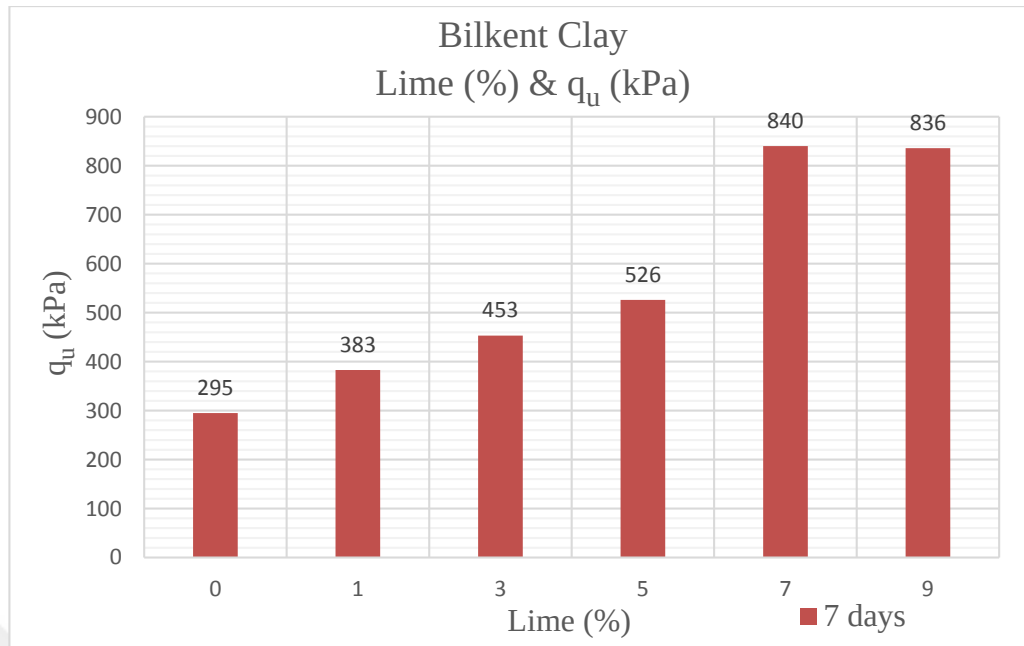


Figure 4.100 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 7 days period cure for Bilkent Clay.

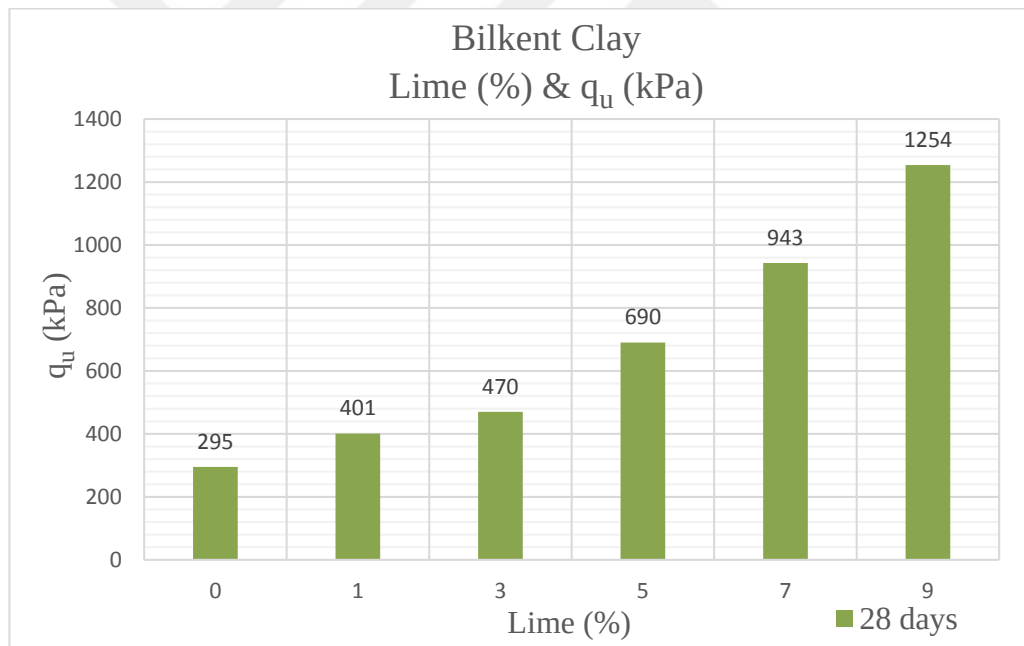


Figure 4.101 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 28 days cure period for Bilkent Clay.

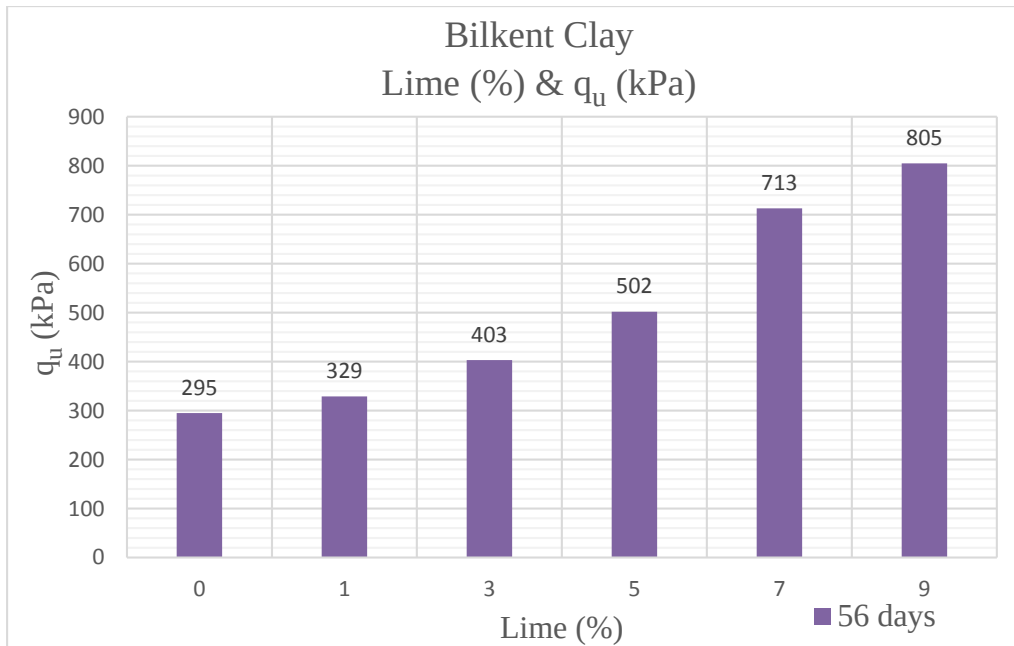


Figure 4.102 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 56 days cure period for Bilkent Clay.

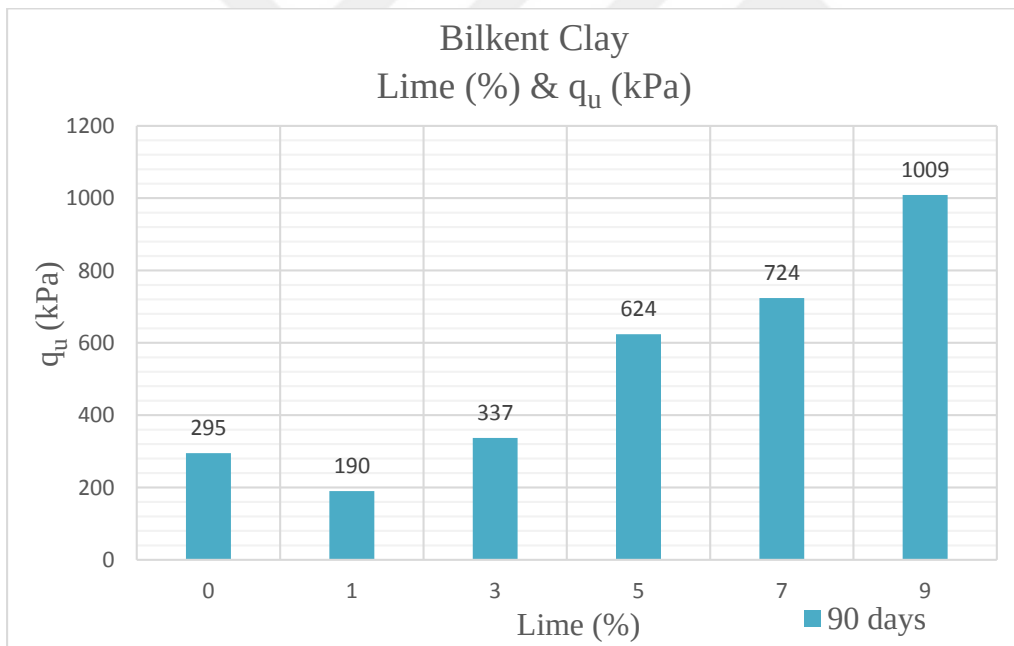


Figure 4.103 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 90 days cure period for Esenboğa Clay.

The above strength values for the Bilkent Clay are all given in Figure 4.104 below. It is noted that 28-days cure period gives the maximum strength in all lime contents.

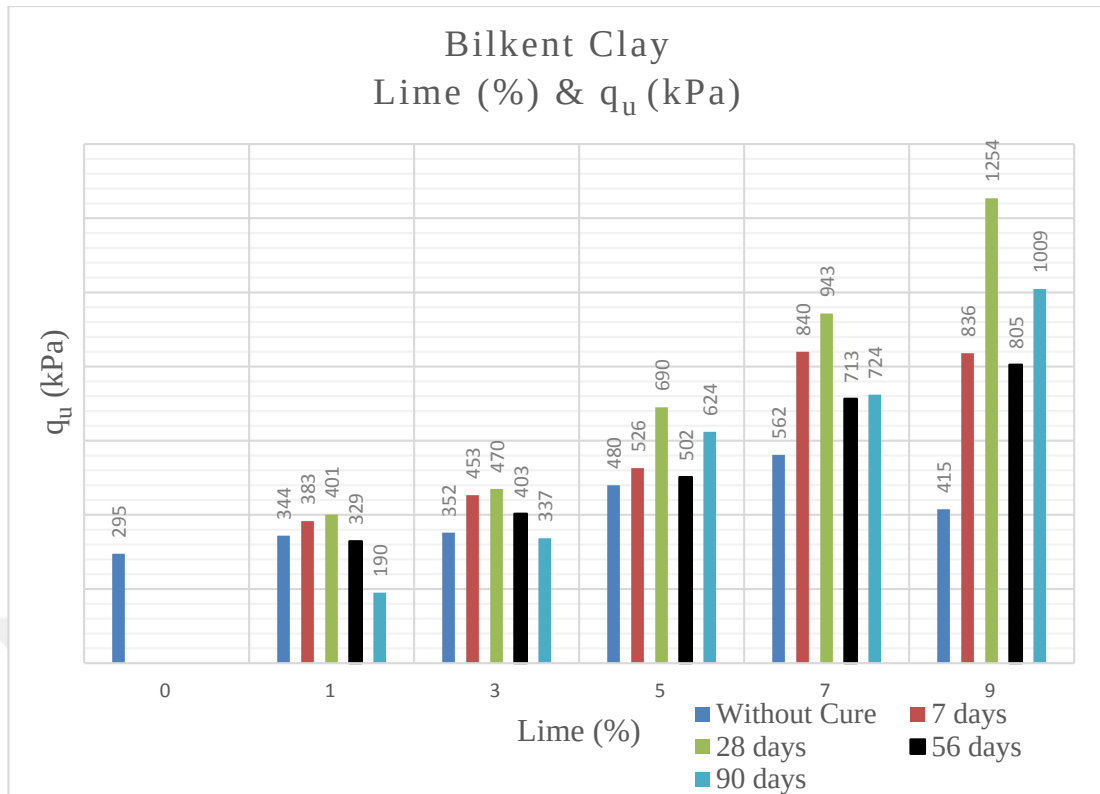


Figure 4.104 : Variation of unconfined compressive strength ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.

4.7.2 SEM (Scanning Electron Microscope) analyses

SEM analyses have been performed first for natural Esenboğa Clay and natural Bilkent Clay. Then, the microstructural development of a lime-treated clay sample (only for 7% lime content) extracted from a resilient modulus test sample and then waxed, has been imaged by SEM analyses after a curing period of 30 months for Esenboğa Clay. The magnifications in SEM analyses were 350, 1000, 2000, 5000 and 25000 in this study. EDAX equipment has been utilized for calcium mapping.

4.7.2.1 SEM for natural Esenboğa Clay

The SEM images for natural Esenboğa Clay with same magnifications but from different angles and with different magnifications are shown in Figure 4.105 to 110. The same magnification from different angles gave approximately similar soil fabric as expected in natural Esenboğa Clay. Soil fabric is imaged in detail with larger magnifications up to 5000 in this study. Figure 4.110 shows the maximum magnification of 5000 times for Esenboğa Clay samples. Smectite group minerals are

frequently characterized by rose-shaped images (Fesharaki et al., 2007, Zangue Adjia et al., 2013). Similar rose-shaped images shown in Figure 4.110 give indication the montmorillonite minerals which tend to occur in thin equidimensional flakes that have a film-like appearance. The images shown in Figure 4.110 leaves an impression of cohesive thin films composing of basic texture of this kind of clays.

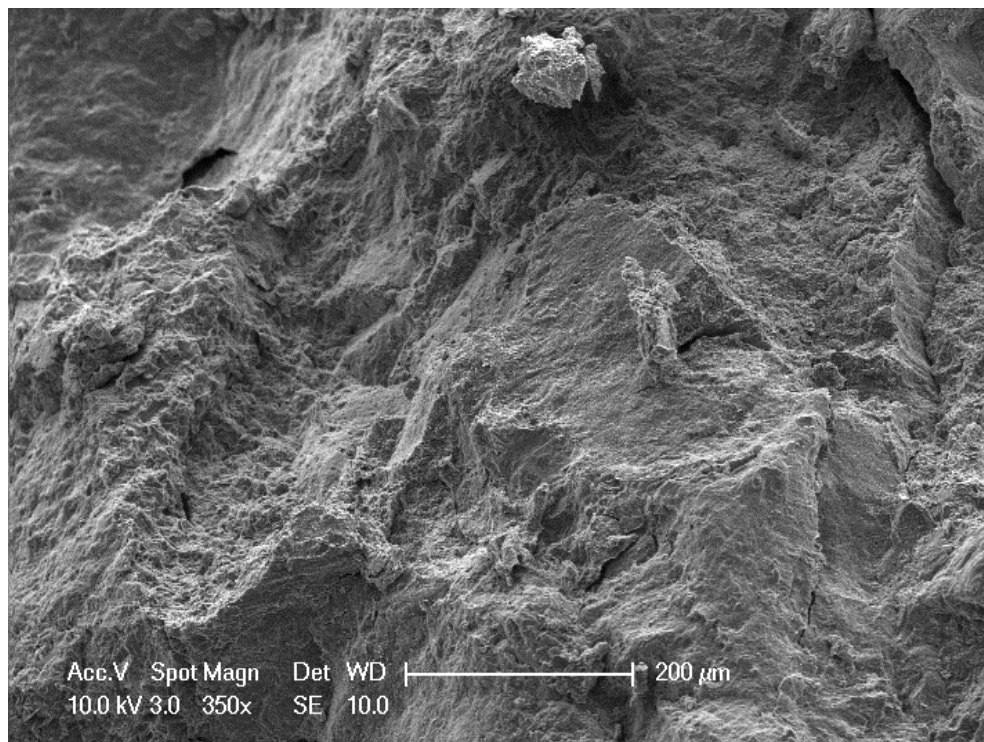


Figure 4.105 : SEM image taken at 350x magnification for natural Esenboğa Clay (1).

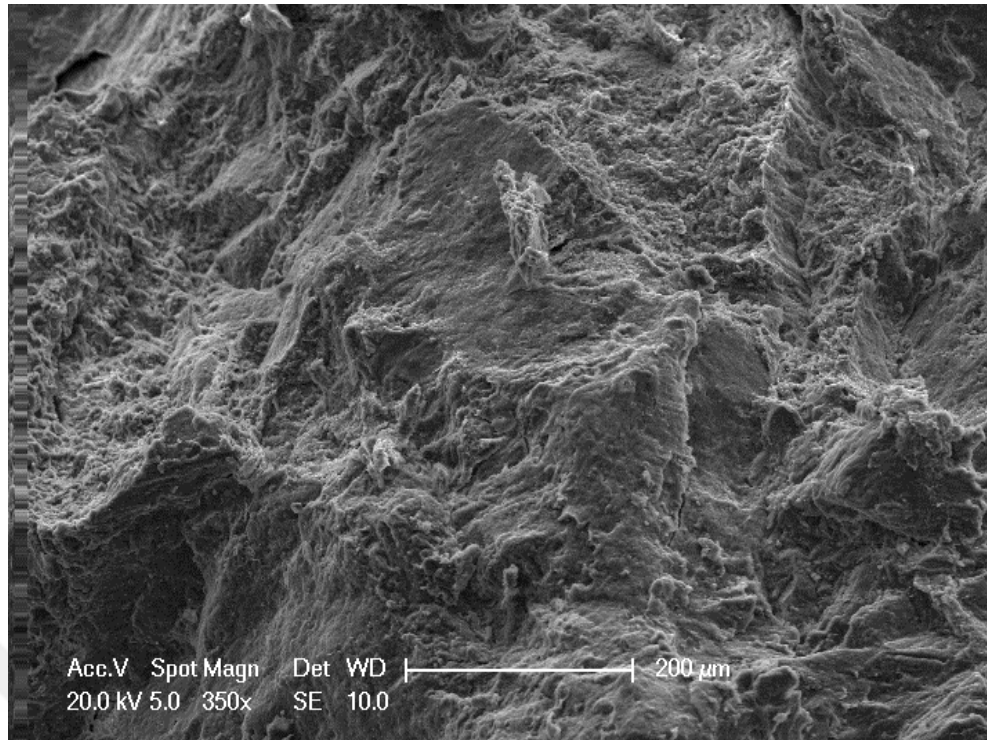


Figure 4.106 : SEM image taken at 350x magnification for natural Esenboğa Clay (2).

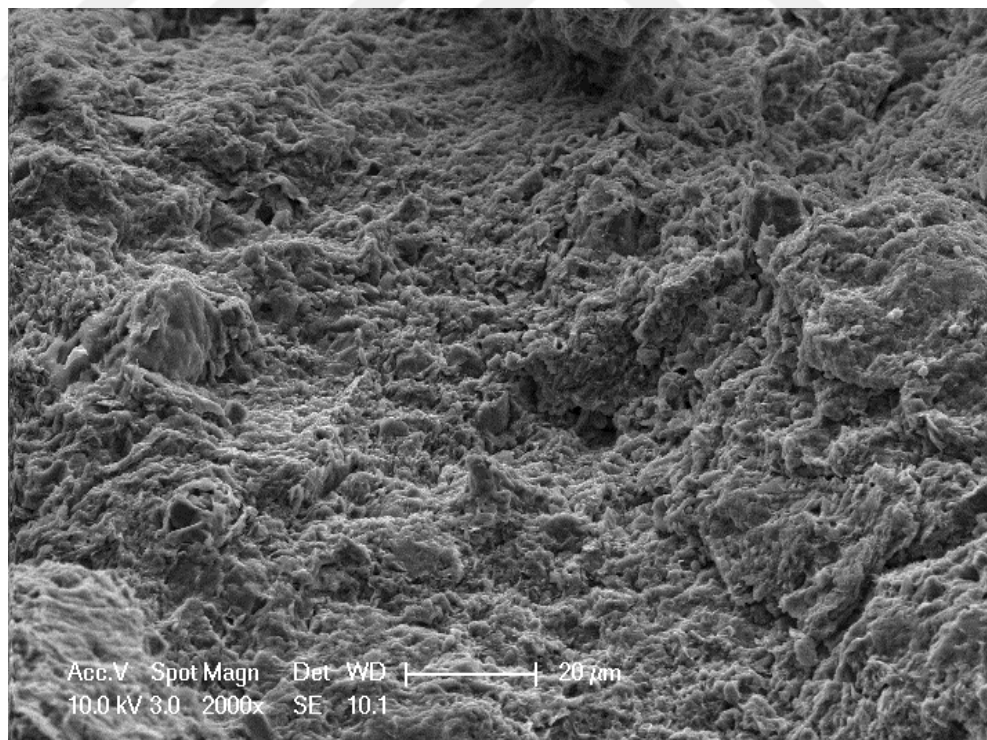


Figure 4.107 : SEM image taken at 2000x magnification for natural Esenboğa Clay (1).

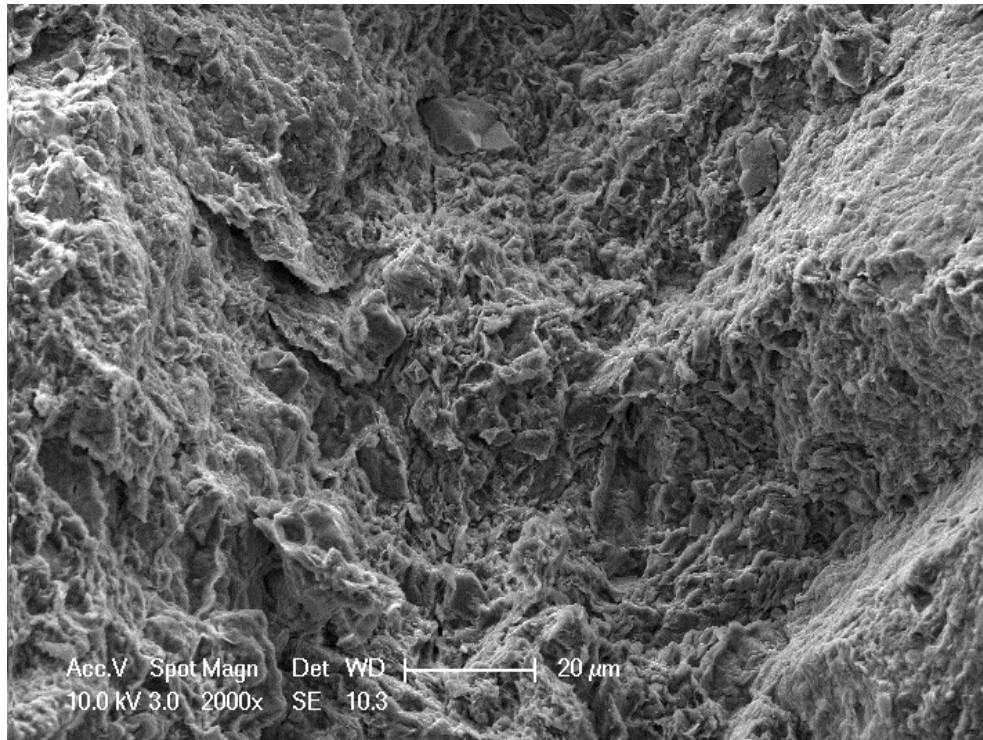


Figure 4.108 : SEM image taken at 2000x magnification for natural Esenboğa Clay (2).

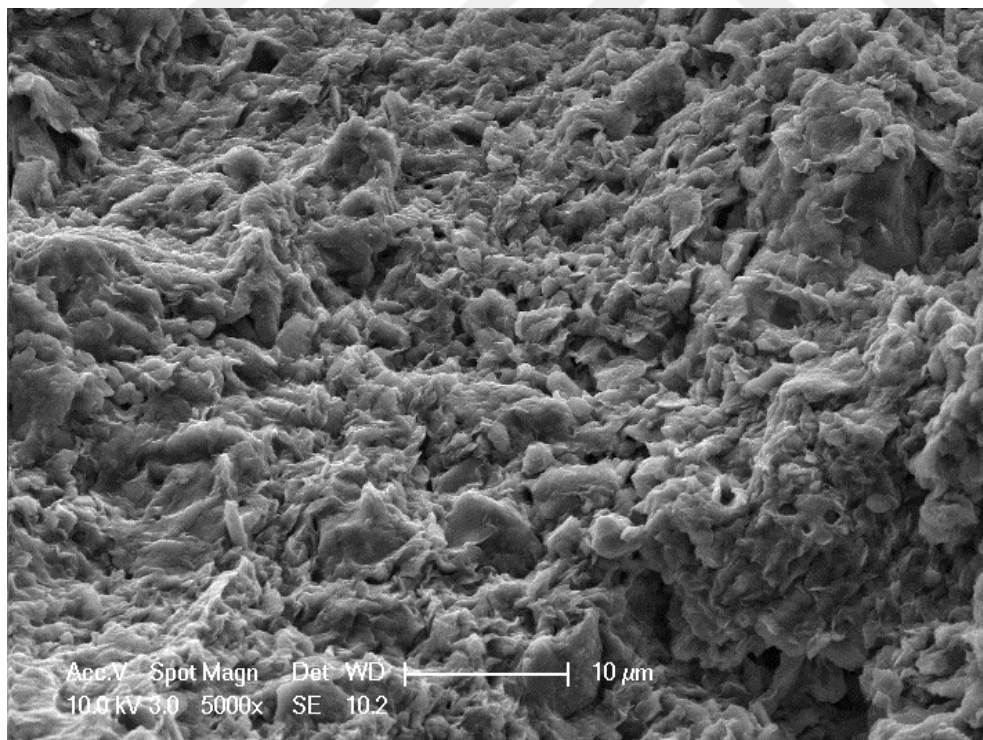


Figure 4.109 : SEM image taken at 5000x magnification for natural Esenboğa Clay (1).

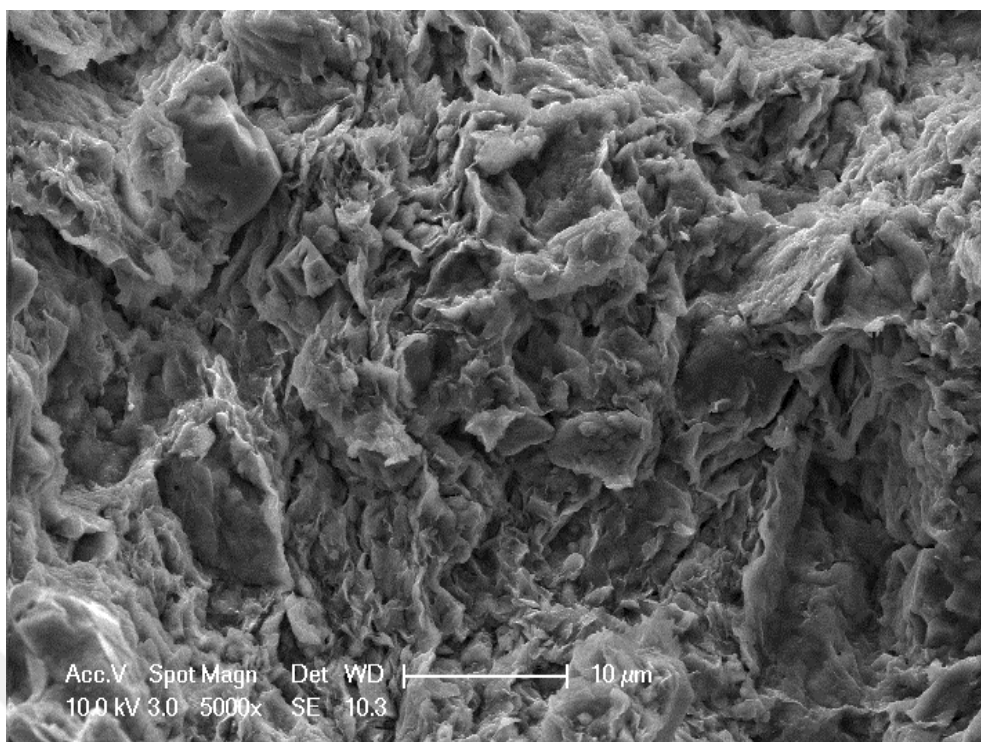


Figure 4.110 : SEM image taken at 5000x magnification for natural Esenboğa Clay (2).

Spectrum view, calcium mapping and chemical composition of Esenboğa Clay as deduced from SEM analyses are shown in Figure 4.111, Figure 4.112 and Figure 4.113 respectively.

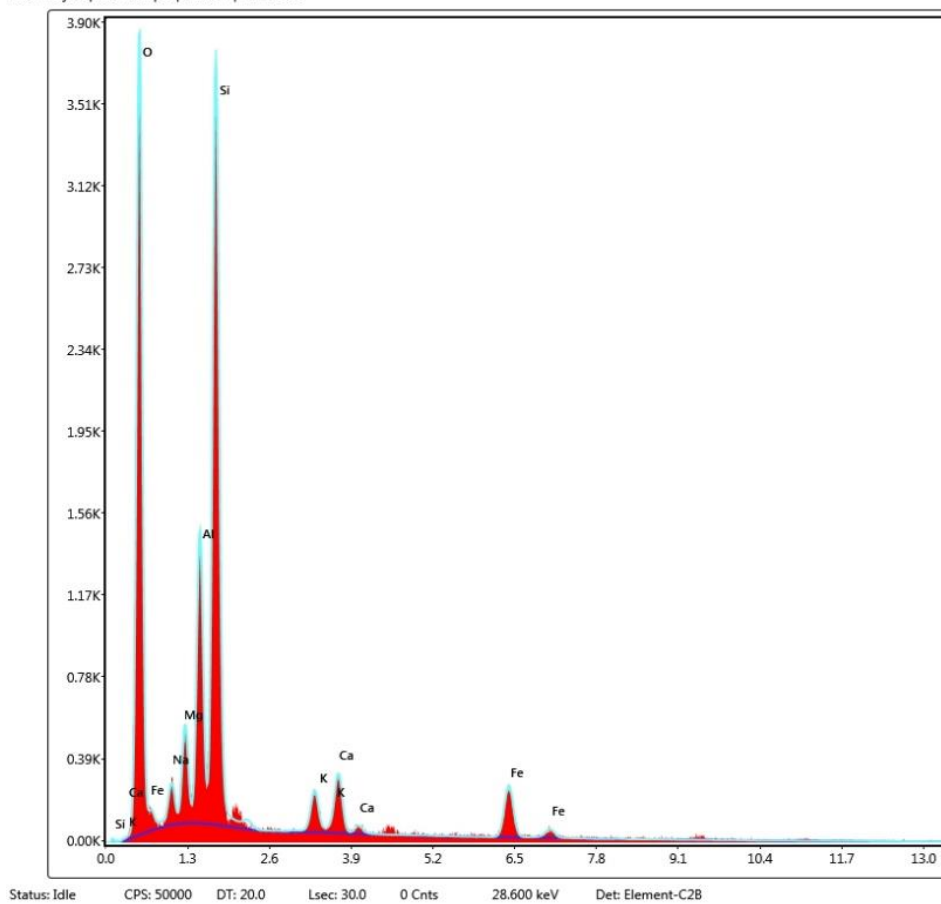
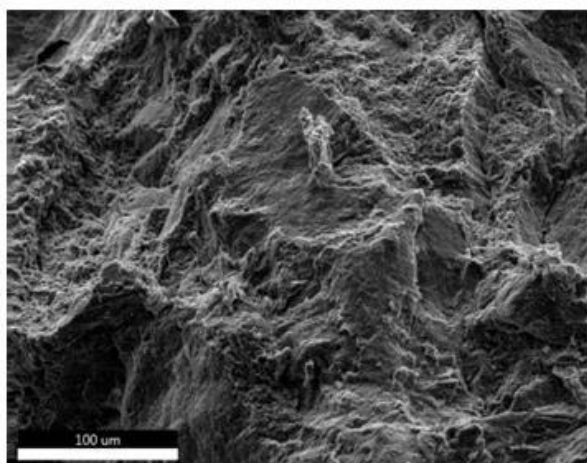


Figure 4.111 : Spectrum view for natural Esenboğa Clay

New Project

Author: Apex User
Creation: 10/25/2018 11:35:36 AM
Sample Name: New Sample

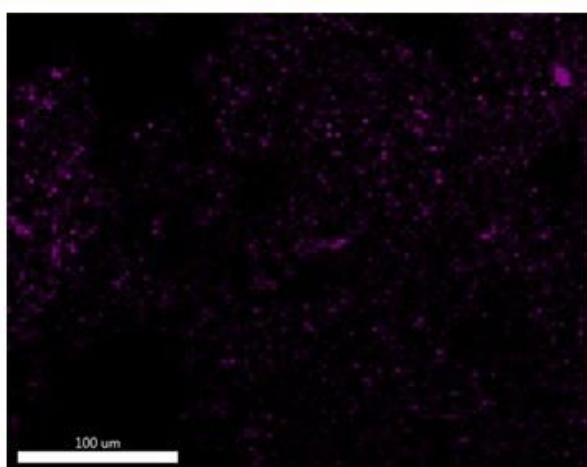
Area 97



Image

Live Map 1

ElementOverlay

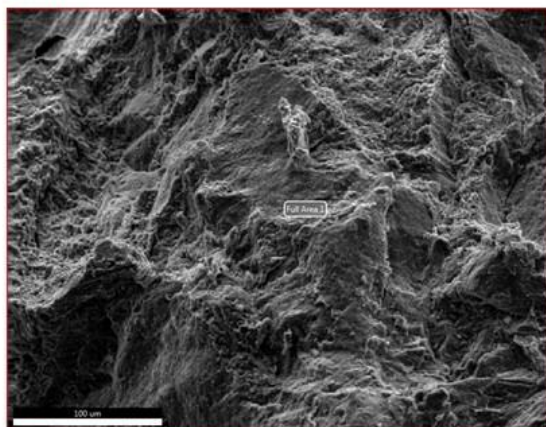


100% CaK

Figure 4.112 : Calcium mapping for natural Esenboğa Clay.

Author: Apex User
 Creation: 10/25/2018 11:28:07 AM
 Sample Name: New Sample

Area 97



Smart Quant Results

Element	Weight %	Atomic %	Error %
New Project New Sample Area 97 Full Area 1			
O K	50	65.05	8.05
NaK	2.38	2.16	15.17
MgK	3.76	3.22	8.69
AlK	9.6	7.41	5.96
SiK	23.42	17.35	5.06
K K	1.88	1	11.04
CaK	3.24	1.68	6.24
FeK	5.73	2.13	4.91

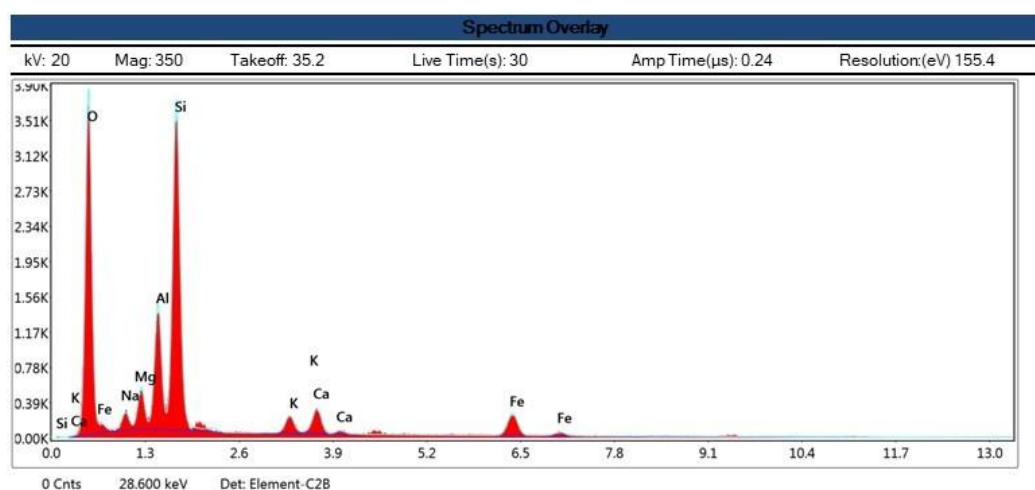


Figure 4.113 : Chemical composition for natural Esenboğa Clay.

4.7.2.2 SEM for Lime-Treated Esenboğa Clay

SEM analyses shown in Figure 4.114 to 4.119 are for lime-treated samples (7% lime content) with different magnification factors. Meaningful differences in soil fabric are observed for lime-treated samples. As shown in Figure 4.115, two different parts of the samples have been observed in SEM analysis. Right and left views are shown Figure 4.116 and 4.117 respectively with 2000 times magnification. It is noticed that the left part of the SEM image is denser than the other side. Porous nature of 7% lime-treated samples is quite pronounced at the right view shown in Figure 4.115.

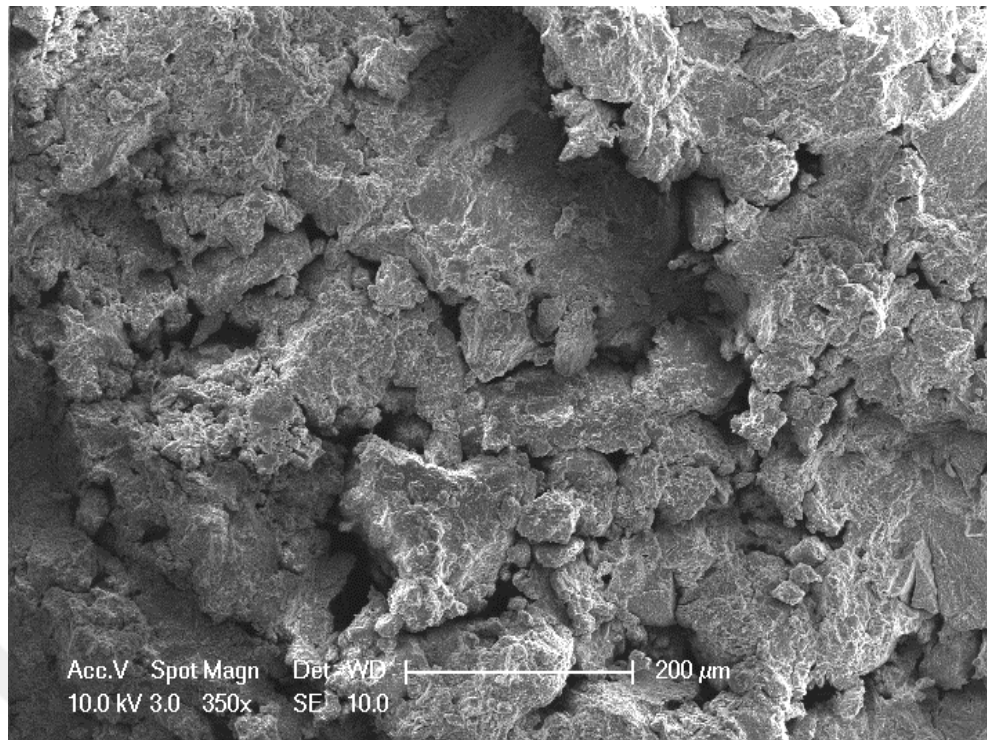


Figure 4.114 : SEM image taken at 350x magnification for lime-treated Esenboğa Clay.

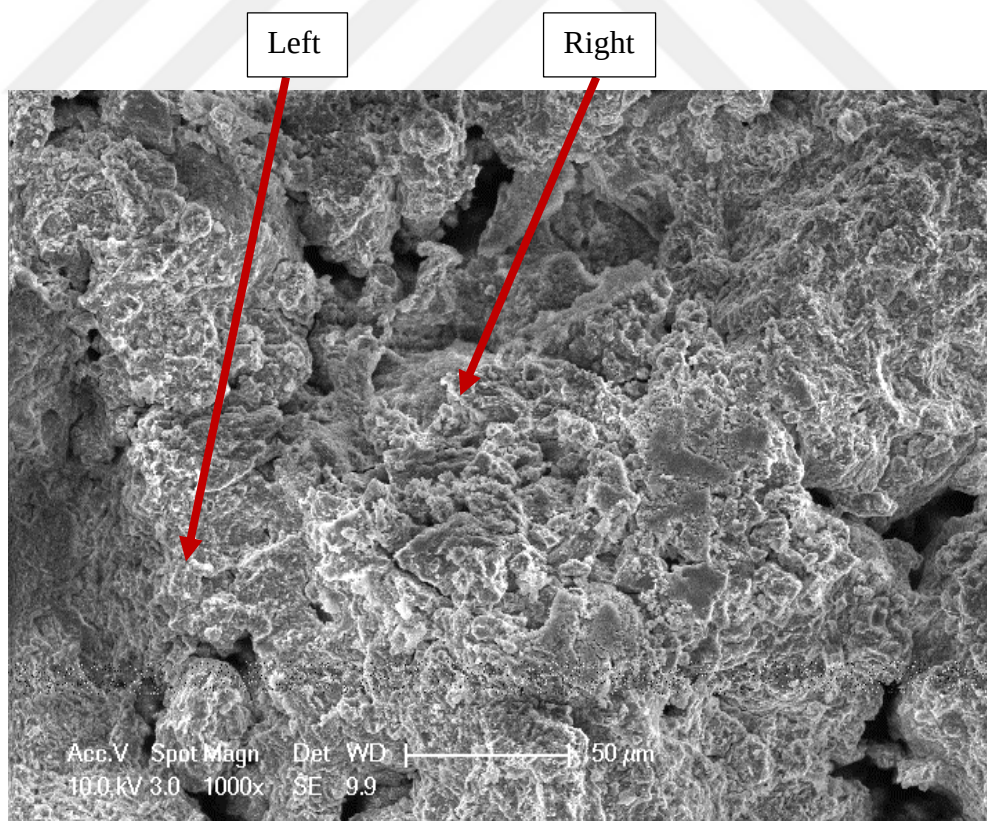


Figure 4.115 : SEM image taken at 1000x magnification for lime-treated Esenboğa Clay.

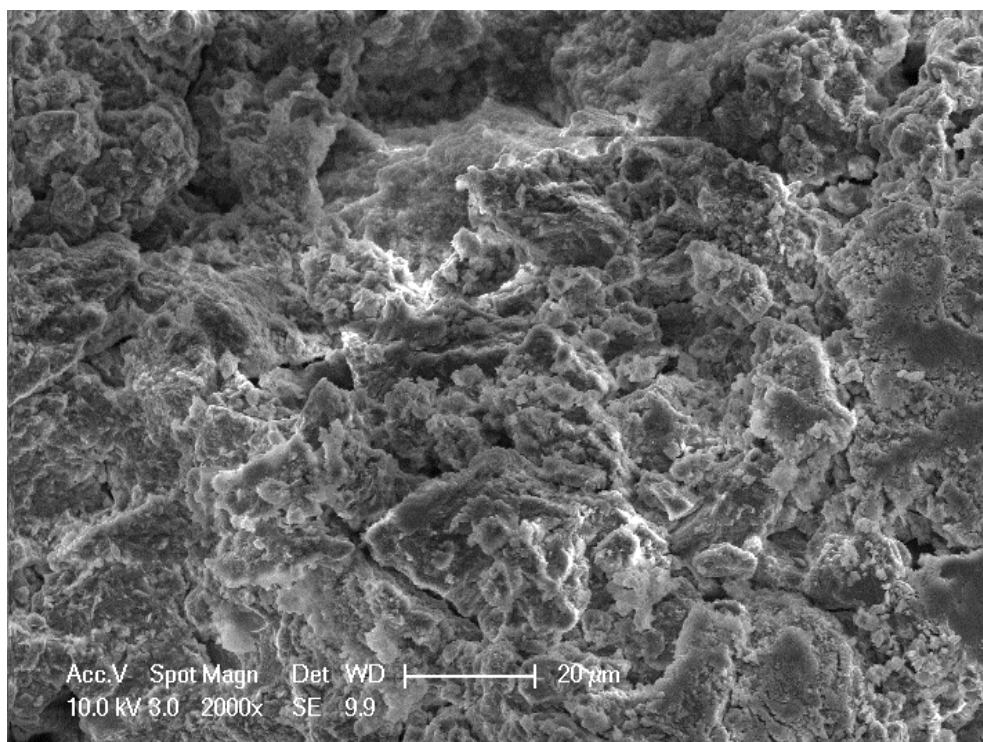


Figure 4.116 : SEM image taken at 2000x magnification for lime-treated Esenboğa Clay (Right in Figure 4.115).

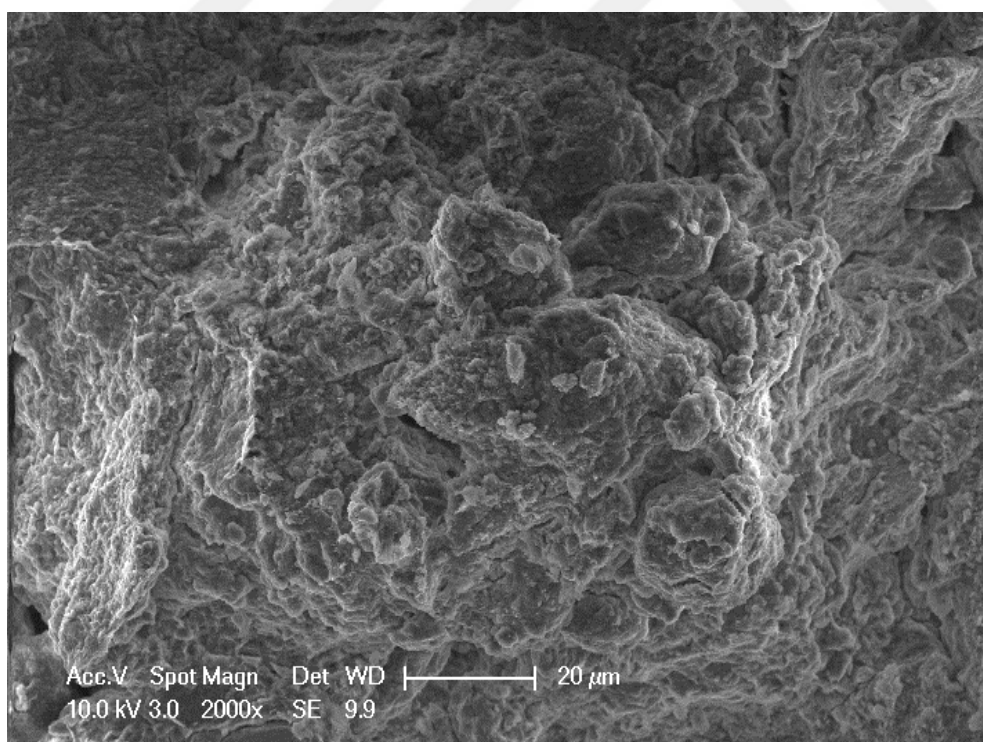


Figure 4.117 : SEM image taken at 2000x magnification for lime-treated Esenboğa Clay (Left in Figure 4.115).

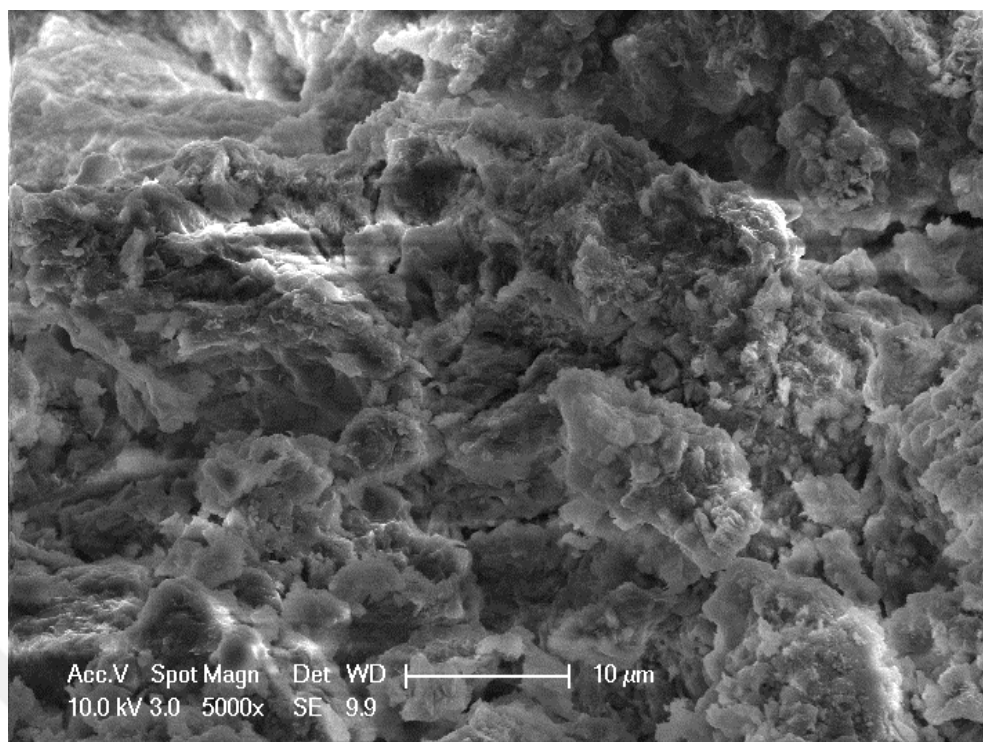


Figure 4.118 : SEM image taken at 5000x magnification for lime-treated Esenboğa Clay (1).

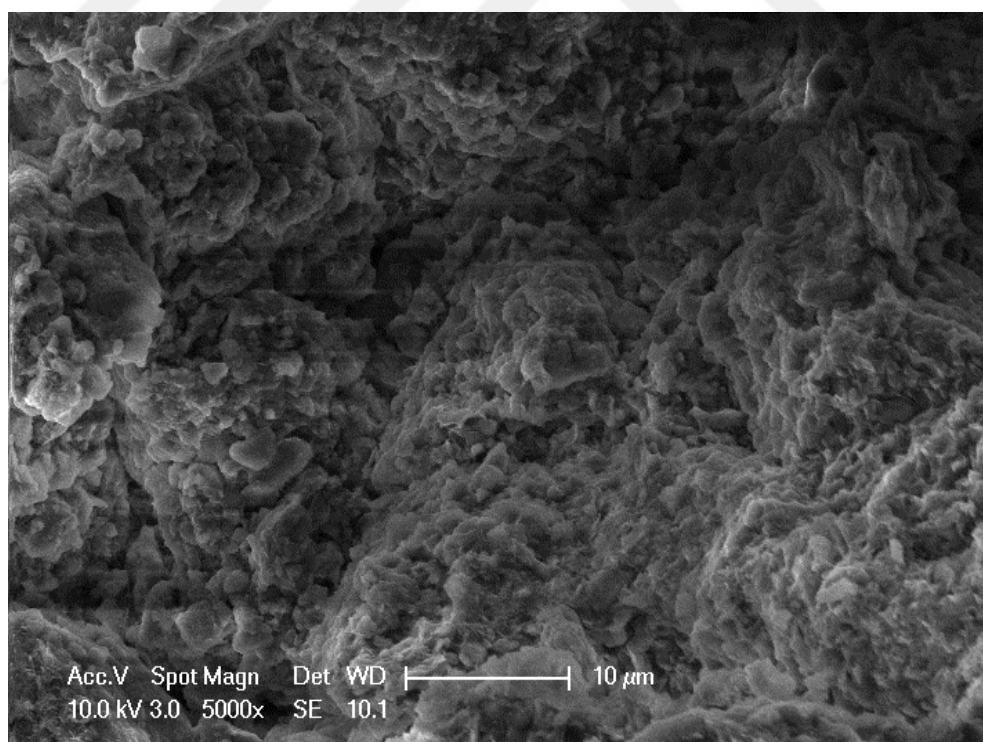


Figure 4.119 : SEM image taken at 5000x magnification for lime-treated Esenboğa Clay (2).

Spectrum view of the lime-treated Esenboğa Clay is shown in Figure 4.120.

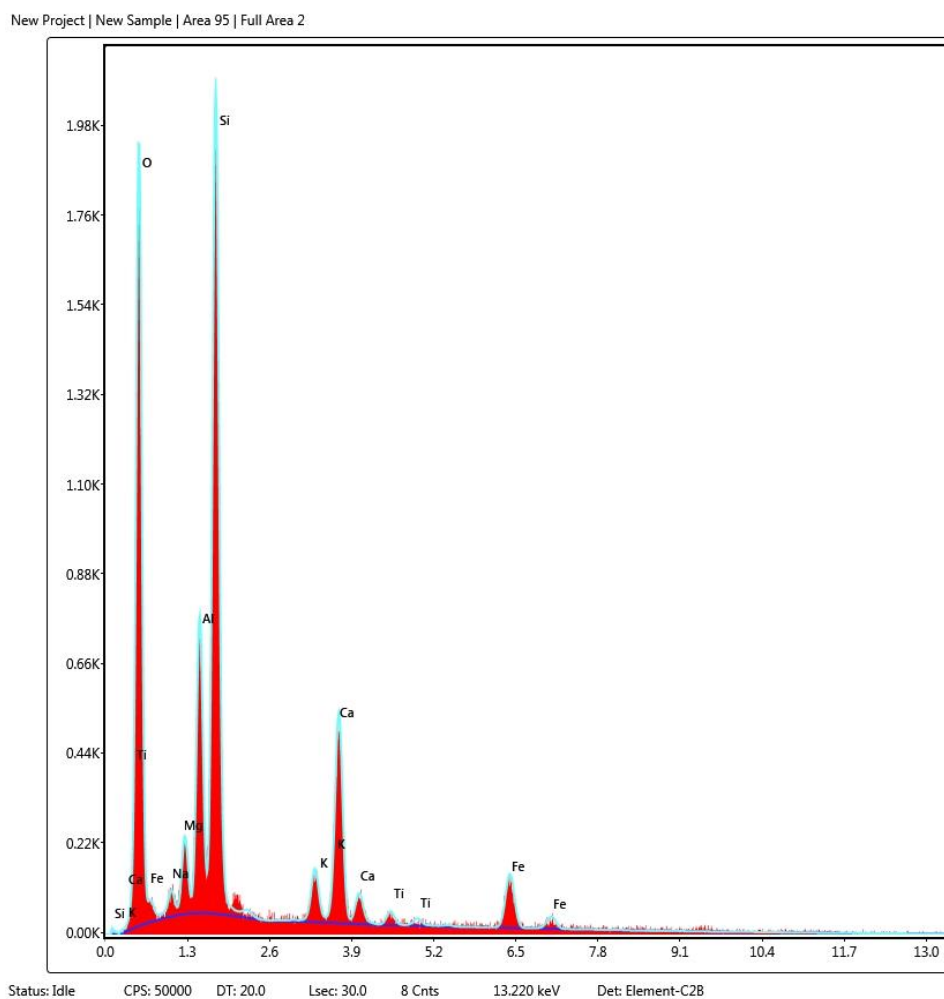


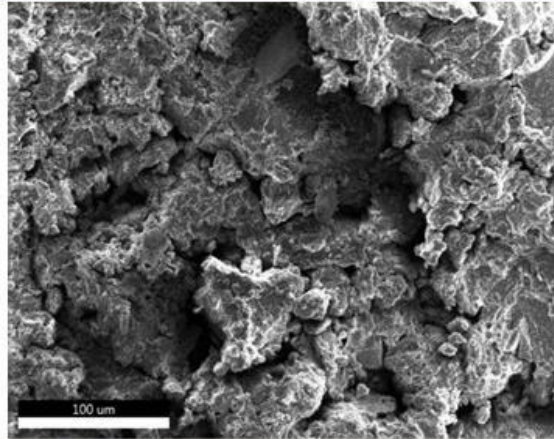
Figure 4.120 : Spectrum view for lime-treated Esenboğa Clay.

When the calcium mapping shown in Figure 4.121 is compared with the mapping of natural Esenboğa Clay, it is observed that calcium-rich regions shown by purple points is more widespread in Figure 4.121.

New Project

Author: Apex User
Creation: 10/25/2018 11:06:03 AM
Sample Name: New Sample

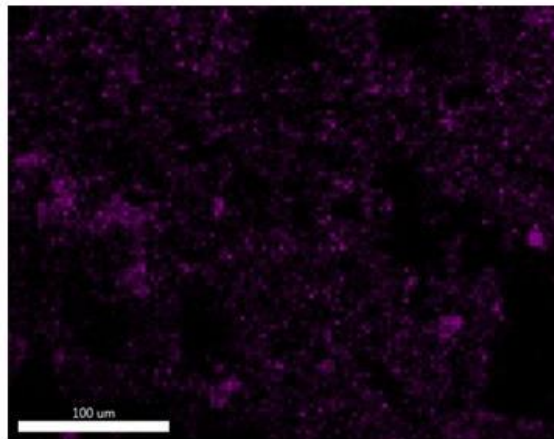
Area 95



Image

Live Map 2

ElementOverlay



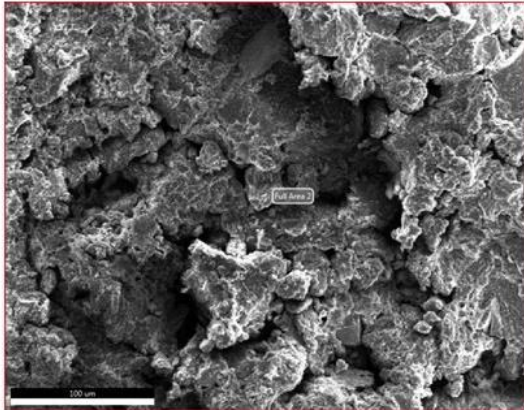
100% CaK

Figure 4.121 : Calcium mapping for lime-treated Esenboğa Clay.

Chemical analysis results are given in Figure 4.122 below. Calcium silicate hydrate and calcium aluminate hydrate formations, abbreviated as CSH and CAH respectively in literature, occurs due to high concentration of calcium. The calcium mapping given above shows a significant increase in calcium concentration when compared with the natural Esenboğa Clay. Morphology changes from dense to porous. This is an indication of flocculation and agglomeration. Thus, pozzolanic reactions occur in the long term. It should be remembered that the tested sample here has been cured for about 30 months.

Author: ApexUser
 Creation: 10/25/2018 10:57:00 AM
 Sample Name: New Sample

Area 95



Smart Quant Results

Element	Weight %	Atomic %	Error %
O K	51.69	67.75	8.91
MgK	2.99	2.58	10.14
AlK	8.12	6.31	6.34
SiK	20.66	15.43	5.15
K K	1.94	1.04	11.9
CaK	9.57	5.01	3.81
FeK	5.03	1.89	6.84

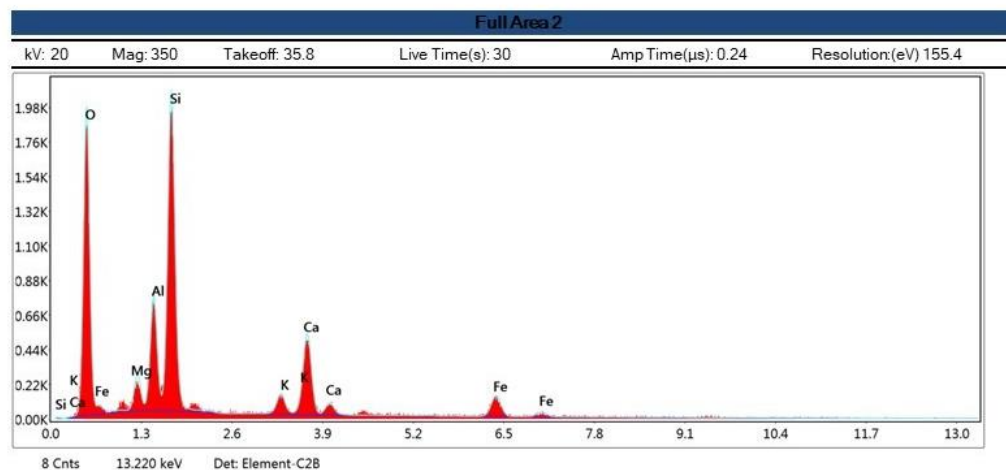


Figure 4.122 : Chemical composition for lime-treated Esenboğa Clay.

4.7.2.3 SEM for natural Bilkent Clay

The SEM images for natural Bilkent Clay with same magnifications but from different angles and with different magnifications are shown in Figure 4.123 to Figure 4.128. The same magnification from different angles gave different soil fabrics in this case in contrast to Esenboğa Clay case. It should be noted that there were some natural lime veins within the natural Bilkent Clay during excavation. Therefore, this difference can be attributed to existing lime veins observed in natural Bilkent Clay. In the first set of figures below, the effect of these lime veins in the natural Bilkent Clay can be clearly seen especially in larger magnifications. In the first set of figures, it is realized

that free calcium content is high and calcium carbonate does not exist. Thus, second set of figures represent the natural Bilkent Clay more properly if an evaluation of a natural clay without veins is needed.

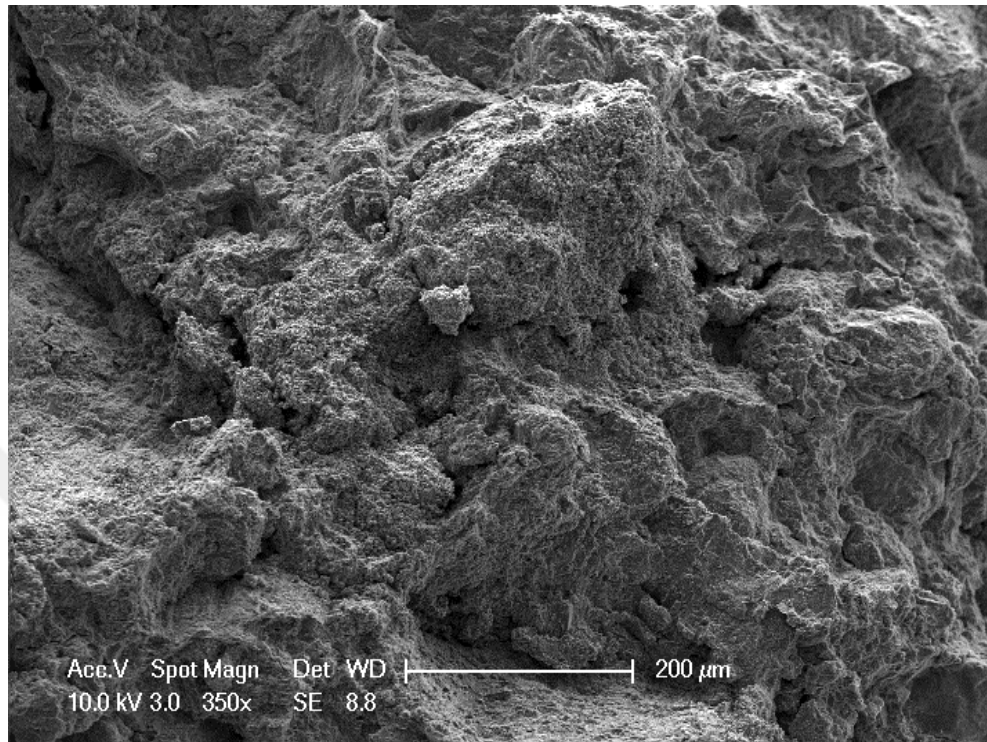


Figure 4.123: SEM image taken at 350x magnification for natural Bilkent Clay (1).

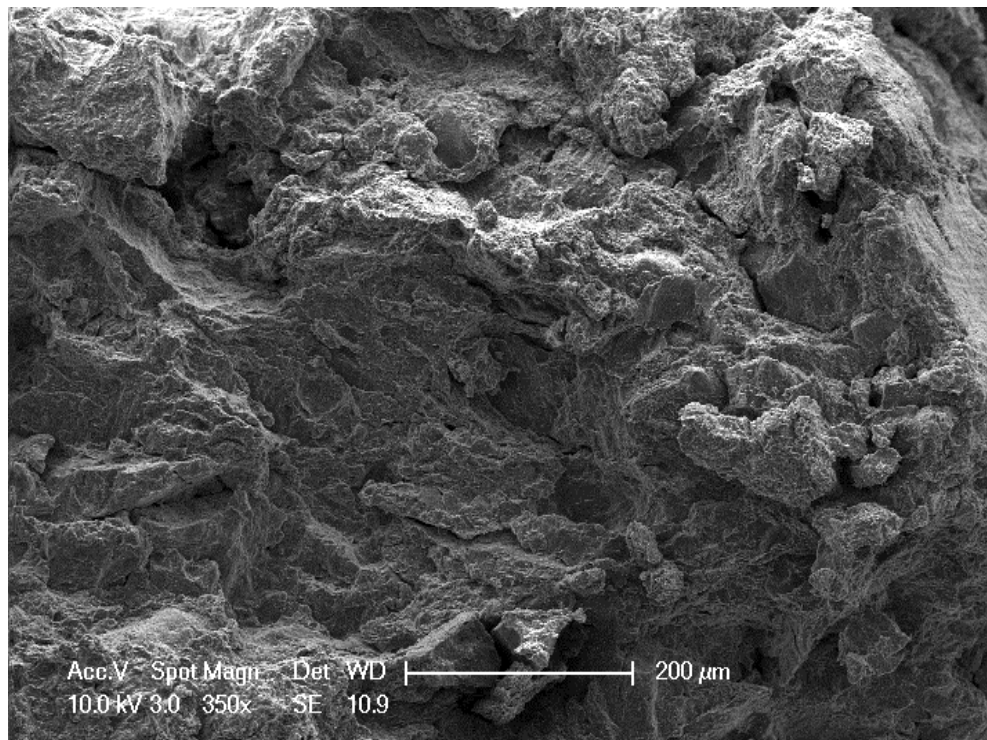


Figure 4.124 : SEM image taken at 350x magnification for natural Bilkent Clay (2).

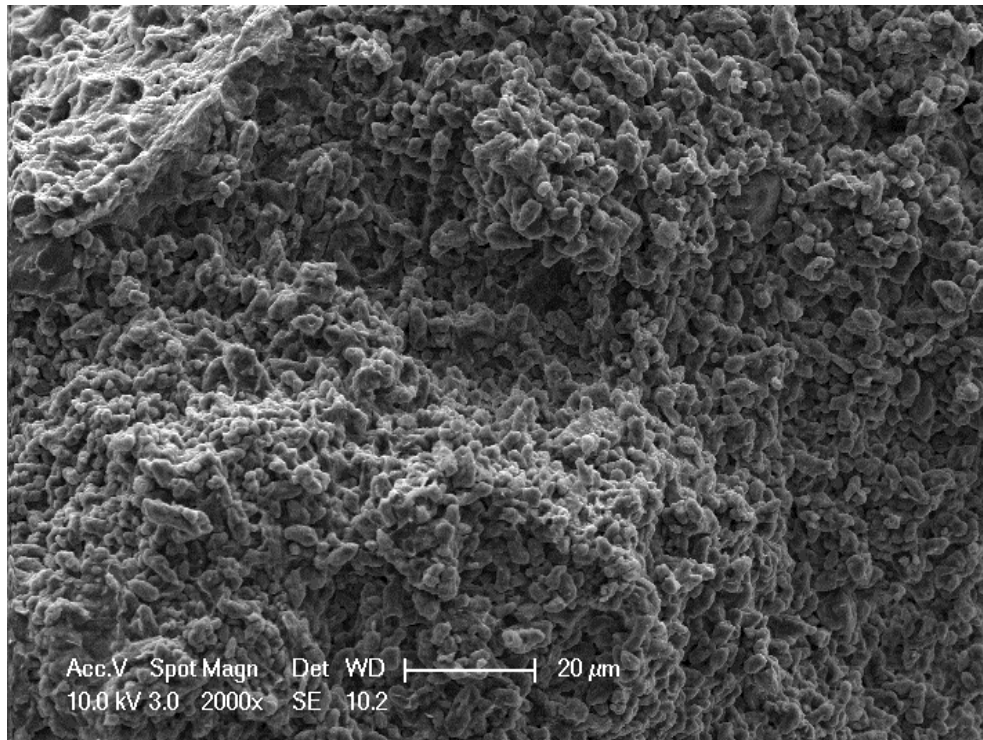


Figure 4.125 : SEM image taken at 2000x magnification for natural Bilkent Clay (1).

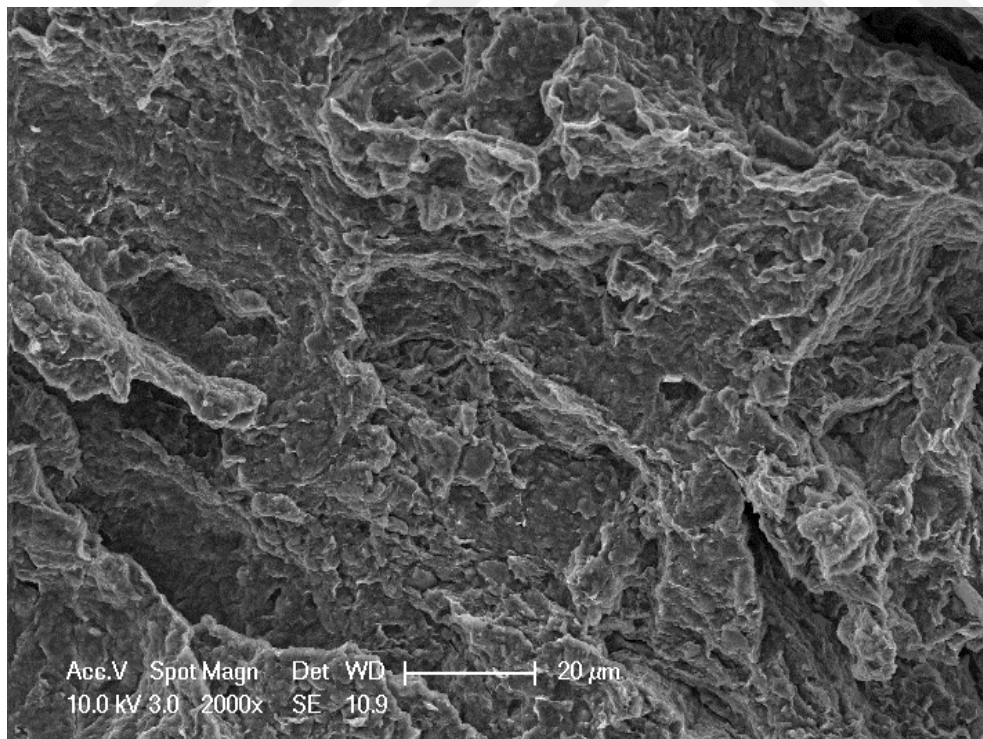


Figure 4.126 : SEM image taken at 2000x magnification for natural Bilkent Clay (2).

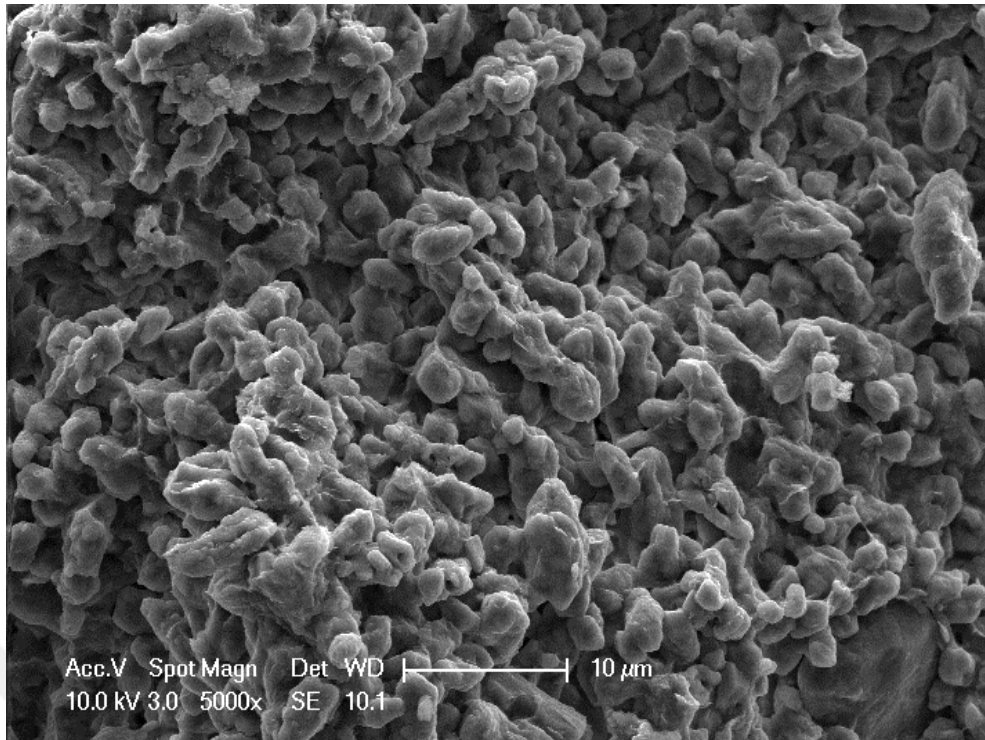


Figure 4.127 : SEM image taken at 5000x magnification for natural Bilkent Clay (1).

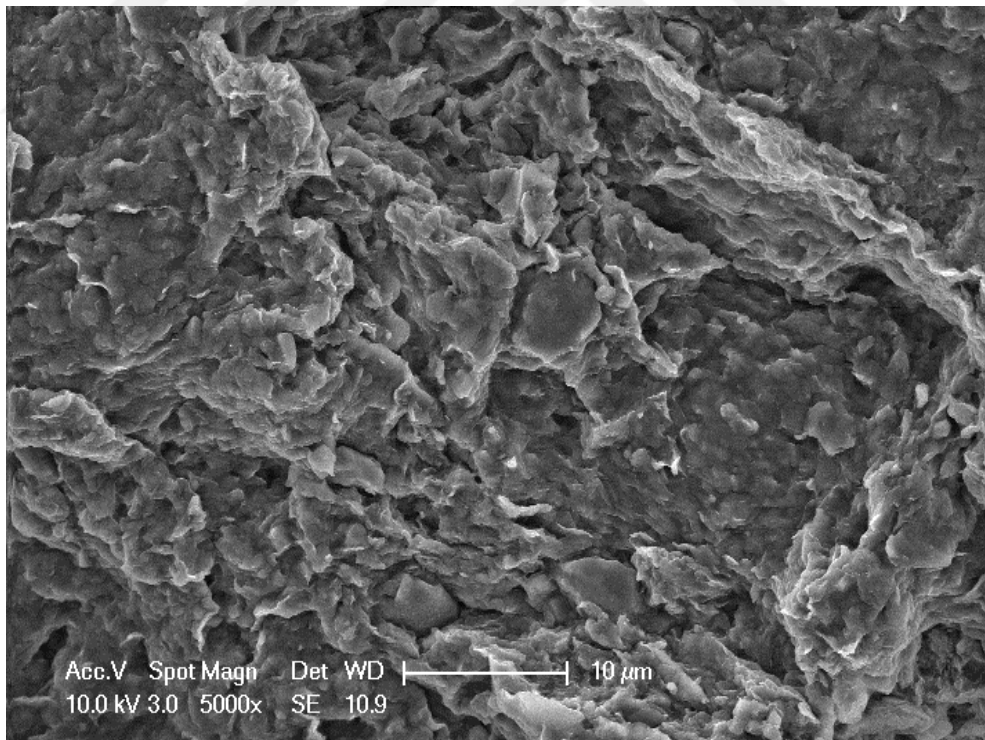


Figure 4.128 : SEM image taken at 5000x magnification for natural Bilkent Clay (2).

The difference in the images due to original lime veins existing in the natural Bilkent Clay explained above can also be noticed in the spectrum views given in Figure 4.129 and Figure 4.130.

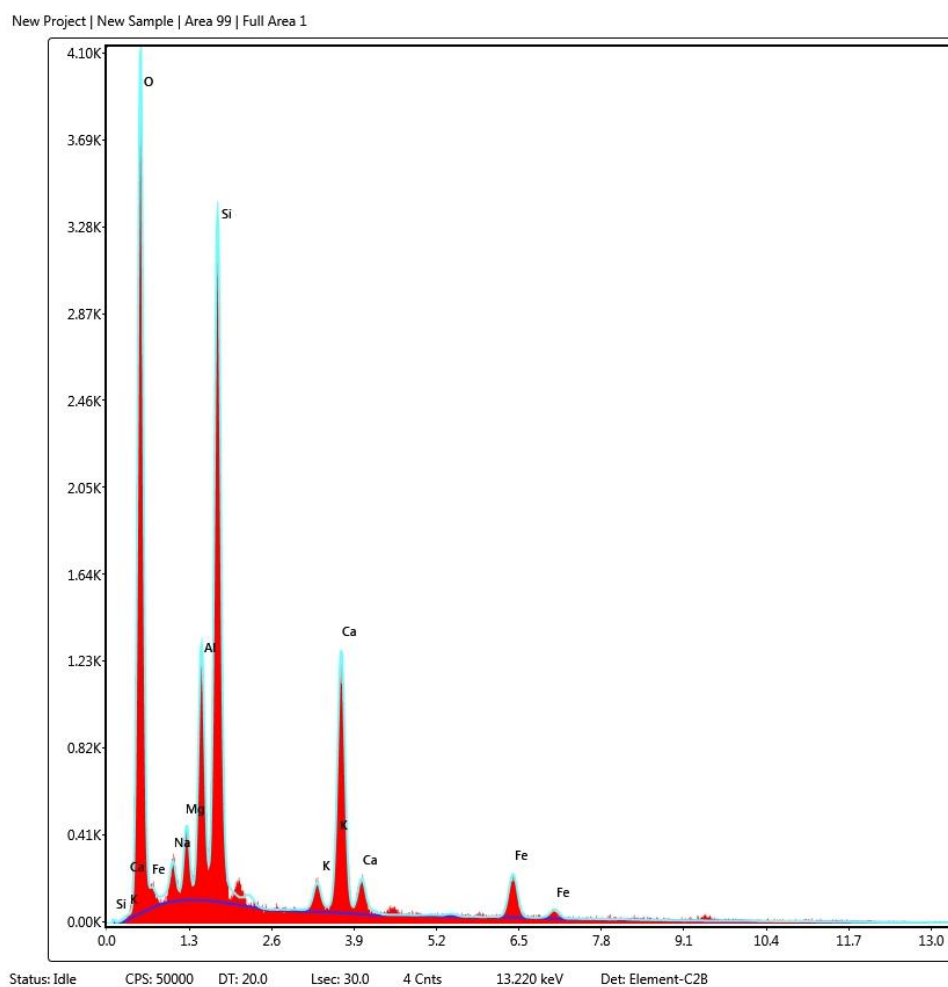


Figure 4.129 : First spectrum view for natural Bilkent Clay.

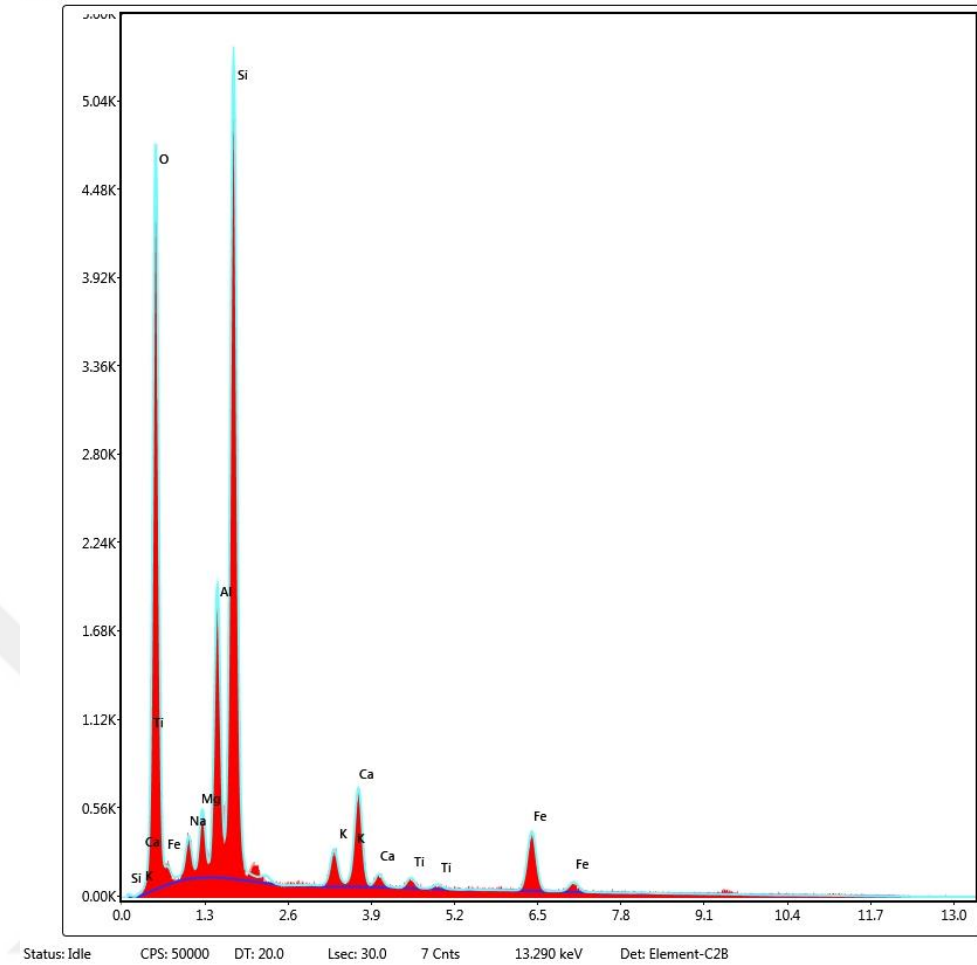


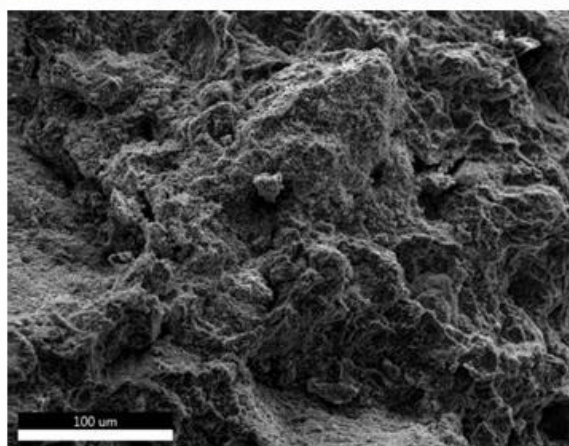
Figure 4.130 : Second spectrum view for natural Bilkent Clay.

Calcium mappings shown in Figure 4.131 to Figure 4.134 below indicate that the existing lime veins in the natural Bilkent Clay is quite dominant in the first set of figures as indicated by the dense purple points in Figure 4.131.

New Project

Author: Apex User
Creation: 10/25/2018 11:55:07 AM
Sample Name: New Sample

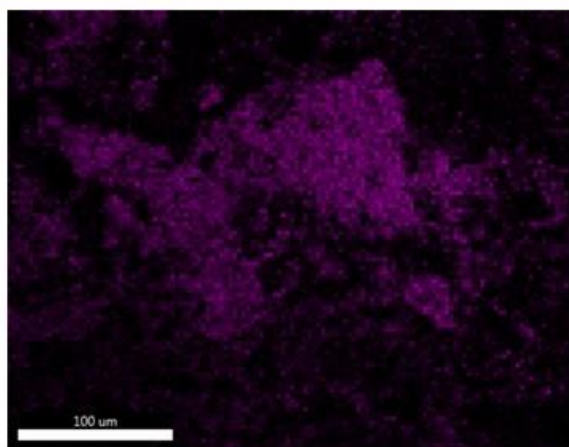
Area 99



Image

Live Map 1

ElementOverlay



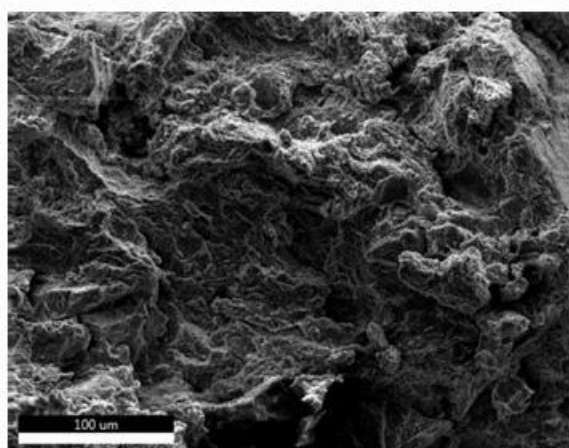
100% CaK

Figure 4.131 : First calcium mapping for Bilkent Clay.

New Project

Author: Apex User
Creation: 10/25/2018 12:06:24 PM
Sample Name: New Sample

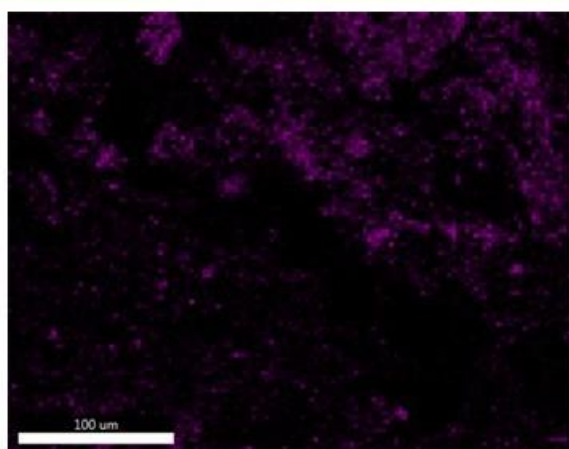
Area 100



Image

Live Map 1

ElementOverlay



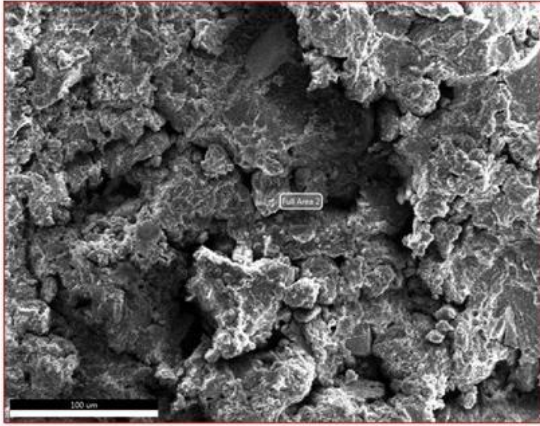
100% CaK

Figure 4.132 : Second calcium mapping for Bilkent Clay.

Chemical compositions of the two sets of images are given in Figure 4.133 and Figure 4.134. Calcium content in the first figure is about two times more than that of the second figure as noticed.

Author: Apex User
Creation: 10/25/2018 10:57:00 AM
Sample Name: New Sample

Area 95



Smart Quant Results

Element	Weight %	Atomic %	Error %
O K	51.69	67.75	8.91
MgK	2.99	2.58	10.14
AlK	8.12	6.31	6.34
SiK	20.66	15.43	5.15
K K	1.94	1.04	11.9
CaK	9.57	5.01	3.81
FeK	5.03	1.89	6.84

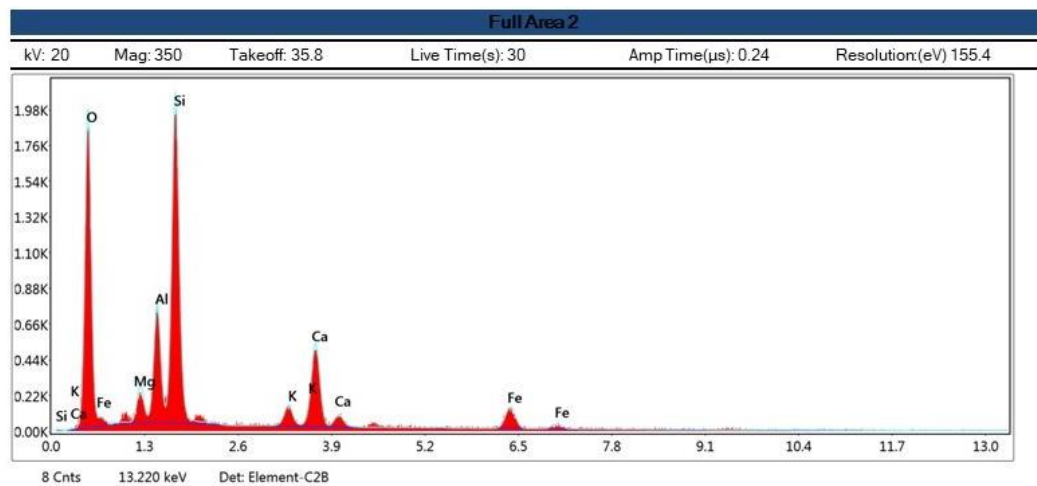
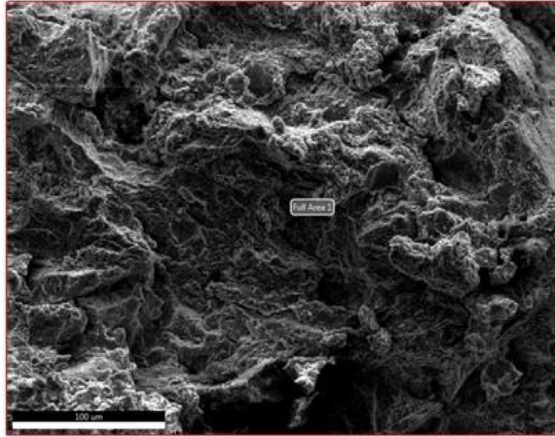


Figure 4.133 : First chemical composition for Bilkent Clay.

Author: ApexUser
 Creation: 10/25/2018 12:00:40 PM
 Sample Name: New Sample

Area 100**Smart Quant Results**

Element	Weight %	Atomic %	Error %
O K	48.56	64.32	8.27
NaK	2.43	2.24	13.25
MgK	2.49	2.17	8.82
AlK	9.05	7.11	5.77
SiK	23.66	17.85	4.84
K K	1.6	0.87	10.55
CaK	5.15	2.72	4.4
TiK	0.63	0.28	19.4
FeK	6.44	2.44	4.43

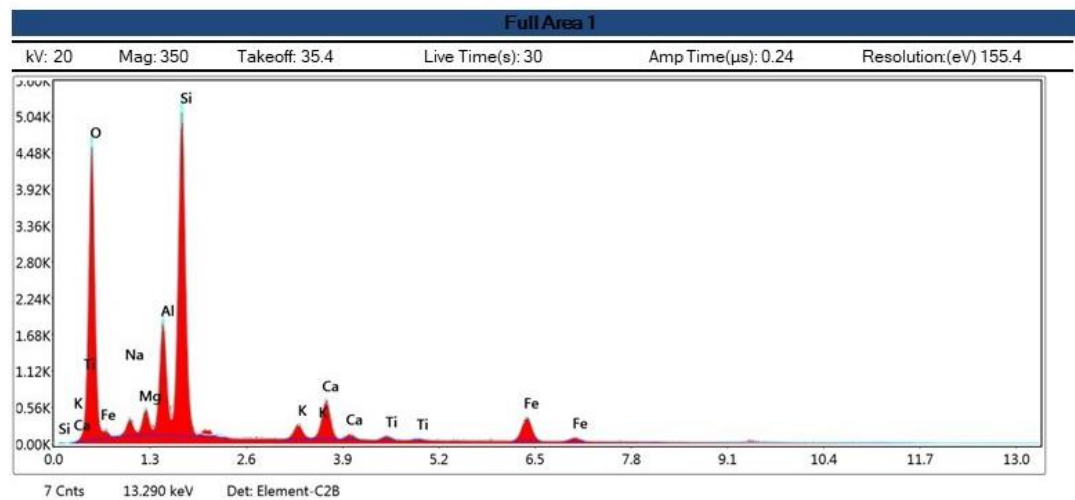


Figure 4.134 : Second chemical composition for Bilkent Clay.



5. DISCUSSION

As a recall, there are two types of expansive clays which have been taken from the different sites in Ankara. The extremely highly and very highly plastic clays ($PI_1=101\%$ and $PI_2=76\%$) called as Esenboğa and Bilkent Clays respectively, have been tested in many respects at the laboratory. The clays have been stabilized by the agent lime and fly ash. The detailed mineralogical analyses of the clays and the agents have been revealed. The index properties, the swelling characteristics, the unconfined compressive strengths for both soaked and non-soaked conditions, the resilient modulus and the methylene blue values, compressibility characteristics have all been found and evaluated within the scope of this research. The laboratory test results presented up to now have been evaluated one by one for each clay.

The results are to be introduced in a comparative manner in this part of the thesis. The results and the conclusions presented up to now are discussed here for both clays in the same graphs. Lime-treated samples of both clays are discussed in the first part of this section. After that, the results of the stabilized samples with fly ash are presented for discussion.

5.1 Comparison of Lime-Treated Samples of Esenboğa and Bilkent Clays

An extremely highly plastic clay type of soil, which was taken from clay deposits from the Akyurt district of Ankara, near Esenboğa Airport (Esenboğa clay) and a very highly plastic clay type of soil taken from clay deposits of the Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway (Bilkent Clay) have been mixed with hydrated lime at different percentages of dry weight of the soil (1%, 3%, 5%, 7% and 9%). Samples have been prepared at optimum water content and maximum dry density by means of Harvard miniature compaction test (Wilson, 1950, ASTM D698- 07). Grain size distribution, Atterberg limits, strength and swelling properties of the lime treated clay samples have been determined at each lime content which is increased step by step. The effect of lime stabilization on Atterberg limits, strength and swelling characteristics are given in Figure 5.1 to Figure 5.11.

5.1.1 Index parameters

When the Figure 5.1 and Figure 5.2 are examined it is seen that liquid limit values decrease regularly for Bilkent Clay (i.e. from 109% of natural clay to 72% of 9% lime content). However, the decrease in liquid limit is quite moderate up to 3% for Esenboğa Clay. There is a sudden decrease between 3% and 5% lime contents. The decrease in liquid limit is also quite moderate after 5% lime content for Esenboğa Clay. These observations are more or less valid for plasticity index. It should be noted here that lime is a quite effective agent to decrease plasticity index in very highly to extremely highly plastic soils like Bilkent and Esenboğa Clays. When the clay is more plastic this effectiveness increases as observed from Esenboğa Clay results (i.e. although Esenboğa Clay is more plastic than Bilkent Clay their plasticity index values are equalized at 9% lime content). Thus, Esenboğa Clay which is extremely highly plastic reacts with lime more effectively as far as the plasticity index is concerned.

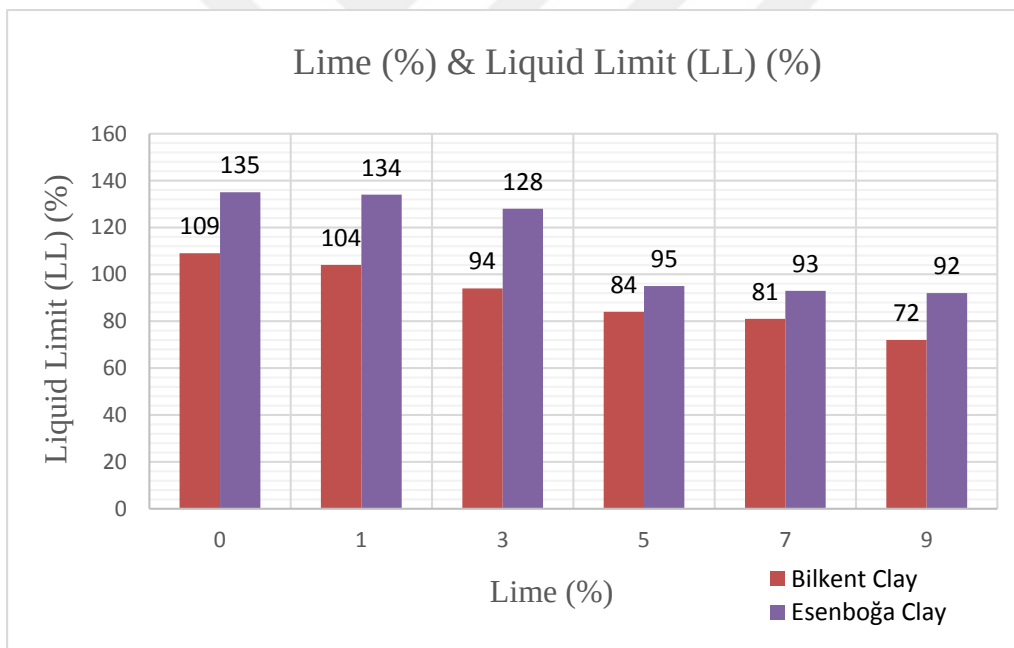


Figure 5.1 : Comparison of liquid limits (LL) with lime content.

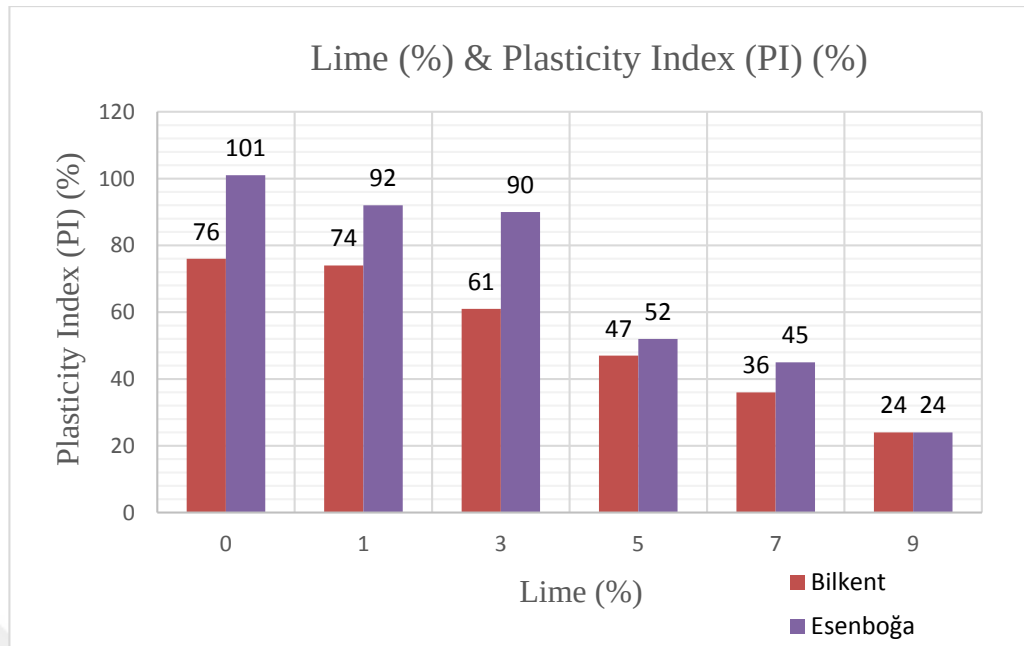


Figure 5.2 : Comparison of plasticity indices (PI) with lime content.

Shrinkage limit increases as the lime content increases as given in Figure 5.3 for both clays. The soils exhibit the highest value of shrinkage limit at 9% lime content. Up to 7% of lime content, the shrinkage limit increments are moderate. Whereas the increments are quite high when the lime content increases from 7 to 9%.

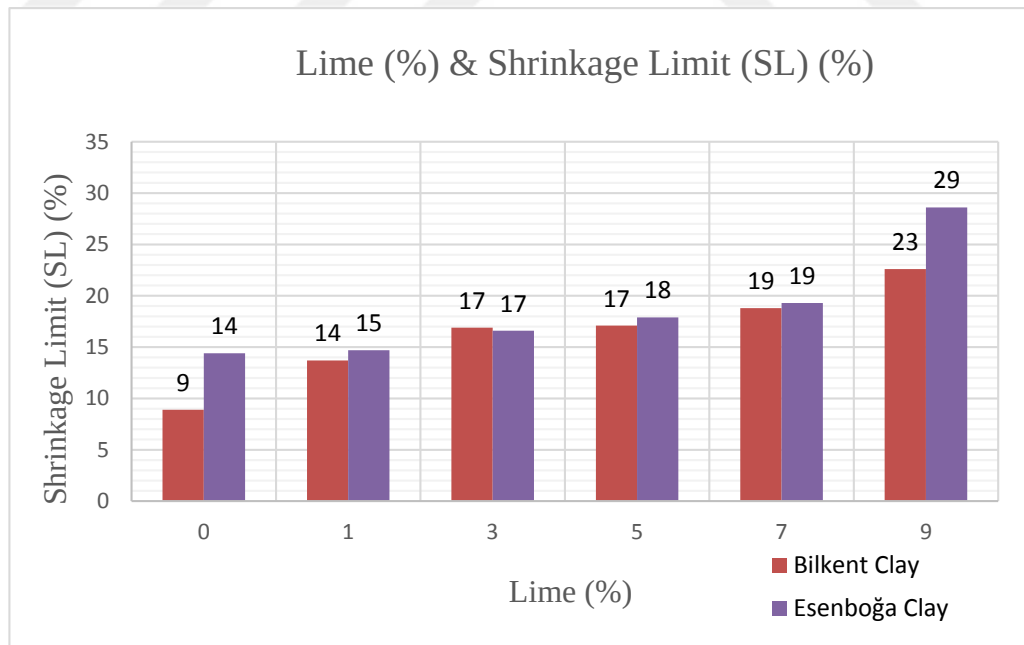


Figure 5.3 : Comparison of shrinkage limits (SL) with lime content.

5.1.2 Swell parameters

The allowable swell percentage for surficial structures is generally accepted as 0.5% as given in Highway Standards (KGM, 2005). Therefore, the lime contents in this study to obtain allowable swell percentage for very highly to extremely high plastic soils like Bilkent and Esenboğa Clays is about 8 to 10% as shown in Figure 5.4. It should be noted here that the lime agent is more effective to decrease the swell percentage in Esenboğa Clay which is more plastic.

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on treated samples are marked on the charts in Figure 4.1 and Figure 4.2, swelling potentials can be evaluated for both clays. According to Seed et al. (1962) and Van der Merwe (1964) approaches, the swelling potential of the samples containing 9% lime, decreases low to medium. However, Dakshanamurthy and Raman's (1973) chart still gives high to very high swell potentials for 9% lime content for both clays. The results of oedometer swell tests in this study show that Dakshanamurthy and Raman's (1973) method, which only takes into account liquid limit, is quite conservative. The methods in which the clay fractions are taken into consideration seems more reliable. As a result, for both clays, Seed et al. (1962) and Van der Merwe (1964) approaches are more compatible with the results of oedometer swell tests performed in this study. It should be stated here that these charts are only for preliminary estimations. It is recommended to perform oedometer tests to make a final design.

Figure 5.5 below shows the net swell percentages with increasing lime content. This chart has been prepared based on a theoretical calculation in which net swell percentages are calculated with proportional to natural clay swell percentage (i.e. for Esenboğa Clay, natural clay is composed of 100% clay and its swell percentage has been measured as 35.5% in experiment, then with 1% lime, the clay content is 99%, thus the net swell percentage is calculated as 35.2%). Same evaluation is valid for Bilkent Clay (26.9% swell percentage of natural clay decreases to 26.7% net swell percentage for 1% lime content). Since Figure 5.5 is based on a proportional theoretical calculation, net swell percentages decrease consistently.

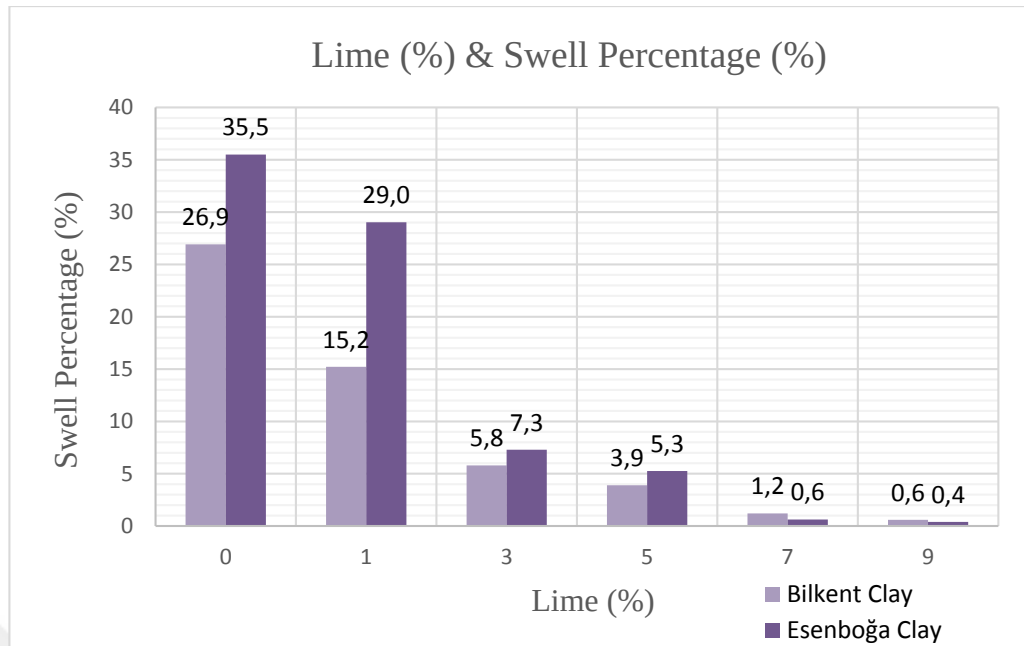


Figure 5.4 : Comparison of swell percentages with lime content.

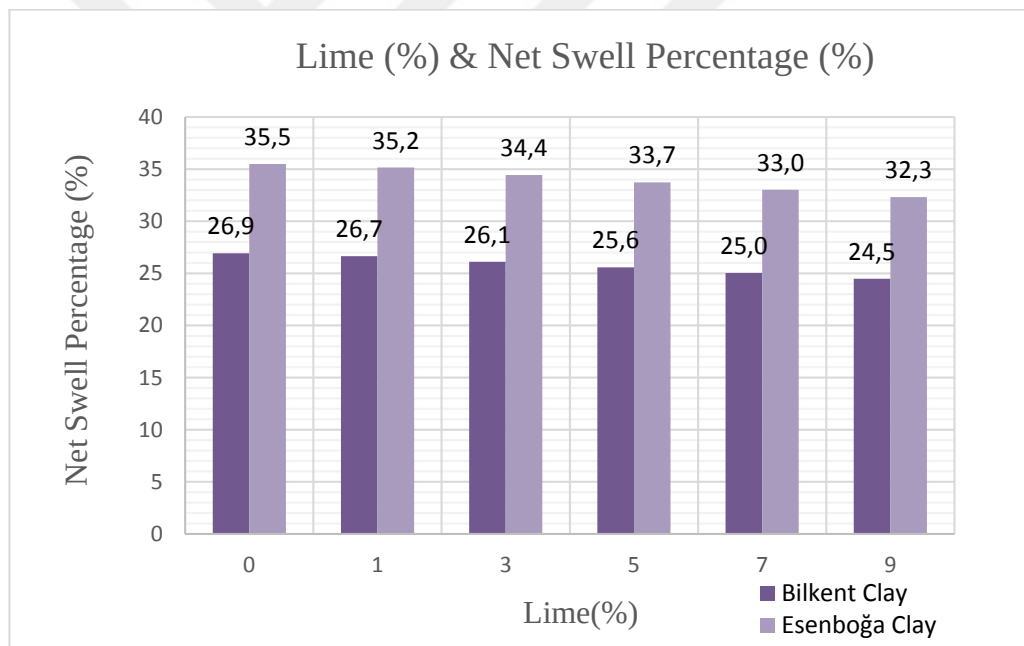


Figure 5.5 : Comparison of net swell percentages with lime content.

Figure 5.6 shows the effect of lime on swell percentage as lime content increases. This chart is simply the difference between the charts given in Figure 5.4 and 5.5. It should be noted here that the swell percentage decrease is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between lime and clay. Figure 5.6 shows only the latter effect. Thus, the effect of lime on swell percentage is maximum when the lime content increases from 1 to 3% for both Esenboğa and Bilkent Clays. The effect is moderate starting from 3% lime content.

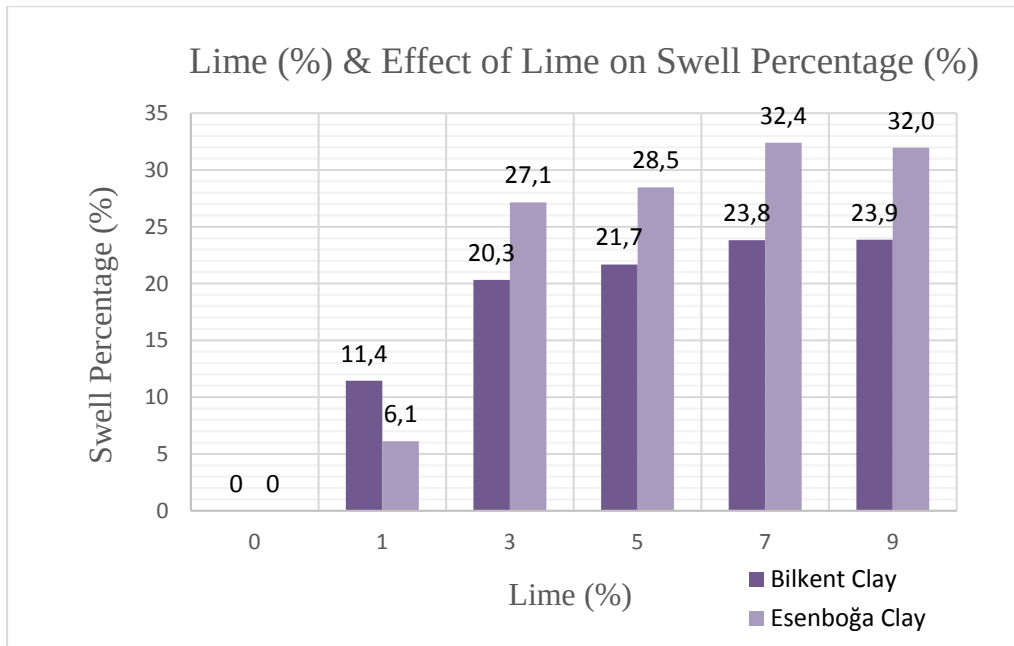


Figure 5.6 : Comparison of effect of lime on swell percentages.

The swell pressure decreased considerably at 7% to 9% lime contents as given in Figure 5.7 in accordance with the above evaluations. It is noted that the decrease in swell pressure is more pronounced for Esenboğa Clay. This result is compatible with the plasticity index and swell percentage results above.

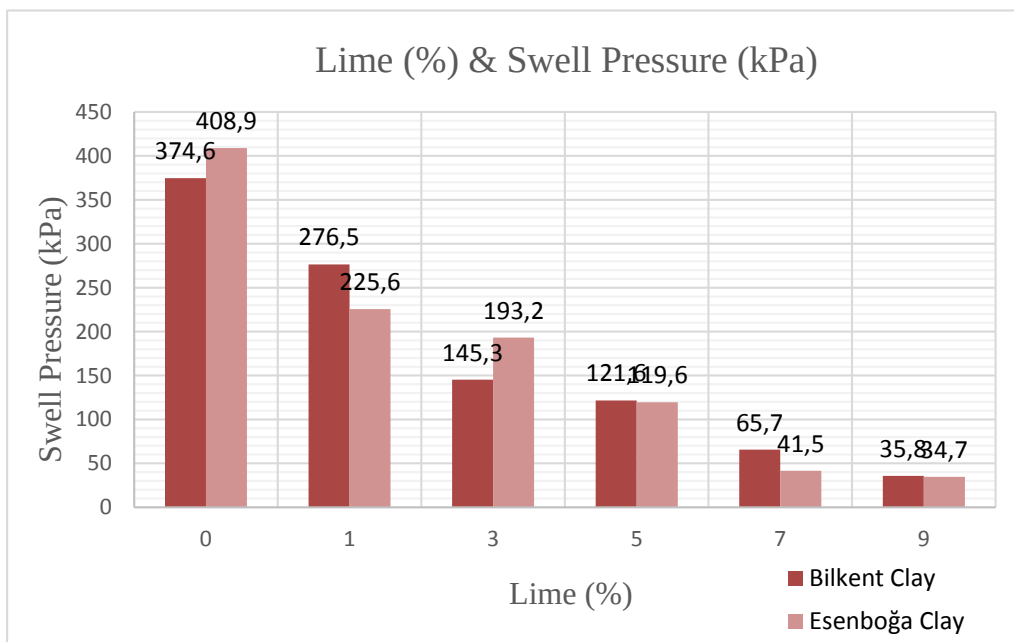


Figure 5.7 : Comparison of swell pressures with lime content.

Figure 5.8 and Figure 5.9 show the net swell pressure and the effect of lime on swell pressure with increasing lime content respectively. The explanations are similar to

ones given in swell percentage subject above. The effect of lime on swell pressure starts quickly when 1% lime is added to Esenboğa Clay as observed in Figure 5.9. The effect of lime on swell pressure is maximum for lime content increase from 1% to 3% in Bilkent Clay. This is compatible with the result observed in swell percentage subject. Effect of lime on swell pressure increases non-linearly as lime content increases for both clays.

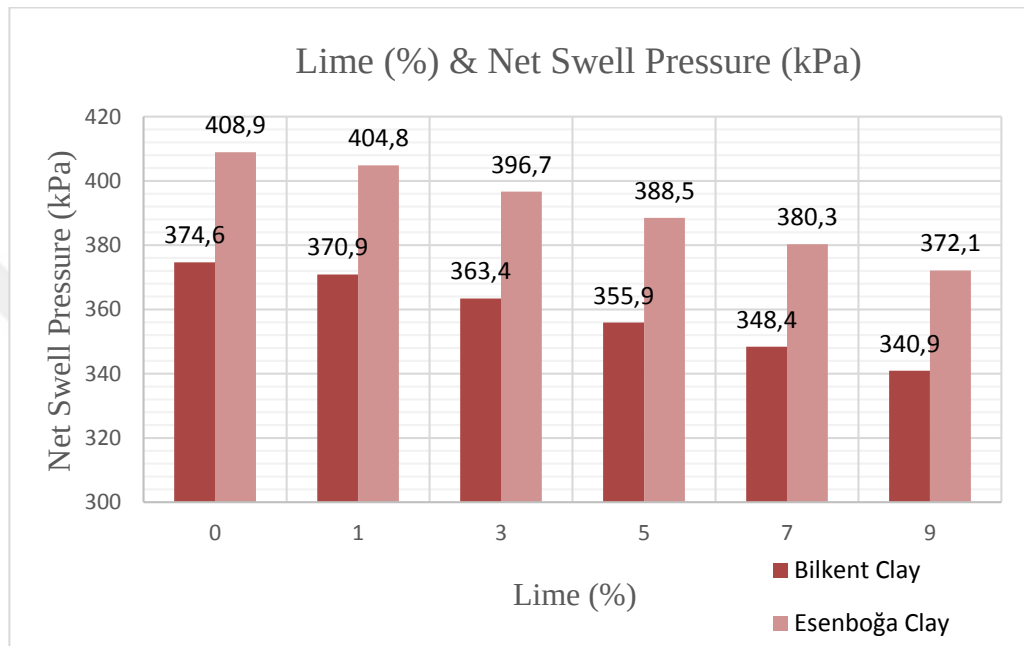


Figure 5.8 : Comparison of net swell pressures with lime content.

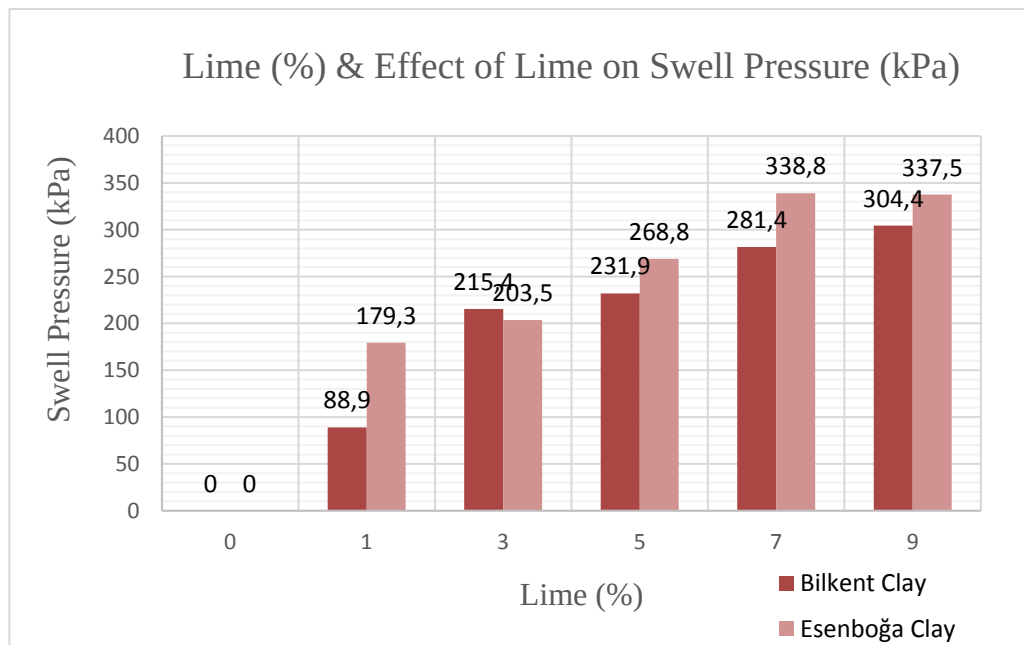


Figure 5.9 : Comparison of effect of lime on swell pressures.

5.1.3 Strength parameters

Figure 5.10 and Figure 5.11 give the results of unconfined compressive strengths depending on lime content increase for both non-soaked and soaked conditions, respectively. Unconfined compressive strength of the natural clay in the soaked condition is considerably lower than the one in non-soaked condition as expected for both Esenboğa and Bilkent Clays.

This behavior is also valid for treated samples up to 3% lime content, although the gap between q_u (non-soaked) and q_u (soaked) is significantly decreased at 3% lime content in Esenboğa Clay. The behavior is reversed at 5 and 7% lime contents. At 7% lime content especially, the unconfined compressive strength in soaked condition is about 68% more than that of the non-soaked condition. This reversal of behavior is attributed to considerable pozzolanic reactions in the long-term between lime and clay starting from 7% lime content in soaked condition. This effective lime percentage is compatible with the findings above for Esenboğa Clay.

If a more refined explanation for pozzolanic reactions is required, for the maximum reactivity between lime and clay, the pH value should be around 12.4 in the presence of water. Aluminum and silicium dissolve in a great potential from the clay mineral structure and as a result of this CSH (calcium silicate hydrate), CAH (calcium aluminate hydrate) and CASH (calcium aluminosilicate hydrate) cementitious compounds are formed. These reactions, known as pozzolanic reactions, are mainly curing-time dependent. In the soaked condition of unconfined compression tests water can reach all the unsaturated voids within the clay samples. Moreover, there is more time in soaked condition to complete the swelling. Thus, after a certain lime content the strengths in soaked condition exceeds the strengths of the one non-soaked.

For Bilkent Clay, it is noticed that the unconfined compressive strengths are greater in non-soaked condition up to 7% lime content. Starting from 7%, the strengths are greater in soaked condition due to pozzolanic reactions which are very dominant in Bilkent Clay (i.e. unconfined compressive strengths increase very significantly as lime content increases for soaked condition).

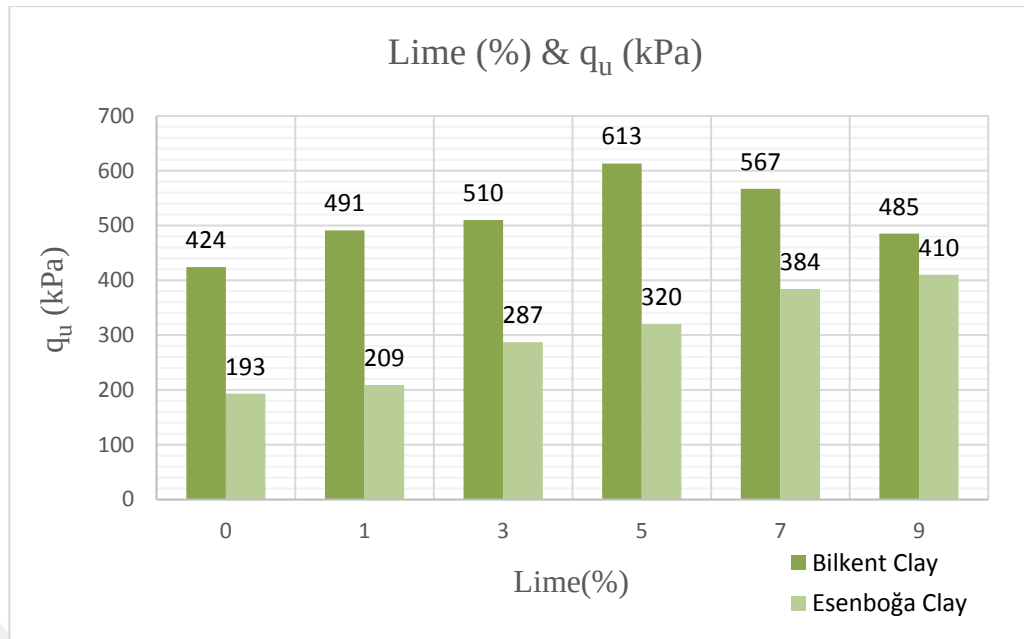


Figure 5.10 : Comparison of unconfined compressive strengths (q_u non-soaked) with lime content.

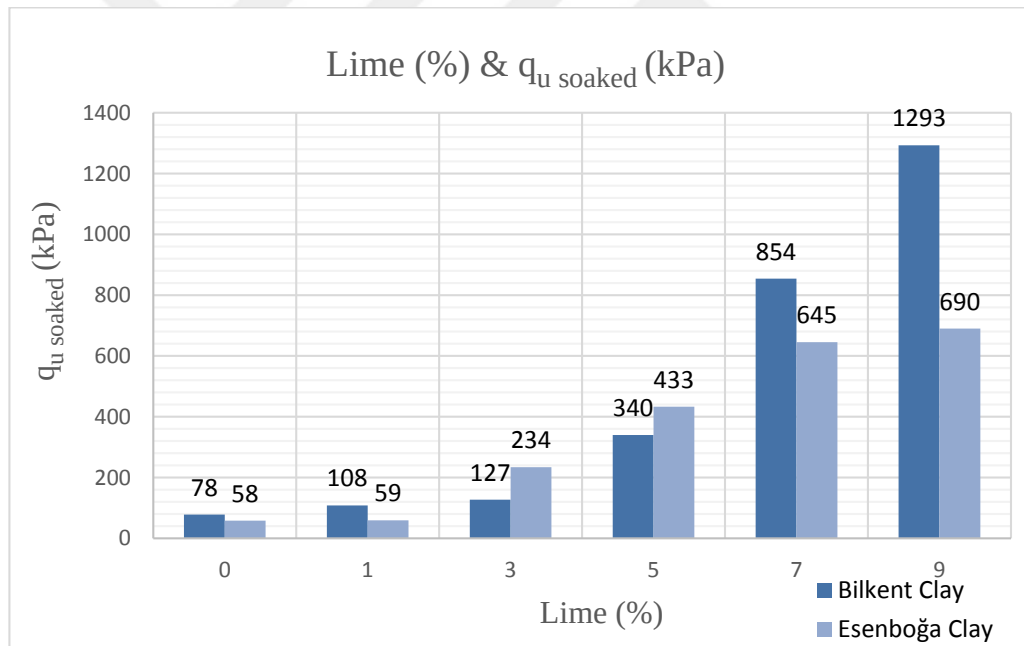


Figure 5.11 : Comparison of unconfined compressive strengths for soaked condition (q_u soaked) with lime content.

5.1.4 Compression parameters

In this part, effect of lime stabilization on compression parameters for Esenboğa and Bilkent Clays are compared.

The variation of compression and expansion (recompression) index values are given in Figure 4.14, Figure 4.15 for Esenboğa Clay, in Figure 4.45 and Figure 4.46 for

Bilkent Clay in Chapter 4. Both compression and recompression curves are quite steep in natural state for both clays indicating high compressibility and expansion capacity. For Esenboğa Clay, it is seen that 1% lime content cannot change compression and expansion indices. Starting from 5% lime content the compression index is markedly reduced. The effective lime content to reduce the expansion index, on the other hand, is 3%. It is believed that the flocculation process causes different void ratio values in lime treated samples. The standard of sample preparation process may also result in different void ratio values. Thus, the optimum lime content as far as the compression properties of the Esenboğa Clay are concerned, can be accepted as 3% to 5%. The similar evaluations are valid for Bilkent Clay at which 1% and 3% lime contents are ineffective again as far as the compressibility characteristics are concerned. The compressibility and swelling properties are improved starting from 5% lime content. It is realized, on the other hand, that void ratio values are about 10% larger in 7% lime content than the ones in 5% lime content. This may be due to flocculation process in lime stabilized samples.

As a result, the optimum lime content can be accepted as 5% for both clays for compressibility and recompressibility properties. It is revealed that lime is an effective agent for the clays studied in this research with regard to compressibility.

5.2 Comparison of Fly Ash-Treated Samples of Esenboğa and Bilkent Clays

An extremely highly plastic clay type of soil, which was taken from clay deposits from the Akyurt district of Ankara, near Esenboğa Airport and a very highly plastic clay type of soil taken from clay deposits of the Bilkent Atatürk Hospital district of Ankara, near Ankara-Eskişehir State Highway have been mixed with fly ash at different percentages of dry weight of the soil (5%, 10%, 15%, 20% and 25%). Samples have been prepared at optimum water content and maximum dry density by means of Harvard miniature compaction test (Wilson, 1950, ASTM D698- 07). Grain size distribution, Atterberg limits, strength and swelling properties of the fly ash treated clay samples have been determined at each fly ash content which is increased step by step. The effect of fly ash stabilization on Atterberg limits, strength and swelling characteristics are given in Figure 5.12 to Figure 5.22.

5.2.1 Index parameters

When the Figure 5.12 and Figure 5.13 are examined it is seen that liquid limit values decrease regularly for Bilkent Clay (i.e. from 109% of natural clay to 83% of 25% fly ash content). On the other hand, liquid limit increases significantly by adding 5% fly ash and then decreases steadily up to 25% fly ash content for Esenboğa Clay. However, the natural clay's liquid limit (LL) which is 135% could hardly be caught between 20-25% fly ash content. These observations are more or less valid for plasticity index. It should be noted here that fly ash agent is not effective on liquid limit (LL) and plasticity index (PI).

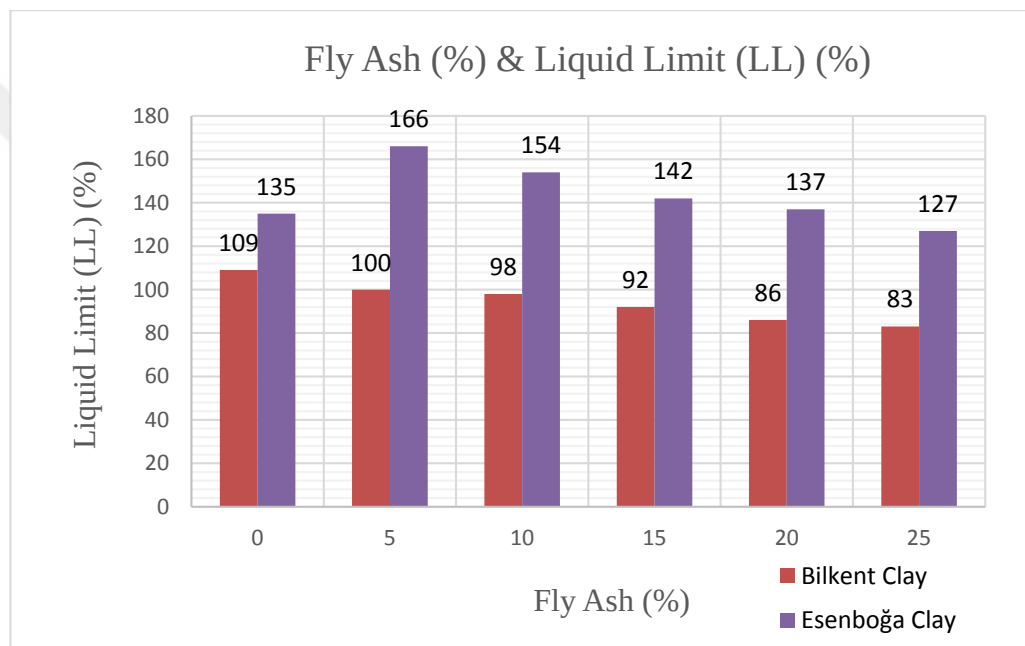


Figure 5.12 : Comparison of liquid limits (LL) with fly ash content.

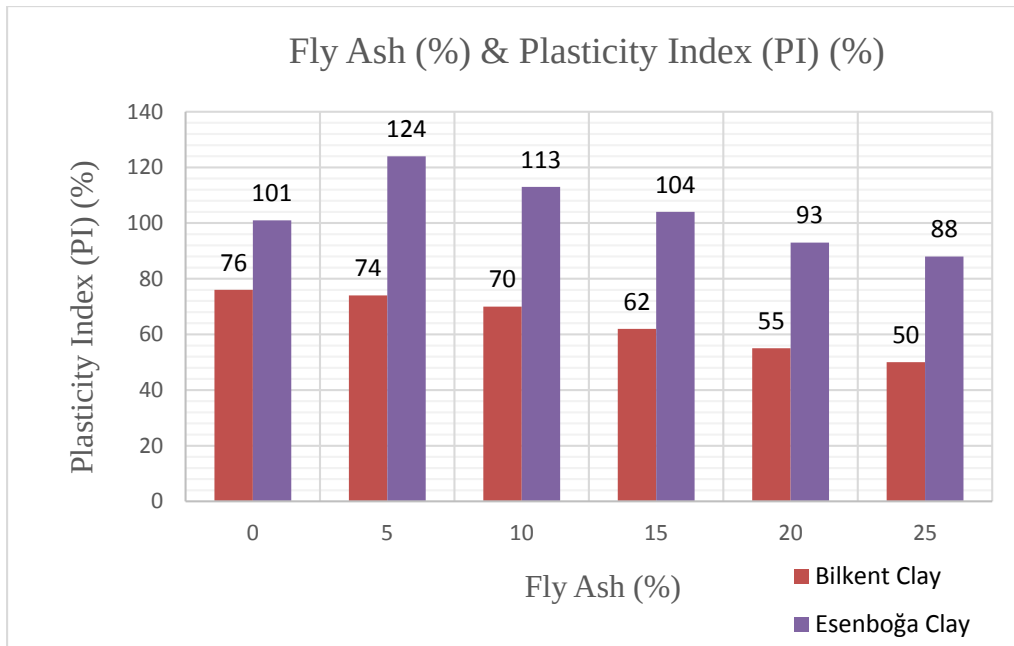


Figure 5.13 : Comparison of plasticity indices (PI) with fly ash content.

Shrinkage limit increases as the fly ash content increases as given in Figure 5.14. The soil exhibits the highest value of shrinkage limit at 25% fly ash content for Esenboğa and Bilkent Clays. Firstly, there is a sudden increase in shrinkage limit (SL) when 5% fly ash is added. Then it is almost same up to 15% fly ash content. The soil exhibits the highest value of shrinkage limit (SL) at 25% fly ash content.

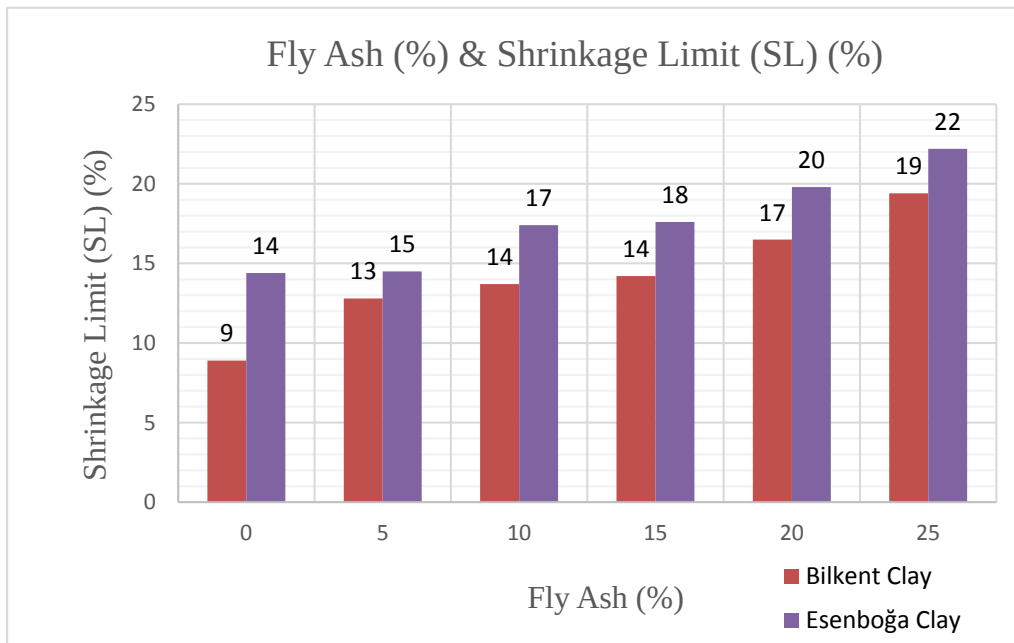


Figure 5.14 : Comparison of shrinkage limits (SL) with fly ash content.

5.2.2 Swell parameters

The allowable swell percentage for surficial structures is generally accepted as 0.5% as given in Highway Standards (KGM, 2005). However, the maximum fly ash content in this study (i.e. 25%) is not adequate to obtain allowable swell percentages for surficial structures, as shown in Figure 5.15. There are significant reductions in swell percentages in Esenboğa Clay. Nevertheless, at the highest amount of fly ash (i.e. 25%) the swell percentage is 8.6% which is still very high. In Bilkent Clay there are also remarkable reductions in swell percentage as fly ash content increases. However, at the maximum fly ash content (i.e. 25%) the allowable swell percentage could not be obtained in Bilkent Clay as in Esenboğa Clay.

When the parameters such as Atterberg limits, clay fraction and activity values obtained from the experiments conducted on treated samples are marked on the charts in Figure 4.1 and Figure 4.2, swelling potentials can be evaluated. According to Seed et al. (1962) and Van der Merwe (1964) approaches, the swelling potential of the samples containing 25% fly ash, decreases low to medium. However, Dakshanamurthy and Raman's (1973) chart still gives high to very high swell potentials for 25% fly ash content for Bilkent Clay. The results of oedometer swell tests in this study show that Dakshanamurthy and Raman's (1973) method, which only takes into account liquid limit, is quite conservative. The methods in which the clay fractions are taken into consideration seems more reliable. As a result, Seed et al. (1962) and Van der Merwe (1964) approaches are more compatible with the results of oedometer swell tests performed in this study. According to the charts it should be noted here that fly ash is more effective in Bilkent Clay as an agent to decrease the swell potential than Esenboğa Clay. In Esenboğa Clay the swell potential reductions are almost negligible and there are no changes in classifications in none of the charts. Thus, it can be concluded that these charts are only for preliminary estimations. It is recommended to perform oedometer tests to make a final design.

Figure 5.16 below shows the net swell percentages with increasing fly ash content. This chart has been prepared based on a theoretical calculation in which net swell percentages are calculated with proportional to natural clay swell percentage (i.e. for Esenboğa Clay, natural clay is composed of 100% clay and its swell percentage has been measured as 35.5% in experiment, then with 5% fly ash, the clay content is 95%,

thus the net swell percentage is calculated as 33.7%). Same evaluation is valid for Bilkent Clay (26.9% swell percentage of natural clay decreases to 25.6% net swell percentage for 5% fly ash content). Since Figure 5.16 is based on a proportional theoretical calculation, net swell percentages decrease consistently.

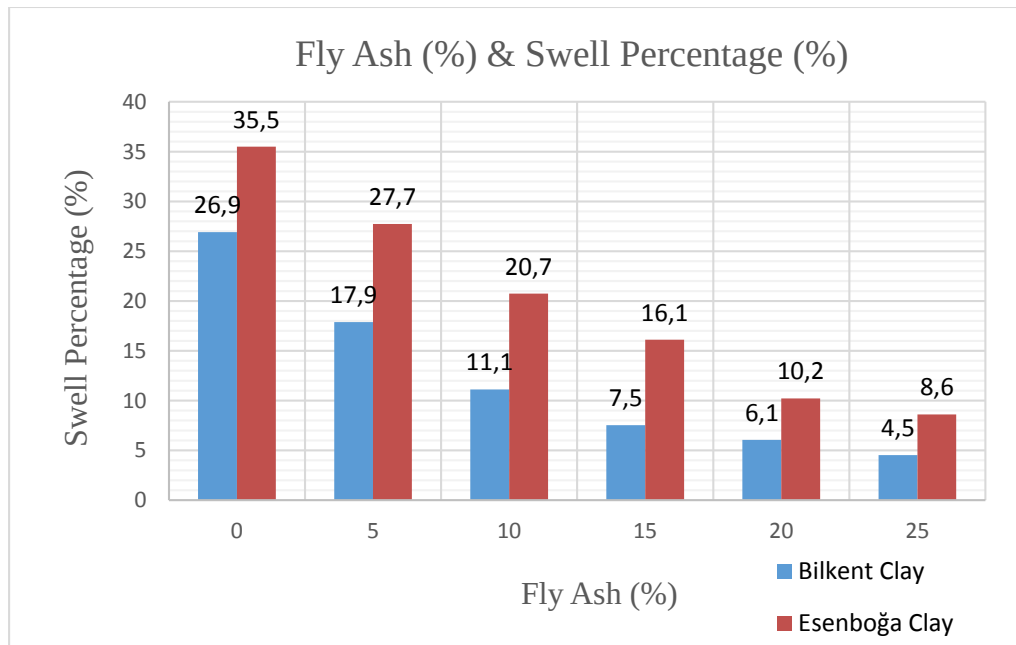


Figure 5.15 : Comparison of swell percentages with fly ash content.

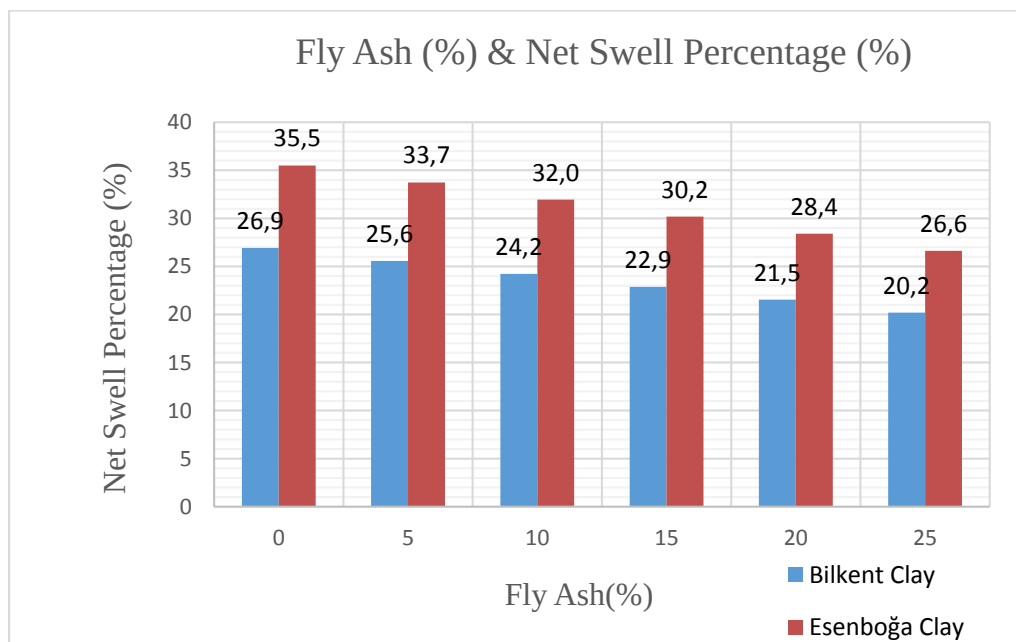


Figure 5.16 : Comparison of net swell percentages with fly ash content.

Figure 5.17 shows the effect of fly ash on swell percentage as fly ash content increases. This chart is simply the difference between the charts given in Figure 5.15 and 5.16. It

should be noted here that the swell percentage decrease is composed of two parts; one is due to clay content decrease and the other one is due to chemical reactions between fly ash and clay. Figure 5.17 shows only the latter effect. Thus, the effect of fly ash on swell percentage is maximum when the fly ash content increases from 5 to 10% for both Esenboğa and Bilkent Clays. But, the increases are minimal starting from 15% fly ash content for Bilkent Clay. In Esenboğa Clay, the effect of fly ash on swell percentage is worthy of note up to 20% fly ash content. There is practically no effect between 20% and 25% fly ash contents in Esenboğa Clay.

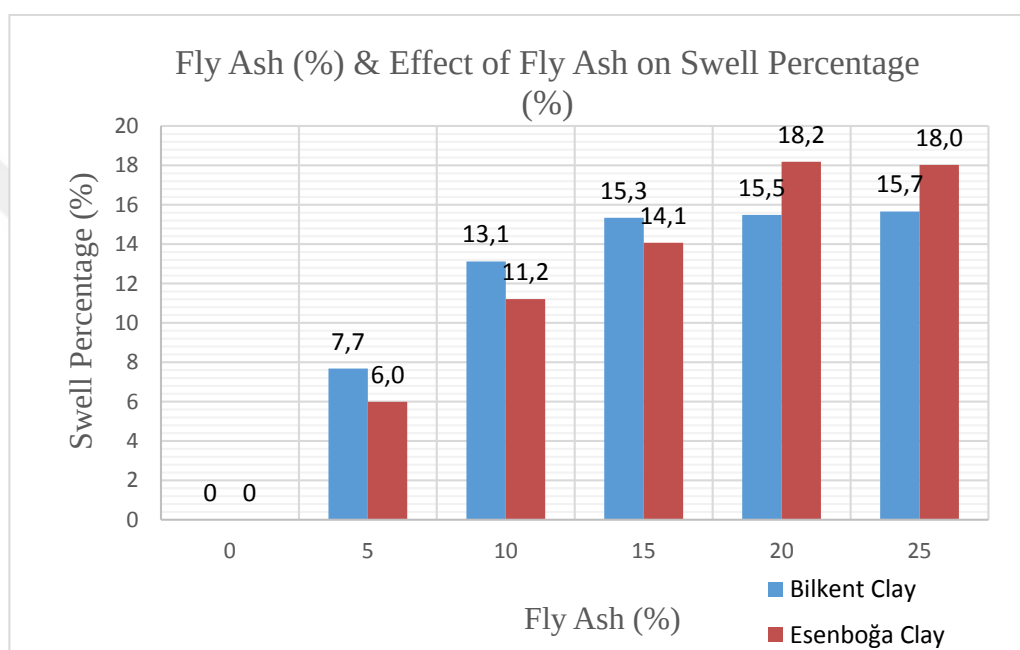


Figure 5.17 : Comparison of effect of fly ash on swell percentages.

The swell pressure decreased considerably at 5% to 10% fly ash contents for Bilkent Clay, as given in Figure 5.18 in accordance with the above swell percentage evaluations. However, the remarkable decrease is between 15% and 20% for Esenboğa Clay. It is noted that the decrease in swell pressure is more pronounced for Bilkent Clay.

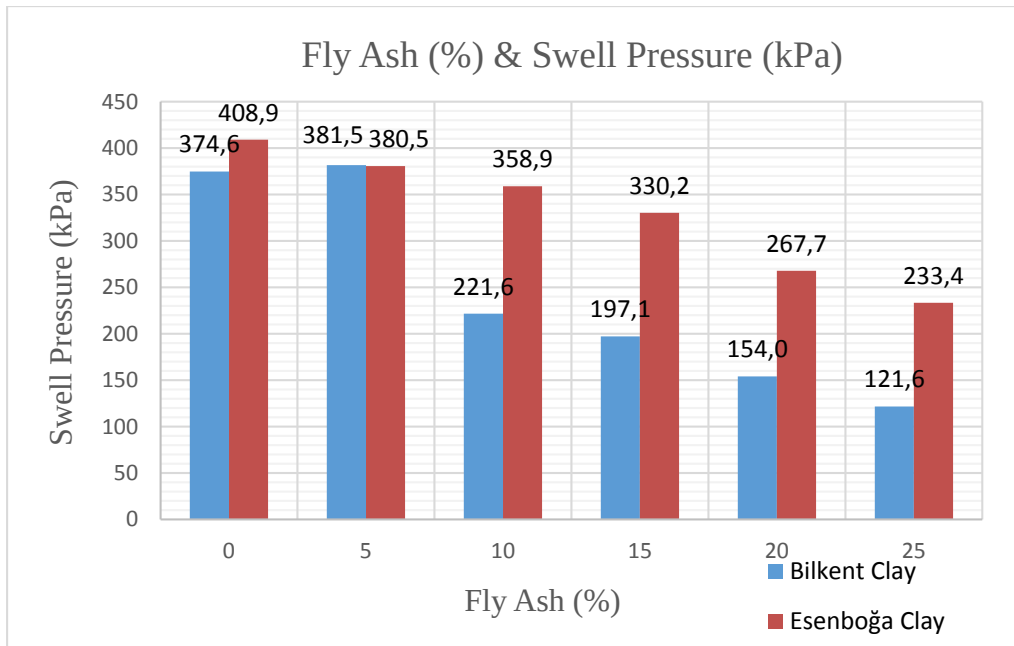


Figure 5.18 : Comparison of swell pressures with fly ash content.

Figure 5.19 and Figure 5.20 show the net swell pressure and the effect of fly ash on swell pressure with increasing fly ash content respectively. The explanations are similar to ones given in swell percentage subject above. The effect of fly ash on swell pressure is maximum between 5% and 10% fly ash contents in Bilkent Clay as observed in Figure 5.18. The effect of fly ash on swell pressure is maximum for fly ash content increase from 15% to 20% in Esenboğa Clay. Effect of fly ash on swell pressure increases non-linearly as fly ash content increases for both clays.

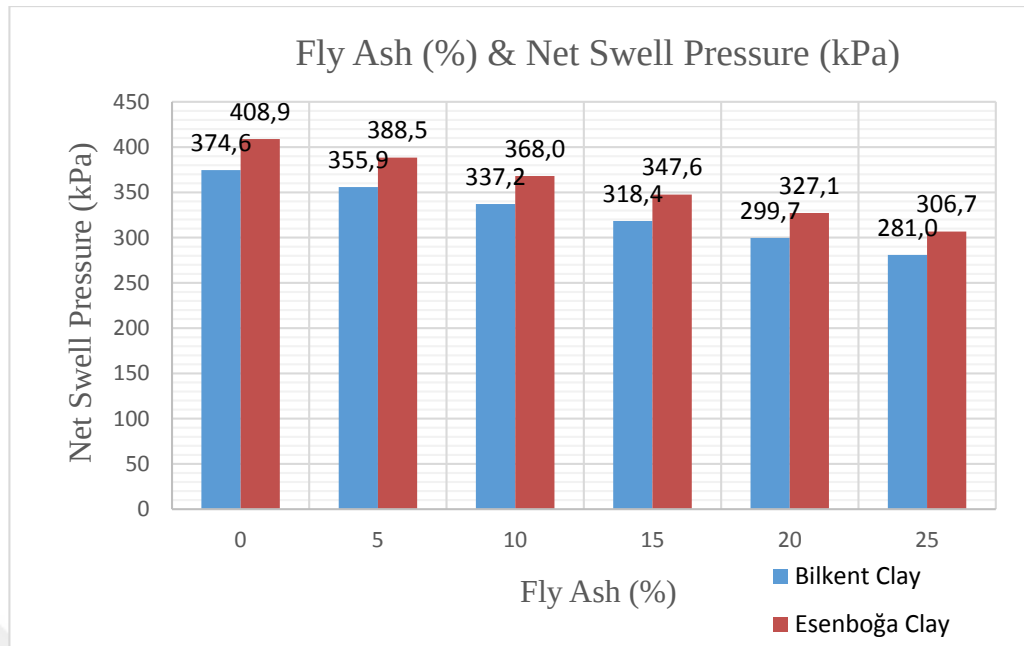


Figure 5.19 : Comparison of net swell pressures with fly ash content.

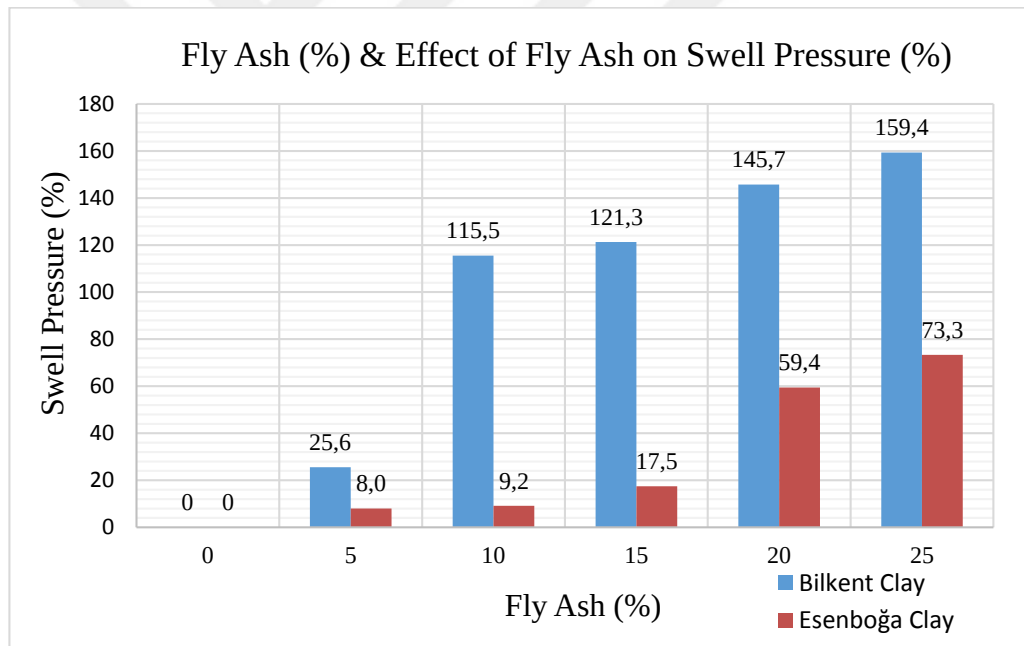


Figure 5.20 : Comparison of effect of fly ash on swell pressures.

5.2.3 Strength parameters

Figure 5.21 and Figure 5.22 give the results of unconfined compressive strengths depending on fly ash content increase for both non-soaked and soaked conditions, respectively. Unconfined compressive strength of the natural clay in the soaked condition is considerably lower than the one in non-soaked condition as expected for

both Esenboğa and Bilkent Clays. This behavior is generally valid for treated samples as well.

For non-soaked condition in Bilkent Clay, it is noticed that first there is a decrease in unconfined compressive strength when 5% fly ash is added. After that, the unconfined compressive strength increases continuously up to 25% fly ash content. For non-soaked condition in Esenboğa Clay on the other hand, there is an optimum fly ash content at which the unconfined compressive strength is maximum (i.e. at 15% fly ash content).

The above behavior is also valid in both clays for soaked condition. When the first 5% fly ash is added there is a slight decrease in unconfined compressive strength in Bilkent Clay. Then it increases consistently. In Esenboğa Clay for soaked condition, there is again an optimum fly ash content which is 15% same as in Bilkent Clay.

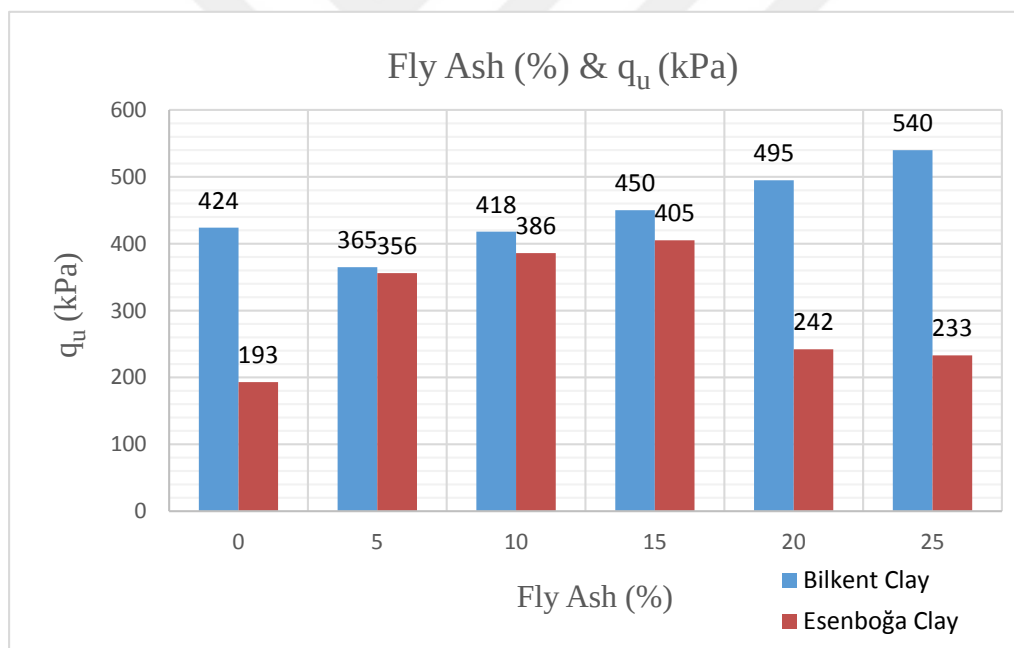


Figure 5.21 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) with fly ash content.

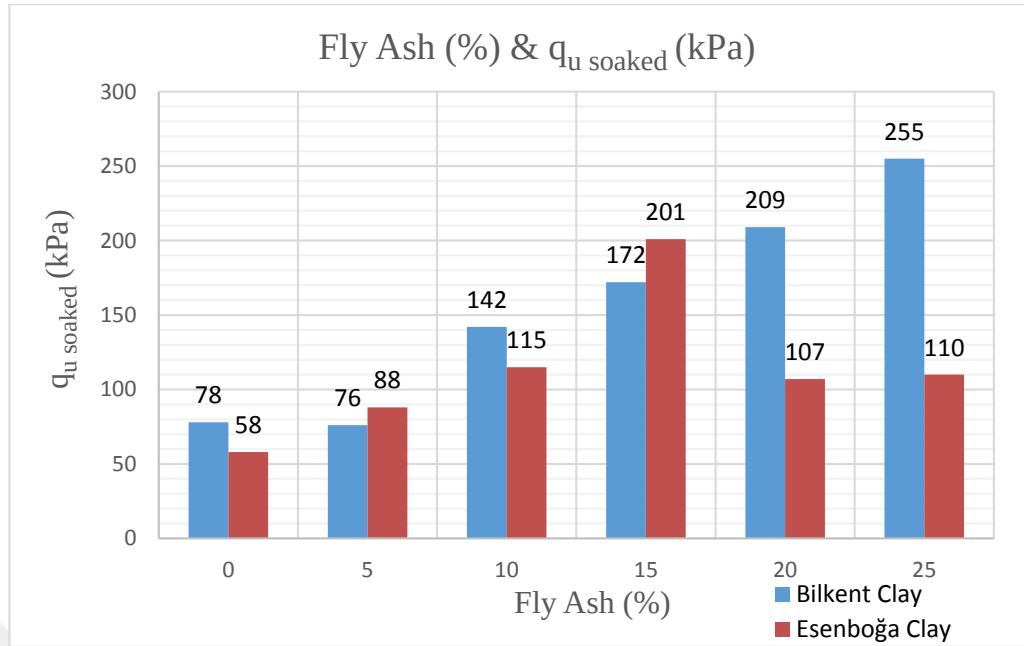


Figure 5.22 : Comparison of unconfined compressive strengths of soaked samples (q_u soaked) with fly ash content.

5.2.4 Compression parameters

In this part, effect of fly ash stabilization on compression parameters for Esenboğa and Bilkent Clays are compared.

The variation of compression and expansion (recompression) index values are given in Figure 4.56, Figure 4.57 for Esenboğa Clay, in Figure 4.67 and Figure 4.68 for Bilkent Clay in Chapter 4. Both compression and recompression curves are quite steep in natural state for both clays indicating high compressibility and expansion capacity.

For Esenboğa Clay, the compressibility and expansion properties are improved slightly starting from 10% fly ash content as noticed in Figure 4.56. However, the compressibility of the fly ash-treated samples starts to increase again in 20% fly ash content. This means that the amount of fly ash has a negative effect at about 20% contents. On the other hand, the expansion index has started to decrease in 10% fly ash content and remained almost unchanged afterwards. As a result, it can be said that the effect of fly ash on the compressibility characteristics of Esenboğa Clay is quite unstable. The same instability is valid for Bilkent Clay also.

As a result, the effect of fly ash on the compressibility characteristics of both Esenboğa and Bilkent Clays which are extremely and very highly plastic is unstable and can be accepted as ineffective.

5.3 Comparison of Resilient Modulus Test

5.3.1 Resilient modulus

Mean resilient modulus values for both types of clays are given in the same graphs below.

The results for the condition without cure is given in Figure 5.23. It is noted that the mean resilient modulus value increases steadily with increasing lime content for Bilkent Clay. However, there is almost no increase in mean resilient modulus value after 3% lime content for Esenboğa Clay.

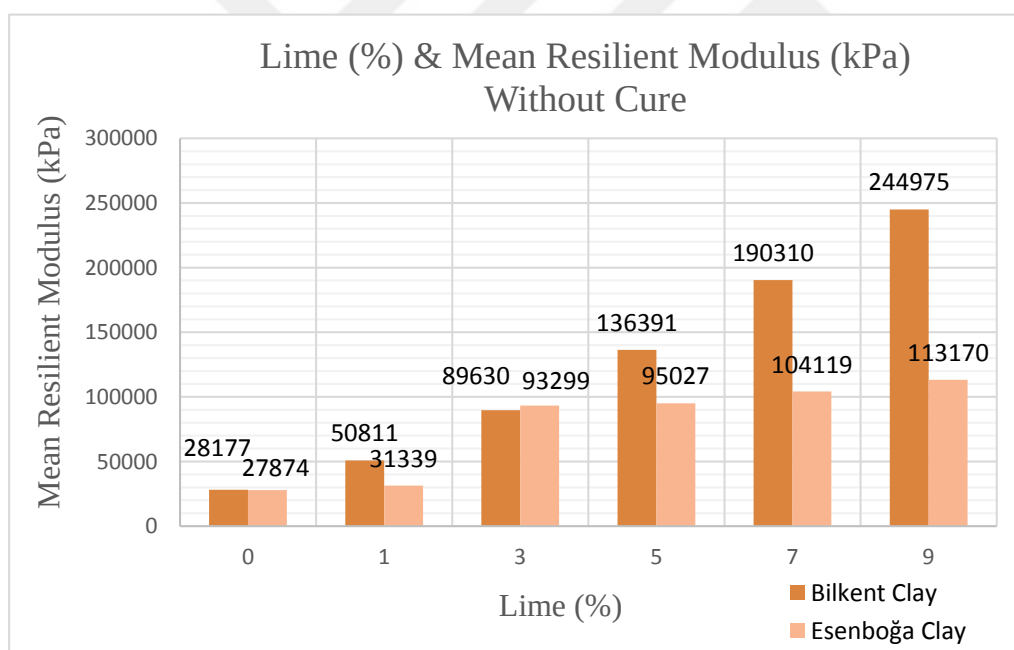


Figure 5.23 : Comparison of mean resilient moduli (M_r) values with lime content without cure.

The mean resilient modulus values for 7-days cure period are given in Figure 5.24. The mean resilient modulus values increase with increasing lime content for both clays. Although the starting values of mean resilient modulus are almost same for both clays, it is seen that the lime agent is more effective in Bilkent Clay than in Esenboğa Clay for 7-days cure period.

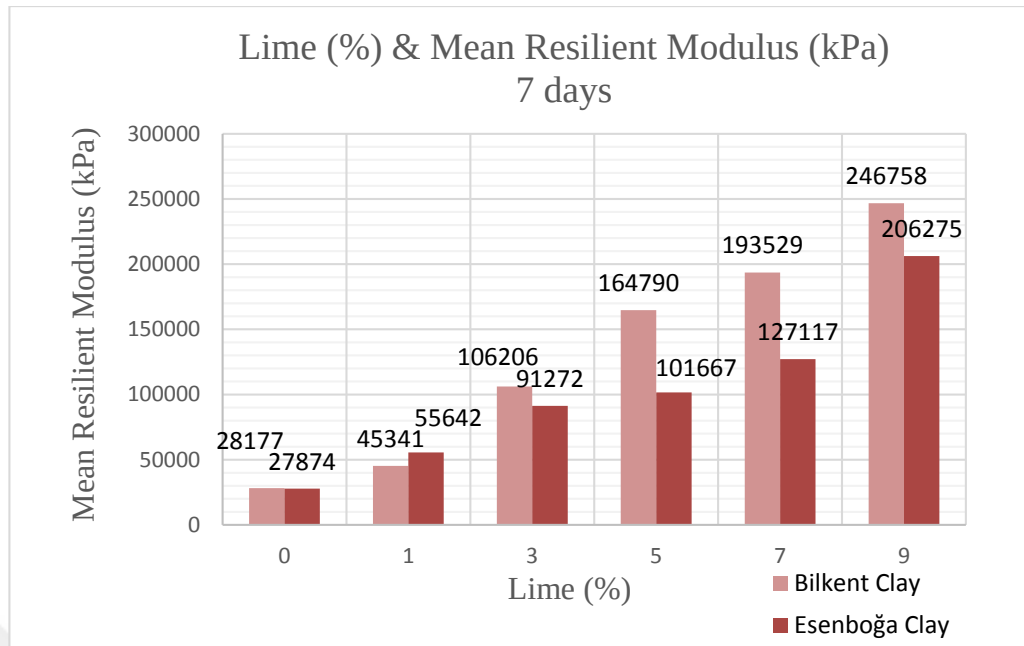


Figure 5.24 : Comparison of mean resilient moduli (M_r) values with lime content for 7 days cure period.

The mean resilient modulus values for 28-days cure period are given in Figure 5.25. The moduli increase steadily for both clays. The increases in Bilkent Clay are more remarkable again. 7% lime content seems quite effective in Bilkent Clay.

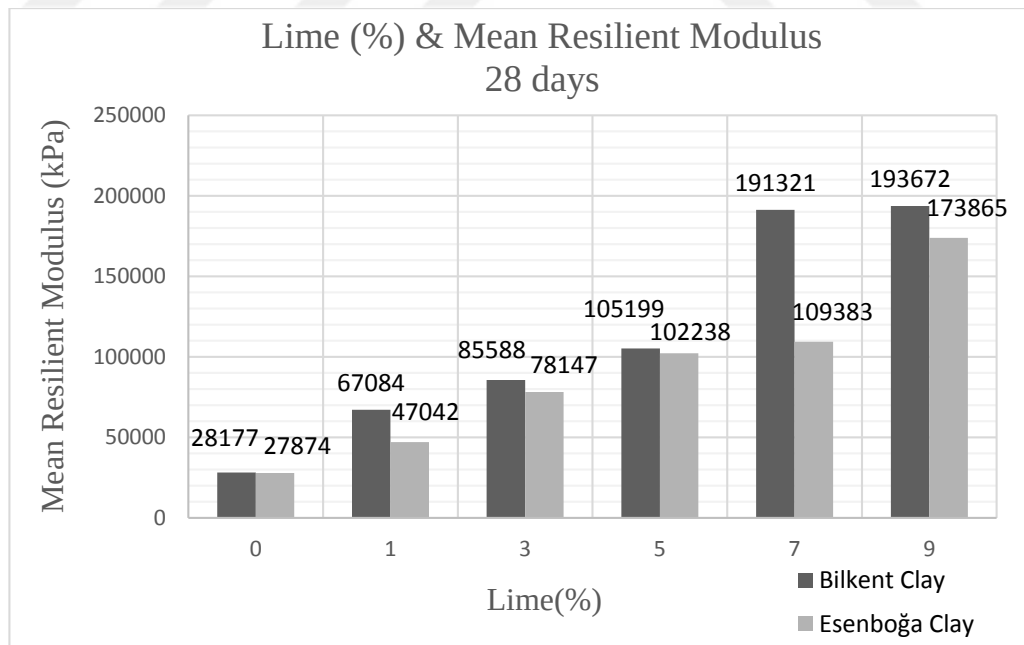


Figure 5.25 : Comparison of mean resilient moduli (M_r) values with lime content for 28 days cure period.

Figure 5.26 and Figure 5.27 give the the mean resilient modulus values for 56 days and 90 days respectively. The moduli increase continuously with increasing lime

content for both clays. The lime agent is more effective in Bilkent Clay. It is observed that 7% lime content is quite effective here again.

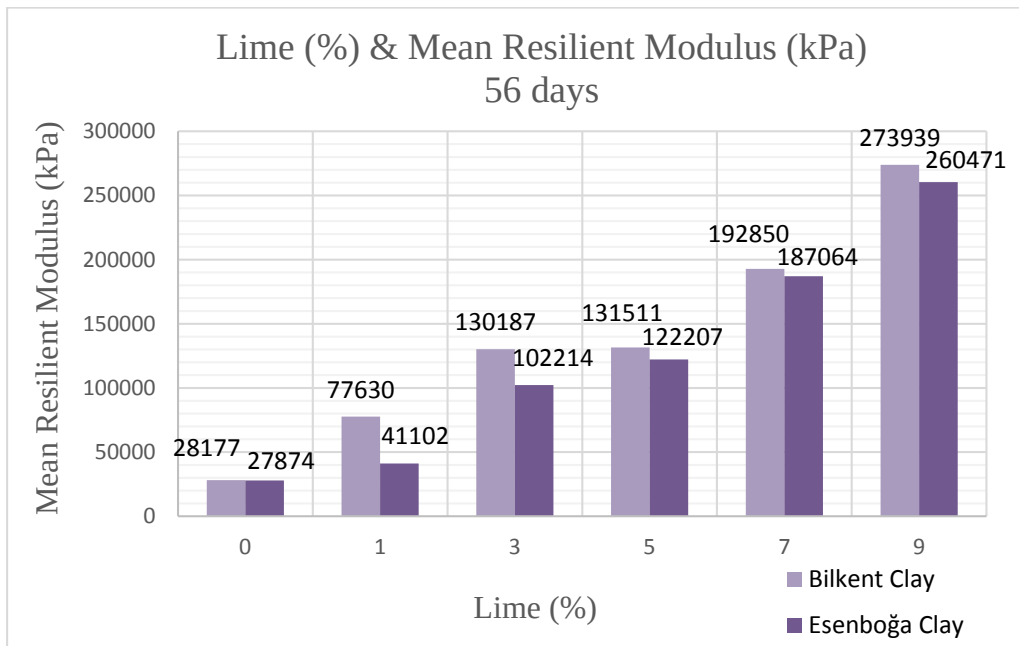


Figure 5.26 : Comparison of mean resilient moduli (M_r) values with lime content for 56 days cure period.

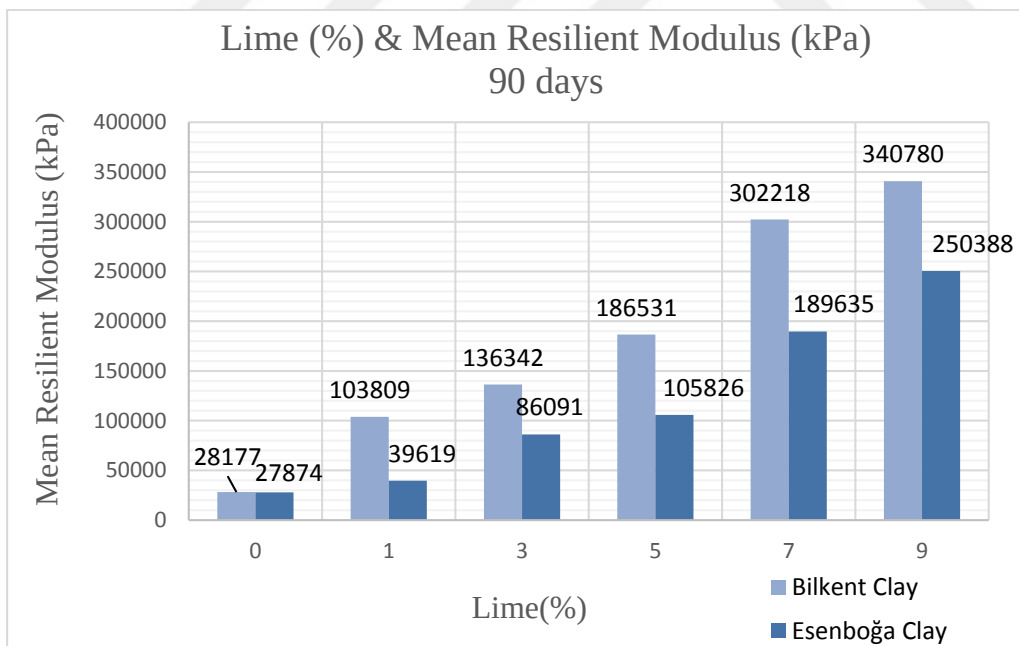


Figure 5.27 : Comparison of mean resilient moduli (M_r) values with lime content for 90 days cure period.

All results are given together in Figure 5.28 and 5.29 below for both clays. As a general trend, as lime content and curing period increase the mean resilient modulus values increase. It is observed that the lime treatment process is more effective in Bilkent

Clay than in Esenboğa Clay. The mean resilient modulus value is about 200.000-300.000 kPa especially for long cure periods (56 and 90 days) when the lime content is 7% and 9% for Bilkent Clay. The same moduli are about 180.000-250.000 kPa for Esenboğa Clay. Moreover, the treatment process starts to be more effective in Bilkent Clay starting from the beginning at 1% lime content. The mean resilient modulus values reach about 130.000 kPa at 3% lime content for Bilkent Clay. This number is about 90.000 kPa for Esenboğa Clay.

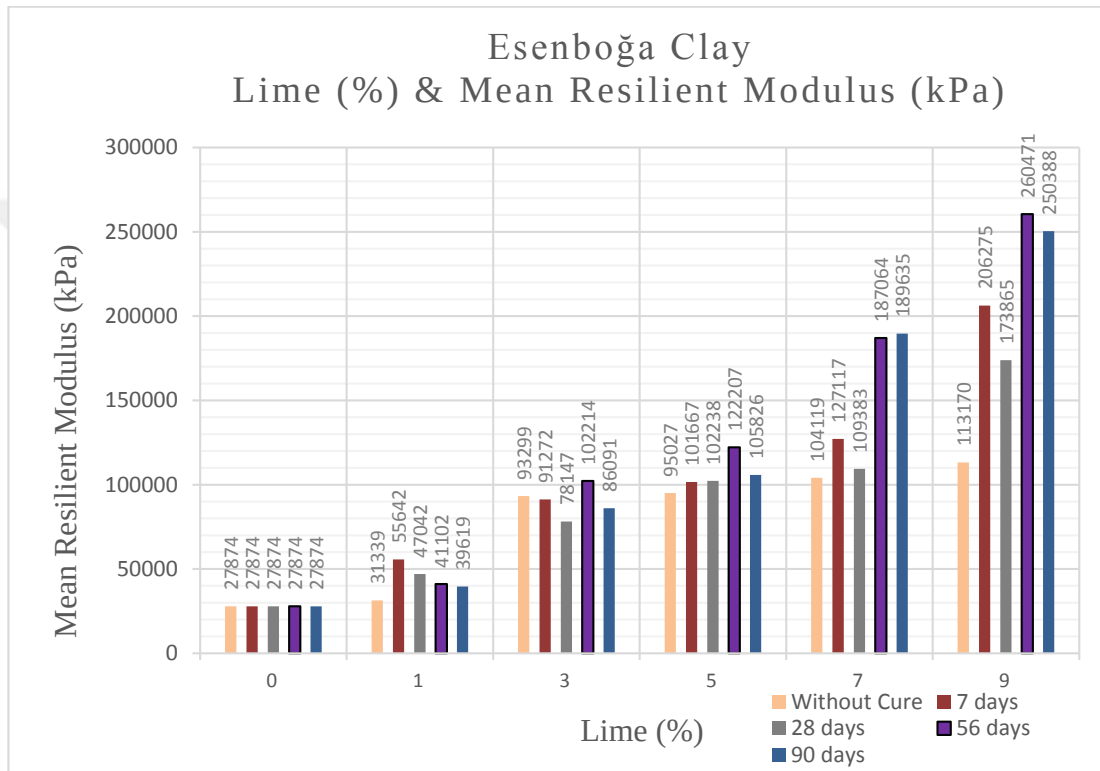


Figure 5.28 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Esenboğa Clay.

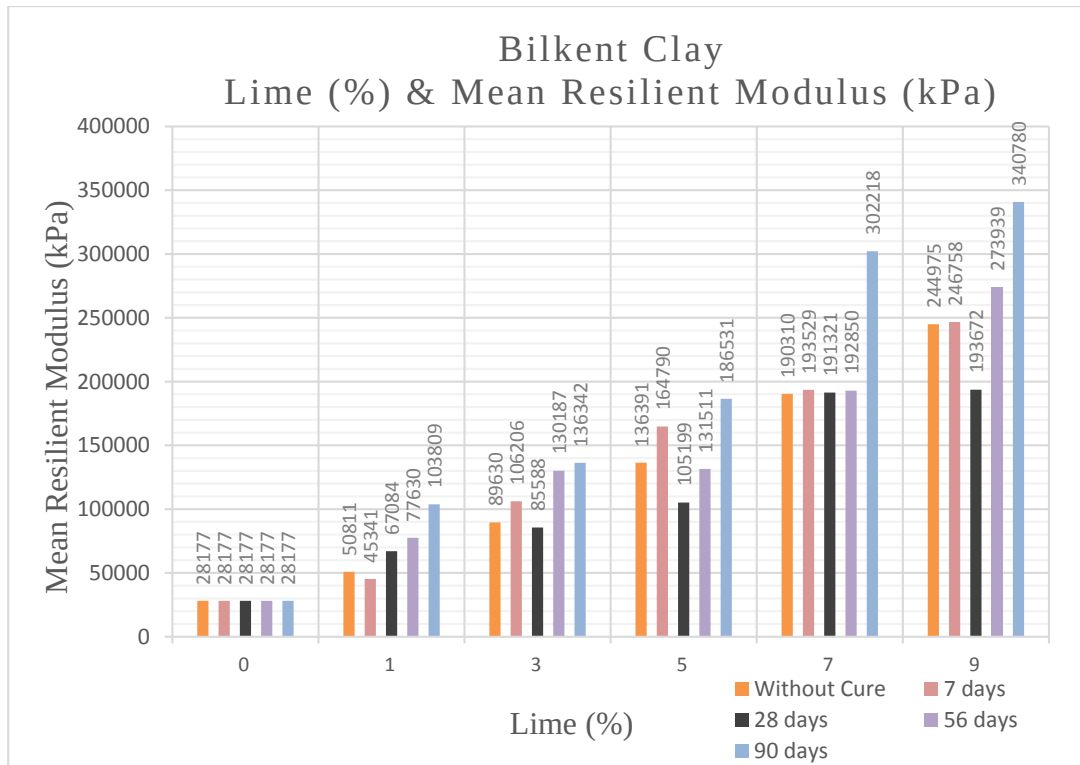


Figure 5.29 : Mean resilient modulus (M_r) values with lime content for 7, 28, 56 and 90 days cure periods for Bilkent Clay.

5.4 Comparison of Methylene Blue Test (MBT)

The lime versus MBV relationship is given in Figure 5.30 below for both Esenboğa and Bilkent Clays. Since the Esenboğa Clay contains smectite minerals and also the clay fraction is more than that of the Bilkent Clay, the MBV values of the Esenboğa Clay is quite higher than that of the Bilkent Clay (i.e. about 5 times larger).

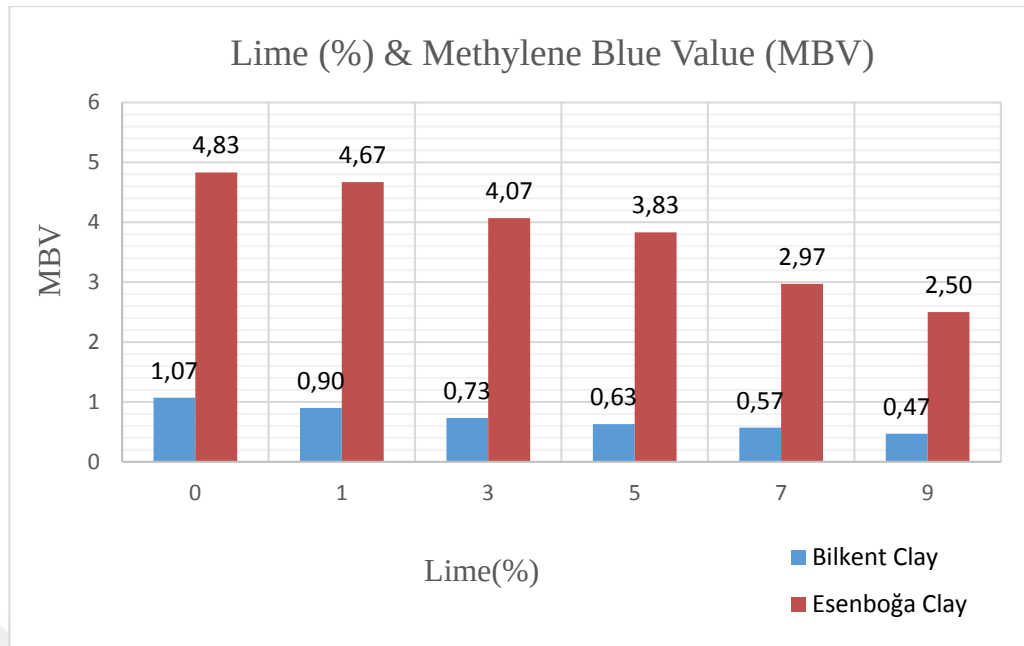


Figure 5.30 : Comparison of methylene blue value (MBV) with lime content.

The activity values with lime are given in Figure 5.31. According to Lautrin (1989) classification which is based on the original definition by Skempton, both of the natural clays are in the range of “active soils”. After treatment, on the other hand, both clays are in the range of “normal clays” with the effect of lime treatment (i.e. activity values are decreased from 1.50~1.70 to 0.89~1.20 in both clays).

As a result, the above conclusions are quite compatible with the decrease of swell potential due to lime treatment. As the swell pressure and swell percentages decrease with the increase in lime content, MBV and activity decrease as well.

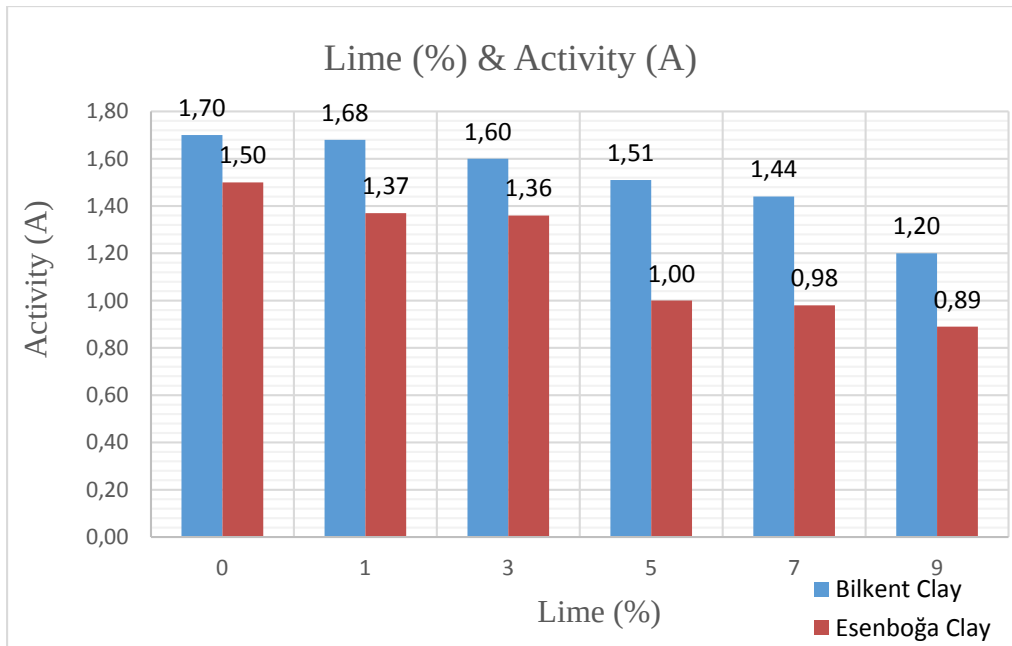


Figure 5.31 : Comparison of activity (A) with lime content.

5.5 Comparison of Tests Conducted after Resilient Modulus Test on RM Test Specimens

The unconfined compression tests have been conducted on the samples which were before tested in the resilient modulus apparatus. These results have been presented in Chapter 4 for each clay separately. The results for both clays in this case are given in the following figures (From Figure 5.32 to Figure 5.36).

5.5.1 Unconfined compression test

The strength values without cure are given in Figure 5.32 for both clays. The behaviors of the clays without cure are quite similar. There is an optimum stage at 7% lime content in both clays.

The unconfined compressive strengths of the cured samples are given in Figure 5.33 to Figure 5.36. Since the initial untreated strength of the Bilkent Clay is quite bigger than the one in Esenboğa Clay, the absolute values of the unconfined compressive strength of the Bilkent Clay seem larger in these figures. However, this does not mean that the treatment process is more effective in Bilkent Clay. On the contrary, the rate of increase in strength values of the Esenboğa Clay is more than the rate in Bilkent Clay. The strength values are more than the three times the value of untreated sample

at only 3% lime content in Esenboğa Clay. On the other hand, similar strength increases could be obtained at about 7% lime content in Bilkent Clay. Thus, it can be concluded that the Esenboğa Clay gives more effective reactions as far as the unconfined compressive strengths after the resilient modulus tests are concerned. This is especially observed as cure period increases in Esenboğa Clay.

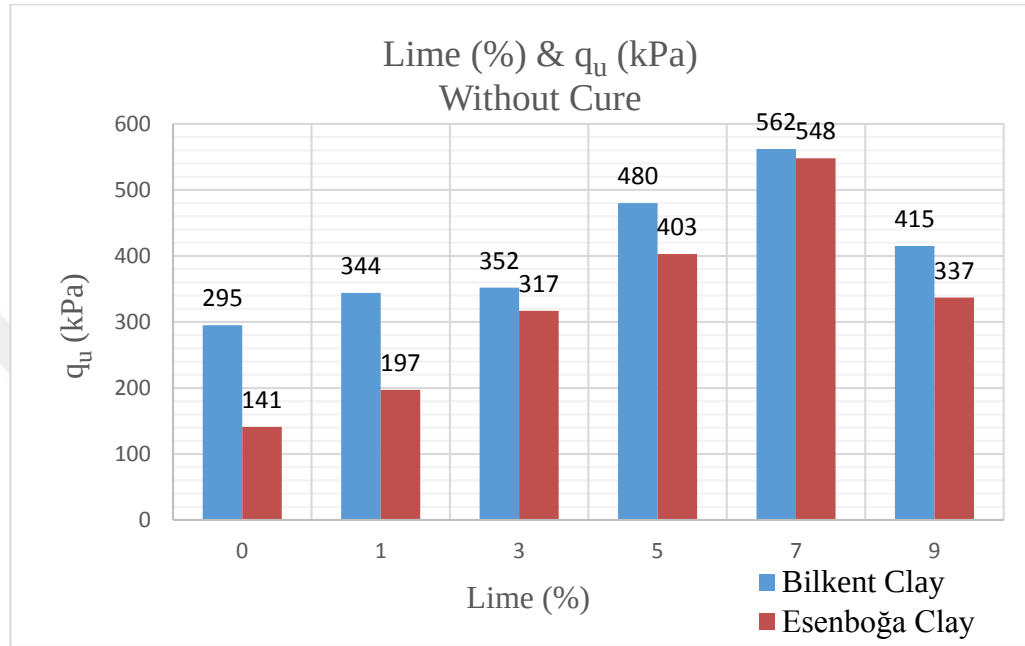


Figure 5.32 : Comparison of unconfined compressive strengths (q_u non-soaked) after resilient modulus test with lime content without cure period.

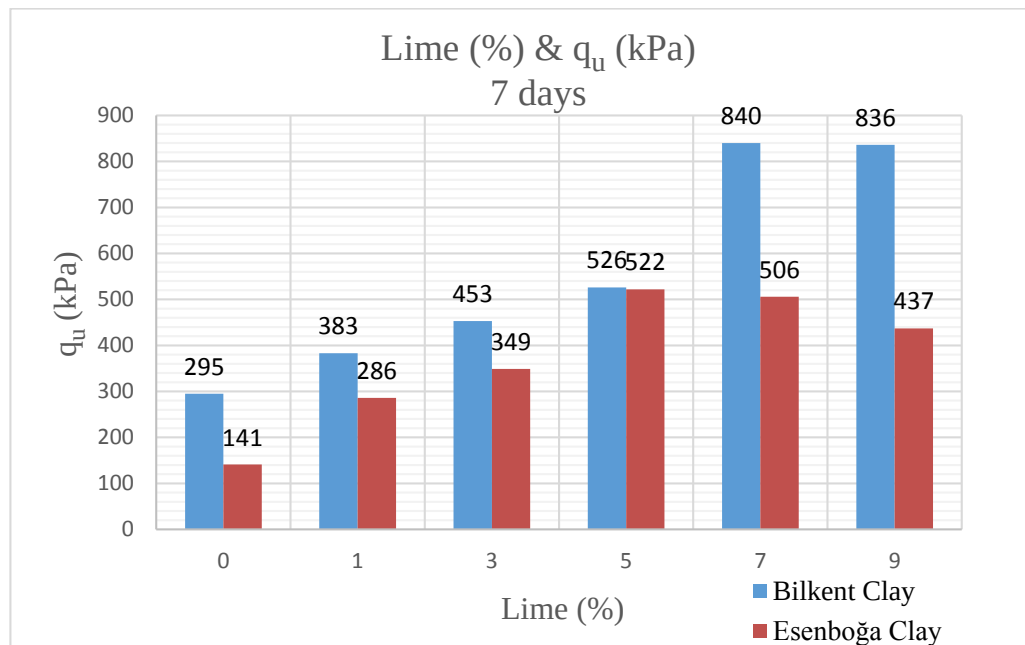


Figure 5.33 : Comparison of unconfined compressive strengths (q_u non-soaked) after resilient modulus test with lime content for 7 days cure period.

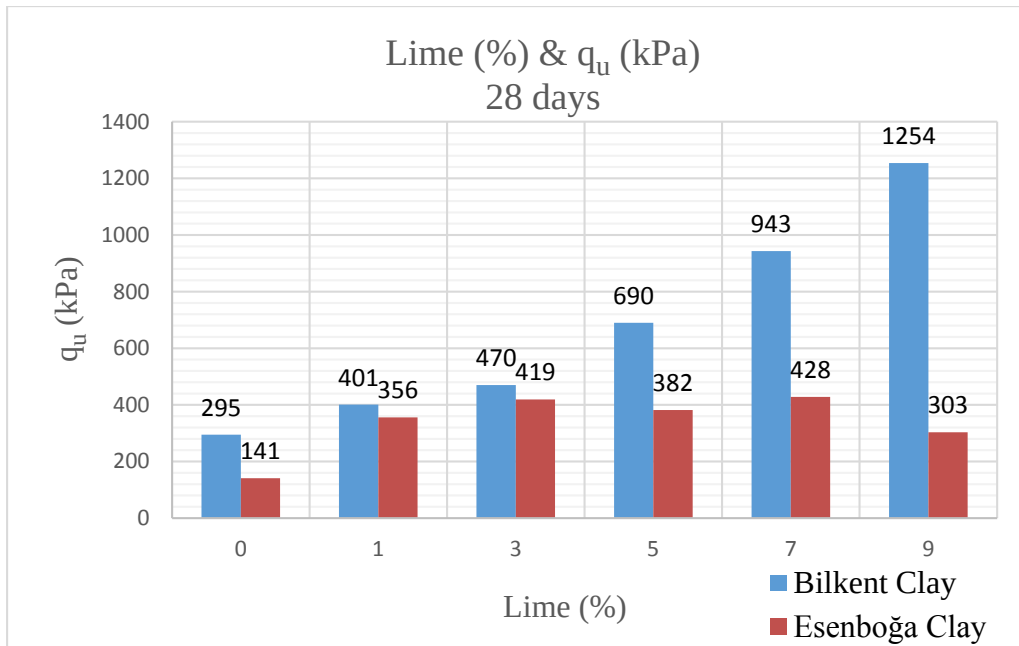


Figure 5.34 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 28 days cure period.

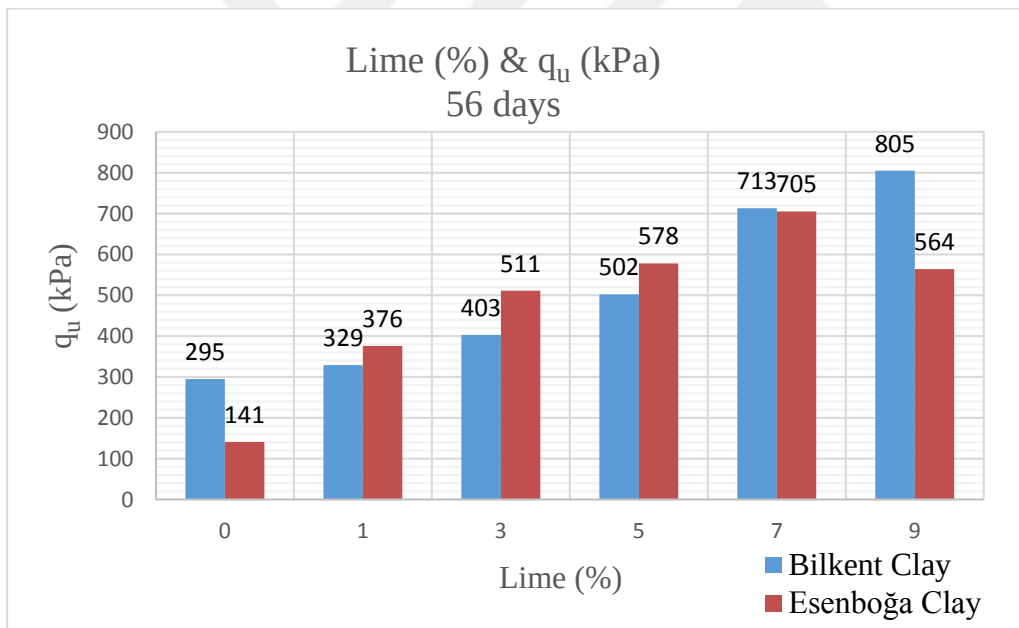


Figure 5.35 : Comparison of unconfined compressive strengths ($q_{u \text{ non-soaked}}$) after resilient modulus test with lime content for 56 days cure period.

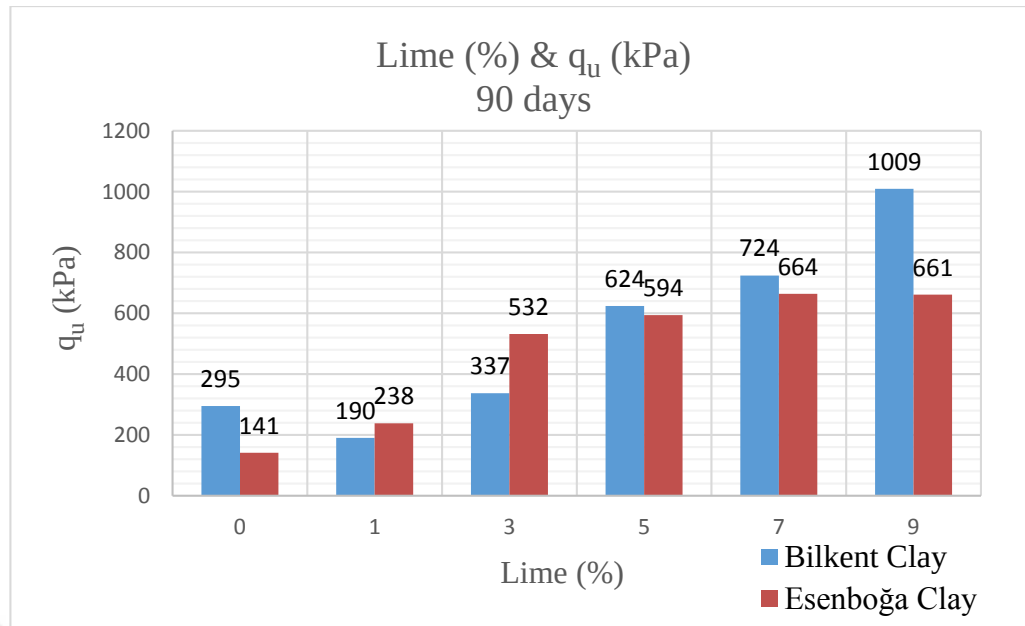


Figure 5.36 : Comparison of unconfined compressive strengths (q_u non-soaked) after resilient modulus test with lime content for 90 days cure period.

5.6. Comparison of SEM (Scanning Electron Microscope) Analyses

The SEM analyses have been performed for natural Esenboğa Clay, lime-stabilized (7% lime content) Esenboğa Clay and natural Bilkent Clay.

Rose-shaped images observed in natural Esenboğa Clay give indication the montmorillonite minerals which tend to occur in thin equidimensional flakes that have a film-like appearance. The effect of lime agent has been observed significantly in lime-stabilized sample which gets a porous nature due to lime addition. However, different views from different angles may result in some discrepancies in the results. In some parts of the lime-stabilized sample, a more dense structure rather than porous has been observed since the sample is not perfectly homogeneous.

Natural Bilkent Clay has been imaged by SEM. Since the Bilkent Clay had some lime veins in it naturally, the results of SEM analyses were different accordingly. In the first set of SEM analyses, the effect of natural lime veins existing in the Bilkent Clay was quite dominant. If an evaluation of a SEM analysis of a natural clay is necessary, the second set of SEM analyses give more proper results.



6. CONCLUSIONS AND RECOMMENDATIONS

Stabilization effect of lime and fly ash on an extremely highly plastic Esenboğa Clay and very highly plastic Bilkent Clay have been investigated with special reference to swell potential and cyclic loading properties in this study.

The conclusions for Esenboğa Clay will be given first:

The effective lime content to decrease the liquid limit and plasticity index can be considered as 4 to 5%.

It has been observed that the activity values decrease generally with the increase in lime content; whereas these values increase with the increase in fly ash content of treated samples. This is because the rate of change in plasticity index values are different from that of clay size fraction values for different agents.

The optimum lime content is about 3% as far as the swelling is concerned. In a design of sub-base of surficial structures (i.e. highways, railways, etc.), this optimum lime content can be taken as a reference. Swell tests should be executed with this reference value. In case of not satisfying the design criteria, this optimum lime content should be increased.

Since lime is more effective than the fly ash, it is highly probable that the design criteria are to be satisfied at reasonable lime contents. For instance, laboratory swell percentage tests show that about 8% lime content for Esenboğa Clay may be sufficient in a surficial sub-structure design as far as the swelling criteria are concerned. Laboratory tests show, on the other hand, that the fly ash agent may not be sufficient alone for swelling design criteria in such type of extremely highly plastic clays.

Lime agent effect on the behavior of swelling is much more effective than the fly ash agent. Swell pressures are about 6.5 times greater in fly ash improvement with 20 to 25% amounts than the pressures found in lime improvement with 7 to 9% amounts.

When the variation of unconfined compressive strength with increasing lime content for non-soaked and soaked conditions is considered, it is noticed that the unconfined compressive strengths are greater in non-soaked condition up to 5% lime content. Starting from 7% especially, the strengths are greater in soaked condition due to pozzolanic reactions.

The unconfined compressive strengths are always greater in non-soaked condition for fly ash treatment. It is clearly seen that 15% fly ash content is optimum since the unconfined compressive strengths decrease at 20% and 25% fly ash contents for both non-soaked and soaked samples.

As a result, the above conclusions are quite compatible with the decrease of swell potential due to lime treatment. As the swell pressure and swell percentages decrease with the increase in lime content, MBV and activity decrease as well.

Curing time effect of lime stabilization in an extremely highly plastic Esenboğa Clay has been investigated in this research. The soil samples for cure periods of 7, 28 and 56 days have been tested.

The effective lime content to decrease the liquid limit and plasticity index has been found as 4 to 5% without cure.

For the curing periods of 28 and 56 days, 5% lime content is the optimum content at which the swell percentage is decreased to null. It is interesting to note that 28 days curing period, which resembles the optimum time for strength gain in concrete, is quite adequate to get allowable swell percentages at about 5% lime content. Then, it can be concluded that 56 days curing period may not be necessary as far as the swell percentage criterion is concerned. This result is also valid in swell pressure tests so that 5% lime content is sufficient to decrease the swell pressures considerably for all curing periods.

Soaked and non-soaked samples have been tested to get unconfined compressive strengths. It has been concluded that the unconfined compressive strengths increase dramatically as compared to non-cured strengths for all cure periods especially for non-soaked samples. 28-day curing period can be accepted as optimum. Unconfined

compressive strengths of soaked samples are always lower than the ones in non-soaked condition in all lime contents for curing periods of 7, 28 and 56 days.

The conclusions for Bilkent Clay which is very highly plastic will be given secondly:

The effective lime content to decrease the liquid limit and plasticity index can be considered as 4 to 5%.

It has been observed that the activity values increase generally in treated samples. This is because the rate of decrease of plasticity index values is generally smaller than the rate of decrease of clay size fraction values.

The optimum lime and fly ash contents are about 3% and 10% respectively as far as the swelling is concerned. In a design of sub-base of surficial structures (i.e. highways and railways, etc.), these optimum lime and/or fly ash contents can be taken as a reference. Swell tests should be executed with these reference values. In case of not satisfying the design criteria, these optimum contents should be increased.

Since lime is more effective than the fly ash, it is highly probable that the design criteria are to be satisfied at reasonable lime contents. For instance, laboratory swell percentage tests show that 9% to 10% lime content for Bilkent Clay may be sufficient in a surficial sub-structure design as far as the swelling criteria are concerned. Laboratory tests show, on the other hand, that the fly ash agent may not be sufficient alone for swelling design criteria in such type of very highly plastic clays.

The effect of lime on swell pressure is maximum in lime content increase from 1% to 3%. This is compatible with the result of swell percentage tests.

The effectiveness of lime agent on swell behavior can also be observed in swell pressure tests. Swell pressures are about 2.7 times greater in fly ash improvement with 20 to 25% amounts than the pressures found in lime improvement with 7 to 9% amounts.

When the variation of unconfined compressive strength with increasing lime content for non-soaked and soaked conditions is considered, it is noticed that the unconfined compressive strengths are greater in non-soaked condition up to 7% lime content.

Starting from 7%, the strengths are greater in soaked condition due to pozzolanic reactions.

The unconfined compressive strengths are always greater in non-soaked condition for fly ash treatment.

When the variation in the mean resilient modulus values are considered for the extremely highly plastic Esenboğa Clay (PI=101%), 3% lime content seems quite effective for both non-cured and cured conditions. 56-days curing period gives maximum mean resilient modulus values in this clay for which the 90-days can be accepted ineffective. In Bilkent Clay which is very highly plastic (PI=76%), 3% lime content can be accepted effective at early stages (i.e. at non cured and 7 days cured conditions) as far as the mean resilient modulus values are concerned. However, 7% lime content gives generally maximum increases in resilient modulus values at later cure periods of 28, 56 and 90 days. 90-days curing period gives always maximum resilient modulus value in the tests for Bilkent Clay. It should be noticed here that the Bilkent Clay is composed of 100% palygorskite clay mineral which may influence the results.

The minimum value of M_r for the subgrade layer in highways, according to Turkish Highway Standard's minimum CBR value of 8, is 66.000 kPa (9500 psi) according to a correlation between M_r and CBR. This condition is satisfied at 3% lime content in all cured samples and without cure condition for both Esenboğa and Bilkent Clays.

The compressibility properties lime-treated of both clays have been examined by consolidation tests. Both compression and expansion (recompression) curves are quite steep in natural state of the clays indicating high compressibility and expansion capacity. The lime agent is again effective here so that the compressibility decreases as lime content increases for both clays. However, at 1% and 3% lime contents, the compressibility of both samples could not be decreased effectively for both clays. When the compression and swelling indices are examined, 5% lime content for both clays seems to be quite effective since the slopes of the compression and recompression lines are quite mild. It can be concluded that 5% lime content for both clays is an optimum stage at which the compressibility characteristics are substantially influenced in a positive sense.

Similarly, fly ash-treated samples have been investigated from the point of compressibility. The compression and expansion index values have been found as unstable with different amounts of fly ash. It can be concluded that fly ash agent can be accepted to be ineffective as far as the compressibility characteristics of the clays are concerned.

The unconfined compressive strengths of the specimens with 10 cm diameter and 20 cm height dimensions have been found after resilient modulus tests. The unconfined compressive strengths of the cured specimens show that the curing time is quite effective in strength increase. The rate of increase in strength values of the Esenboğa Clay is more than the rate in Bilkent Clay. The strength values are more than the three times the value of untreated sample at only 3% lime content in Esenboğa Clay. On the other hand, similar strength increases could be obtained at about 7% lime content in Bilkent Clay. Thus, it can be concluded that the Esenboğa Clay gives more positive and effective reactions as far as the unconfined compressive strengths after the resilient modulus tests are concerned. This is especially observed as cure period increases in Esenboğa Clay.

The results achieved in this research show that lime agent only by itself can be utilized quite effectively in the very highly plastic expansive clay obtained from a construction site in Ankara, especially on the swelling properties. However, this result is not valid for the agent fly ash which is an industrial waste. It is recommended to use fly ash with a calcium-based agent like lime or cement to improve such type of very highly plastic expansive clays in terms of swelling criteria. Using fly ash especially is quite advantageous from an economical and environmental points of view.

The SEM analyses performed in the final part of this research include natural Esenboğa Clay, lime-stabilized (7% lime content) Esenboğa Clay and natural Bilkent Clay.

The montmorillonite minerals determined in Rietveld analysis for Esenboğa Clay have been observed in SEM analysis also. Cohesive thin films composing of basic texture of this kind of clays have been observed with 25000 times magnification for Esenboğa Clay.

The lime-stabilized Esenboğa Clay sample gets a porous nature as determined by SEM analyses. This effect has not been observed equally in all parts of the sample due to probable lack of homogeneity.

The calcium mapping given for lime-stabilized sample shows a significant increase in calcium concentration when compared with the natural Esenboğa Clay. Morphology changes from dense to porous. This is an indication of flocculation and agglomeration. Thus, pozzolanic reactions occur in the long term. It should be remembered that the tested sample here has been cured for about 30 months.

The strength gain is related to pozzolanic reactions in lime-stabilized clays. Since the SEM analyses have proven pozzolanic reactions, the results are compatible with the findings of unconfined compression and Resilient Modulus tests in which significant improvements have been reached for cured samples.

As far as the unconfined compression strengths are concerned, for the maximum reactivity between lime and clay, the pH value should be around 12.4 in the presence of water. Aluminum and silicium dissolve in a great potential from the clay mineral structure and as a result of this CSH, CAH and CASH cementitious compounds are formed. These reactions, known as pozzolanic reactions, are mainly curing-time dependent. In the soaked condition of unconfined compression tests water can reach all the unsaturated voids within the clay samples. Moreover, there is more time in soaked condition to complete the swelling. Thus, after a certain lime content the strengths in soaked condition exceeds the strengths of the one non-soaked.

Since the untreated natural Bilkent Clay had some lime veins in it naturally, the results of SEM analyses were different accordingly. The images taken from the parts where lime veins present have about two times more calcium content than that of the other vein-free parts.

6.1 Recommendations for Future Research

The effect of lime and fly ash treatment on the very highly and extremely highly plastic clays were the subject of this research. The effects of these agents are not well known on organic clays. Organic clays and peats can be the subject of another reaserch.

Moreover, the combination of these agents in a stabilization process was not investigated in very highly and/or extremely highly plastic clays. Lime and fly ash agents can be used at the same time in a treatment work. It has been concluded in this study that fly ash agent alone is not sufficient to get effective stabilization results. Therefore, lime and fly ash together can be utilized in especially extremely highly plastic clays to get satisfactory results. Since fly ash agent is strongly recommended as far as the environmental benefits are concerned, using more fly ash should be investigated in more difficult expansive clays which can be extremely highly plastic or can include sporadic minerals in it.

The effect of mellowing time can be more influential than expected. The effect of this time in the lime and/or fly ash treatment of expansive clays can be investigated in a research.

The leaching characteristics of the fly ash agent are needed to be investigated more deeply in a research since the results in the literature are generally procedure-dependent.



REFERENCES

- AASHTO T307-99** (2007), *Determining the Resilient Modulus of Soils and Aggregate Materials, Standard Specification for Transportation Materials and the Methods of Sampling and Testing*. 20th Edition, AASHTO, Washington D.C.
- Achampong, F.** (1996), *Evaluation of resilient modulus for lime and cement stabilized synthetic cohesive soils*. (Doctoral thesis, Wayne State University, Michigan, U.S.A.). Retrieved from <https://digitalcommons.wayne.edu>
- AFNOR** (1998), *Mesure de la Capacité D'adsorption de Bleu de Méthylène d'un sol ou d'un Matériau Rocheux*. l'Association Française de Normalisation, NF P 94-068.
- American Coal Ash Association** (2003), *Fly Ash Facts for Highway Engineers*. FHWA Report No: FHWA-IF-03-019, Washington D.C.
- ASTM D 422** (2007), *Standard Test Method for Particle- Size Analysis of Soils*. American Society Testing Materials, U.S.A.
- ASTM D 427** (1998), *Standard Test Methods for Shrinkage Factors of Soils by the Mercury Method*. American Society Testing Materials, U.S.A.
- ASTM C 618** (1996), *Fly ash and raw or calcined natural pozzolan for use as a mineral admixture in Portland for use as a mineral admixture in Portland cement concrete*. American Society Testing Materials, U.S.A.
- ASTM D 698** (2007), *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (600 kN/m³)*. American Society Testing Materials, U.S.A.
- ASTM D 854** (2014), *Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer*. American Society Testing Materials, U.S.A.
- ASTM D 2166** (2013), *Standard Test Methods for Unconfined Compressive Strength of Cohesive Soils*. American Society Testing Materials, U.S.A.
- ASTM D 4318** (2010), *Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. American Society Testing Materials, U.S.A.
- ASTM D 4546** (2008), *Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils*. American Society Testing Materials, U.S.A.
- ASTM D 4829-95** (1995), *Standard Test Methods for Expansion Index of Soils*. American Society Testing Materials, U.S.A.

- ASTM D 6913** (2009) *Standard Test Method for Particle- Size Distribution (Gradation) of Soils Using Sieve Analysis*. American Society Testing Materials, U.S.A.
- Athanasopoulou, A., & Kollaros, G.,** (2011). Use of additives to improve the engineering properties of swelling soils in Thrace, Northern Greece. *5th Int. Conf. on Computational Materials Characterization* (pp. 327-338), WIT Press, Southampton.
- Baba, A., & Kaya, A.** (2004). Leaching characteristics of fly ash from thermal power plants of Soma and Tunçbilek, Turkey. *Environmental Monitoring and Assessment*, 91, 171-181, Kluwer Academic Publishers, The Netherlands.
- Baykal, G., Arman, A., & Ferrel, R.,** (1989). Accelerated curing of fly ash-lime-soil mixture. *Transportation Research Record*, 1219, 82-92.
- Baykal, G., & Döven A.G.** (2000). Utilization of fly ash by pelletization process: theory, application areas and research result. *Resources, Conservation and Recycling*. 30, 59-77.
- Baykal, G., Edinçliler, A., & Saygılı, A.** (2004). Highway embankment construction using fly ash in cold regions. *Resources, Conservation and Recycling*. 42, 209-222.
- Bish, D.L., & Post, J.E.** (1993). Quantitative mineralogical analysis using the Rietveld full-pattern fitting method. *American Mineralogist*, 78, 932-940.
- Bish, D.L., & Howard, S.J.** (1988). Quantitative phase using the Rietveld method. *J.Appl. Cristallography*, 21, 86-91.
- Bohor, B.F., & Hughes, R.E.** (1971). Scanning electron microscopy of clays and clay minerals. *Clays and Clay Minerals*, 19, 49-54.
- Bonetto, R.D., Zalba P.E., Conconi, M.S., & Manassero, M.** (2003). The Rietveld method applied to quantitative phase analysis of minerals containing disordered structures. *Rev. Geologica de Chile*, 30(1), 103-115.
- Bozbey, İ., & Garaisayev, S.** (2010). Effects of soil pulverization quality on lime stabilization of an expansive clay, *Environmental Earth Science*, 60(6), 1137–1151.
- Bozbey, İ., Demir, B., Sağlık, A., Çömez, Ş., & Mert, A.** (2015). Effects of lime on resilient modulus properties of a clayey soil. *IRF (International Road Federation) Europe and Central Asia Regional Congress*, İstanbul, Turkey.
- Bravo, C., Rodriguez, A., Saavedra, L., Trujillo, Q., Mejia, S., & Huaypar, V.** (2003). Caracterización mineralógica de suelos agrícolas de Vicus. *Región Piura, por DRX y espectroscopia Mössbauer*. *Rev. XIV Simposio Peruano de Física*.
- Chen, F.H.** (1988). *Foundation on Expansive Soil*. Elsevier Science, New York.
- Christopher, B.R., Schwartz, C., & Boudreau, R.** (2006). *Geotechnical Aspects of Pavements*, *FHWA Report, FHWA NHI-05-037*. National Highway Institute, U.S.A.

- Chu, S.C., & Kao, H.S.** (1993). A study of engineering properties of a clay modified by fly ash and slag. *Fly Ash for Soil Improvement, Geotechnical Special Publication*, 36, 89-99.
- Chuan-De, R., & Ward, C.** (2003). Quantitative X-ray powder diffraction analysis of clay minerals in Australian coals using Rietveld methods. *Appl. Clay Science*, 21, 227-240.
- CMC (China Ministry of Construction).** (2004). *Specifications for the design of highway subgrades*, JTG D 30, Beijing, China.
- Cockrell, C.F., & Leonard, J.W.** (1970). *Characterization and utilization studies on limestone modified fly ash*, Technical Report, Coal Research Bureau, West Virginia University.
- Coduto, D.P.** (2001). *Foundation design: principles and practices*. 2nd Edition, Prentice Hall, U.S.A.
- Çokça, E.** (1991). *Swelling potential of expansive soils with a critical appraisal of the identification of swelling of Ankara soils by Methylene Blue Tests*, (Unpublished doctoral thesis), Middle East Technical University, Ankara, Turkey.
- Çokça, E.** (2001). Use of class C fly ash for the stabilization of an expansive soil. *J. Geotech. Geoenviron. Eng.*, ASCE, 127(7), 568-573.
- Çokça, E.** (2002). Relationship between methylene blue value, initial soil suction and swell percent of expansive soils. *Turkish Journ. of Eng. Env. Sciences, TÜBİTAK*, 26, 521-529.
- Dakshanamurthy, V., & Raman, V.** (1973). A simple method of identifying an expansive soil. *Soils and Foundations*, 13(1), 13-22.
- Das, B.M.** (1999). *Principles of foundation engineering*. 4th Edition, PWS Publishing, California, U.S.A.
- Dash, S.K., & Hussain, M.** (2012). Lime stabilization of soils: Reappraisal. *Journal of Materials in Civil Engineering, ASCE*, 24(6), 707-714.
- De la Torres, A.G., Bruqueand, S., & Aranda, M.A.G.** (2001). Rietveld quantitative amorphous content analyses. *J.Appl. Cristallography*, 34, 196-202.
- Dermatas, D., & Dadachov, M.** (2003). Rietveld quantification of montmorillonites in lead contaminated soils. *Appl. Clay Science*, 23, 245-255.
- Eades, J.L., & Grim, R.E.** (1960). Reaction of hydrated lime with pure clay minerals in soil stabilization. *Bulletin* 262, Highway Research Board, Washington, D.C.
- Edil, T.B., Acosta H.A., & Benson C.H.** (2006). Stabilizing soft fine-grained soils with fly ash. *Journal of Materials in Civil Engineering, ASCE*. 18(2), 283-294.
- Fesharaki, O., Garcia-Romero, E., Cuevas-Gonzales, J., & Lopez-Martines, N.** (2007). Clay mineral genesis and chemical evolution in the Miocene Sediments of Somosoguas, Madrid Basin, Spain. *Clay Minerals*, 42, 187-201.

- Fossberg, P. E.** (1969). Some deformation characteristics of a lime-stabilized clay. *Highway Research Record*, 62, HRB, 19-25.
- Geiman, C.M.** (2005). *Stabilization of soft clay subgrades in Virginia Phase-I laboratory study* (Masters thesis, Virginia Polytechnic Institute and State University, U.S.A.). Retrieved from <http://citeseerx.ist.psu.edu>
- George, S., Ponniah, D., & Little, J.** (2006). Effect of temperature on lime-soil stabilization. *Constr. Building Materials*, 6(4), 247-252.
- Handy, R.L.** (1995). *The day the house fell*. ASCE Press, New York.
- Huang, R., & Wu, L.** (2007). Stability analysis of unsaturated expansive soil slope. *Earth Science Frontiers*, 14(6), 129-133.
- ICDD** (2017). *Mineral Powder Diffraction File Search Manual (Digital Form)*, International Centre for Diffraction Data (inspected by ASTM).
- Ismail, H.A.H.** (2006). *Treatment and improvement of the geotechnical properties of different soft fine-grained soils using chemical stabilization*. (Doctoral thesis, Martin Luther Halle-Wittenberg University, Germany). Retrieved from <https://sundoc.bibliothek.uni-halle.de>
- Jones, R., & Bish, D.** (1991). Quantitative X-ray diffraction analysis of soils: Rietveld full pattern mineral concentration and pattern curve fitting vs. P sorption of bauxite soils, *Proc. of the Amer. Clay Min. Soc. Meeting 28* (pp. 84-94).
- Kavak, A.** (1996). *The behavior of lime-stabilized clays under cyclic loading*, (Unpublished doctoral thesis), Boğaziçi University, İstanbul, Turkey.
- KGM** (2005). *Technical Specification for Lime Improvement*, General Directorate of Highways, Ankara, Turkey (in Turkish).
- KGM** (2008). *Esneklik Modülü Deneyi*, General Directorate of Highways, Ankara, Turkey (in Turkish).
- Kumar, B.R.P., & Sharma, R.S.** (2004). Effect of fly ash on engineering properties of expansive soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(7), 764-767.
- Lautrin, D.** (1987). Une procedure rapide d'identification des argiles. *Bull. Liasion Labo. Et Ch.*, 152, 75-84.
- Lav, A.H., & Lav, M.A.** (2000). Microstructural development of stabilized fly ash as pavement base material. *Journal of Materials in Civil Engineering*, 12(2), 157-163.
- Lav, A.H., Lav, M.A., & Göktepe, A.B.** (2006). Analysis and design of a stabilized fly ash as pavement base material. *Fuel (Elsevier)*, 85(16), 2359-2370.
- Lav, M.A., & Lav, A.H.** (2014). Effects of stabilization on resilient characteristics of fly ash as pavement material. *Construction and Building Materials (Elsevier)*, 54, 10-16.
- Little, D.N.** (1995). *Handbook for stabilization of pavement subgrades and base courses with lime*. Kendall/Hunt U.S.A.

- Mallela, J., Harold Von Quintus, P., Smith, K.L., & Consultants, E.** (2004). *Consideration of lime-stabilized layers in mechanistic-empirical pavement design*. The National Lime Association, Arlington, Virginia, U.S.A.
- Mc Custer, L.B., Von Dreele, R.B., Cox, D.E., Löuer, D., & Scardi, P.** (1999). Rietveld refinement guidelines. *J. of Appl. Cristallography*, 32, 36-50.
- Mellas, M., Hamdane, A., Benmeddour, D., & Mabrouki, A.** (2012). Improvement of the expansive soils by the lime for their use in road works, *Proc. 10th Int. Conf. on Advances in Civil Eng.* (pp. 1-8). Middle East Technical University, Ankara, Turkey.
- Mitchell, J.K.** (1993). *Fundamentals of soil behavior*. 2nd Edition, John Wiley&Sons, Inc., U.S.A.
- Mitchell, J.K., & Raad, L.** (1973). Control of volume changes in expansive earth materials. In D.R. Lam, & S.J. Hanna (Eds.), *Proc. Workshop Expansive Clays and Shales in Highway Design and Construction* (pp. 200-219), Federal Highway Administration, Denver, CO.
- Mitchell, J.K., & Soga, K.** (2005). *Fundamentals of soil behavior*. 3rd Edition, John Wiley&Sons, Inc., U.S.A.
- Nalbantoğlu, Z.** (2006). Lime stabilization of expansive clays. In A.A. Al-Rawas, & M.F.A. Goosen (Eds.), *Expansive Soils-Recent Advances in Characterization and Treatment*, Taylor & Francis, The Netherlands.
- Nelson, J.D., & Miller, D.J.** (1992). *Expansive soils: Problems and practice in foundation and pavement engineering*. John Wiley&Sons, Inc., U.S.A.
- Nelson, J.D., Chao, K.C., Overton D.D. & Nelson, E.J.** (2015). *Foundation engineering for expansive soils*. John Wiley&Sons, Inc., U.S.A.
- O'Neill, M.W., & Poormoayed, N.** (1980). Methodology for foundations on expansive clays. *Journal of Geotechnical Engineering Division, ASCE*, 106(6), 1345-1367.
- Ortiz, A., Cumbrera, F., Sanchez Bajo, F., Guibertau, F., Xu, H., & Padture, N.** (2000). Quartz phase composition analysis of liquidphase sintered silicon carbide using the Rietveld method. *J. Am. Ceram. Soc.*, 83(9), 2282-2286.
- Pascoal, C., Macado, R., & Pandeolfelli, V.** (1999). Composicao quimica e mineralogica de bauxitas refratarias. *Anais do 43 Congresso Brasileiro de Ceramica*, 8301-8314.
- Patel, M.A., & Patel, S.A.** (2012). A review on effects of stabilizing agents for stabilization of weak soils. *Civil and Environmental Research*, 2(6).
- Peck, R.B., Hanson, W.E., & Thornburn, T.H.** (1974). *Foundation engineering*. John Wiley&Sons, Inc., U.S.A.
- Phanikumar, B.R.** (2009). Effect of lime and fly ash on swell, consolidation and shear strength characteristics of expansive clays: A comparative study. *Geomechanics and Geoengineering: An International Journal*, 4(2), 175-181.

- Phanikumar, B.R., and Sharma, R.S.** (2007). Volume change behavior of fly ash-stabilized clays. *Journal of Materials in Civil Engineering, ASCE*, 19(1), 67-74.
- Phanikumar, B.R., Sreedharan, R., & Aniruddh, C.,** (2015). Swell-compressibility characteristics of lime-blended and cement-blended expansive clays - A comparative study. *Geomechanics and Geoengineering: An International Journal*, 10(2), 153-162.
- Raudsepp, M., Pani, E., & Diple, G.** (1999). Measuring mineral abundance in skarn; I, The Rietveld method using X-ray powder diffraction data. *Canadian Min.*, 37, 1-15.
- Rietveld, H.M.** (1969). A profile refinement method for nuclear and magnetic structures. *J. Appl. Crystal*, 2, 65-71.
- Robnett, Q.L., & Thompson, M.R.** (1976). Effect of lime treatment on the resilient behavior of fine grained soils. *Transportation Research Record*, 215, 8-23.
- Santamarina, J.C., Klein, K.A., Wang, Y.H., & Prencke, E.** (2002). Specific surface: determination and relevance. *Canadian Geotechnical Journal*, 39, 233-241.
- Sauer, E.K., & Weimer, N.F.** (1978). Deformation of lime modified clay after freeze-thaw. *Transportation Engineering Journal, TE2*, 201-212
- Seco, A., Ramirez, F., Miqueleiz, L., & Garcia, B.** (2001). Stabilization of expansive soils for use in construction. *Applied Clay Science, Elsevier*, 51, 348-352.
- Seed, H.B., Woodward, R. J. Jr., & Lundgren, R.,** (1962). Prediction of swelling potential for compacted clays. *Soil Mechanics and Foundation Division, American Society of Civil Engineers*, 88(SM3), 53-87.
- Sharma, R.S., Phanikumar, B.R., & Rao, B.V.,** (2008). Engineering behaviour of a remoulded expansive clay blended with lime, calcium chloride, and rice-husk ash. *Journal of Materials in Civ. Eng., ASCE*, 20(8), 509-515.
- Sharma, A.K., & Sivapullaiah, P.V.,** (2017). Swelling behaviour of expansive soil treated with fly ash – GGBS based binder. *Geomechanics and Geoengineering: An International Journal*, 12(3), 191-200.
- Singh, G., & Akoto, B.K.A.,** (1981). Rational design of lime stabilized laterite roads. *Journal of Transportation Engineering, ASCE*, 107(TE6), 653-665.
- Skempton, A.W.,** (1953). The colloidal activity of alays, *Proc. of the 3rd Int. Conf. on Soil Mech. and Found. Eng.* (pp.57-67). Zurich, Switzerland.
- Snedker, E. A.,** (1996). M 40-Lime stabilization experiences, *Proc. Seminar on Lime Stabilization* (pp. 142-158). London, England: Loughborough Univ.
- Snethen, D.R., Johnson, L.D., & Patrick, D.M.** (1977). *An Investigation of the natural microscale mechanisms that cause volume change in expansive clays*, Federal Administration Report, FHWA-RD-77-75, Springfield, VA.

- Şenol, A., Bin-shafique, Edil, T.B., & Benson, C.H.** (2002). Use of Class C fly ash for stabilization of soft subgrade. *Fifth International Congress on Advances in Civil Engineering*, Istanbul, Turkey: İstanbul Technical University.
- Şenol, A., & Edil, T.B.** (2004). Uçucu kül ile stabilize edilen yumuşak zeminlerin CBR sonuçlarının değerlendirilmesi. *Zemin Mekaniği ve Temel Mühendisliği Onuncu Ulusal Kongresi*, İstanbul, Turkey: İstanbul Teknik Üniversitesi (in Turkish).
- Taubaso, C., Torres Sanchez R.M., & Dos Santos Afonsa, M.** (2004). Modelling soil surface charge density using mineral composition. *Geoderma*, 121, 123-133.
- Taylor, J.C., & Rui, Z.** (1992). Simultaneous use of observed and calculated standard profiles in quantitative XRD analysis of minerals by the multiphase Rietveld method: The determination of pseudorutile in mineral sands products. *Powder Diffraction*, 7(3), 152-161.
- TS EN ISO 9001** (2008). *Quality Management Systems-Requirements*, Turkish Standard, Turkey.
- TRB Report** (1987). *Lime stabilization: reactions, properties, design, and construction*, State of the Art Report 5, Transportation Research Board, Washington D.C.
- Türker D., & Çokça, E.,** (2006). Effects of addition of fly ash on swell potential of an expansive soil, In A.A. Al-Rawas, & M.F.A. Goosen (Eds.), *Expansive Soils-Recent Advances in Characterization and Treatment*, Taylor & Francis, The Netherlands.
- Url-1** <www.carmeusena.com>, date retrieved 15.12.2018.
- Url-2** <www.dappolonia.com>, date retrieved 15.12.2018.
- Url-3** <www.geoengineer.com>, date retrieved 15.12.2018.
- Url-4** <www.gosanangelo.com>, date retrieved 15.12.2018.
- Url-5** <www.somatermik.com.tr>, date retrieved 15.12.2018.
- van der Merwe, D. H.,** (1964). The Prediction of heave from the plasticity index and percentage clay fraction of soils. *Civil Engineer in South Africa*, 6(6), 103-107.
- van Olphen, H.** (1977). *An introduction to clay colloid chemistry*. 2nd Edition, Wiley Interscience, New York.
- Yükselen, Y., & Kaya A.** (2006). Prediction of cation exchange capacity from soil index properties, *Clay Minerals*, 41, 827-837.
- Weidler, P., Luster, J., Scheider, J., Sticher, H., & Gehring, U.** (1998). *The Rietveld method applied to quantitative mineralogical and chemical analysis of ferralitic soil*. *Europ. J. of Soil Sci.* 49, pp. 95-105
- Wilson, S.D.,** (1950). Small soil compaction apparatus duplicates field results closely. *Engineering News Record*, 135(19), 34-36.
- Zangue Adjia, H., Villieras, F., Kamga, R., & Thomas, F.** (2013). Mineralogy and physico-chemical properties of alluvial clays from Far-North Region of

Cameroon: A Tool for an environmental problem. *Intern. J. of Water Resources and Environmental Engineering*, 5(1), 54-66.

Zumrawi, M.M.E., Abdelmarouf, A.O., & Gameil, A.E.A. (2017). Damages of buildings on expansive soils: Diagnosis and avoidance. *Inter. J. of Multidisciplinary and Scientific Emerging Research*, 6(2), 108-116.



APPENDICES

APPENDIX A : Properties of Esenboğa and Bilkent Clays with lime and fly ash agents

APPENDIX B : XRD analysis results

APPENDIX C : XRF analysis results

APPENDIX D: Effects of stabilization on compression parameters



APPENDIX A

Table A.1 : Properties of Esenboğa Clay with lime agent.

Sample	G _s	Void Ratio **	Sieve Analysis			Atterberg Limits					Unconfined Comp.Test *	Unconfined Comp.Test *	Permeability Coeff. **	Proctor Test		Swell	Swell
			+No.4	-No.200	Clay Size	LL	PL	PI	Ls	SL	q _u	q _u	k (m/sec)	γ _d max	W _{opt}	Pressure *	Percentage *
			(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(kPa)	(kPa)		g/cm ³	%	(kPa)	(%)
No		e _o				Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Shrinkage Limit	(Non-soaked)	(Soaked)					
Natural Sample**		1,47									74	27	5,949 x 10 ⁻¹⁰			29,4	2,1
Natural Sample*	2,70	1,56	0	99,8	67,5	135	34	101	28	14	193	58		1,16	39,5	408,9	35,5
1%Lime	2,67	1,30	0	99,4	67	134	42	92	29	15	209	59		1,17	40,8	225,6	29,0
3%Lime	2,64	1,17	0	99,6	66	128	38	90	30	17	287	234		1,27	36,8	193,2	7,3
5%Lime	2,67	1,13	0	99	52	95	43	52	24	18	320	433		1,24	45,2	119,6	5,3
7%Lime	2,66	1,45	0	98,8	46	93	48	45	25	19	384	645		1,13	45,2	41,5	0,6
9%Lime	2,60	1,17	0	99	27	92	68	24	21	29	410	690		1,19	43,0	34,6	0,4

Table A.2 : Properties of Esenboğa Clay with fly ash agent.

Sample	G _s	Void Ratio **	Sieve Analysis			Atterberg Limits					Unconfined Comp. Test *	Unconfined Comp. Test *	Permeability Coeff. **	Proctor Test		Swell	Swell
			+No.4	-No.200	Clay Size	LL	PL	PI	Ls	SL	q _u	q _u	k (m/sec)	γ _d max	W _{opt}	Pressure *	Percentage *
			(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(kPa)	(kPa)		g/cm ³	%	(kPa)	(%)
No		e _o				Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Shrinkage Lim	(Non-soaked)	(Soaked)					
Natural Sample**		1,47									74	27	5,949 x 10 ⁻¹⁰			29,4	2,1
Natural Sample*	2,70	1,56	0	99,8	67,5	135	34	101	28	14	193	58		1,16	39,5	408,9	35,5
5% Fly Ash	2,62	1,17	0	98,4	59	166	42	124	33	15	356	88		1,28	33,5	380,5	27,7
10% Fly Ash	2,55	1,03	0	97,9	52	154	41	113	30	17	386	115		1,31	35,5	358,9	20,7
15% Fly Ash	2,57	1,05	0	96,6	46,5	142	38	104	29	18	405	201		1,31	36,5	330,2	16,1
20% Fly Ash	2,50	1,21	0	94,4	41,5	137	44	93	28	20	242	107		1,20	35,0	267,7	10,2
25% Fly Ash	2,59	1,20	0	92,1	39	127	39	88	26	22	233	110		1,19	33,5	233,4	8,6

Table A.3 : Properties of Bilkent Clay with lime agent.

Sample	Gs	Void Ratio	Sieve Analysis			Atterberg Limits					Unconfined Comp.Test	Unconfined Comp.Test	Permeability Coeff.**	Proctor Test		Swell	Swell
			+No.4	-No.200	Clay Size	LL	PL	PI	Ls	SL	q _u	q _u	k (m/sec)	γ _d max	W _{opt}	Pressure	Percentage
No		e _o	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(kPa)	(kPa)		g/cm ³	%	(kPa)	(%)
						Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Shrinkage Limit	(Non-soaked) *	(Soaked) *					
Natural Sample**		1,28									385	37	5,007 x 10 ⁻⁹			49,8	4,3
Natural Sample*	2,74	1,04	0,5	79,2	45	109	33	76	25	9	424	78		1,37	29,0	374,6	26,9
1%Lime	2,59	0,99	0,5	79,2	44	104	30	74	25	14	491	108		1,32	27,5	276,5	15,2
3%Lime	2,59	0,98	0,5	79,2	38	94	33	61	24	17	510	127		1,33	29,2	145,3	5,8
5%Lime	2,59	0,92	0,5	79,2	31	84	37	47	21	17	613	340		1,33	29,2	121,6	3,9
7%Lime	2,60	1,02	0,5	79,2	25	81	45	36	21	19	567	854		1,34	29,0	65,7	1,2
9%Lime	2,64	0,98	0,5	79,2	20	72	48	24	16	23	485	1293		1,38	30,6	35,8	0,6

Table A.4 : Properties of Bilkent Clay with fly ash agent.

Sample	G _s	Void Ratio	Sieve Analysis			Atterberg Limits					Unconfined Comp.Test	Unconfined Comp.Test	Permeability Coeff.**	Proctor Test		Swell	Swell
			+No.4	-No.200	Clay Size	LL	PL	PI	Ls	SL	q _u	q _u	k (m/sec)	γ _{dmax}	W _{opt}	Pressure	Percentage
			(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(kPa)	(kPa)		g/cm ³	%	(kPa)	(%)
No		e _o				Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Shrinkage Limit	(Non-soaked) *	(Soaked) *					
Natural Sample**		1,28									385	37	5,007 x10 ⁻⁹			49,80	4,30
Natural Sample*	2,74	1,04	0,5	79,2	45	109	33	76	25	9	424	78		1,37	29,0	374,6	26,9
5% Fly Ash	2,69	1,01	0,5	79,2	28	100	26	74	26	13	365	76		1,32	27,7	381,5	17,9
10% Fly Ash	2,64	1,02	0,5	79,2	22	98	28	70	23	14	418	142		1,32	28,0	221,6	11,1
15% Fly Ash	2,62	0,97	0,5	79,2	18	92	30	62	22	14	450	172		1,32	29,4	197,1	7,5
20% Fly Ash	2,59	0,98	0,5	79,2	15	86	31	55	21	17	495	209		1,35	28,5	154,0	6,1
25% Fly Ash	2,58	0,94	0,5	79,2	14	83	33	50	20	19	540	255		1,37	27,3	121,6	4,5

APPENDIX B



Universiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Müşteri : Odtü İnşaat Mühendisliği Bölümü
Geoteknik Bilim Dalı

Rapor Tarihi : 09/06/2017
Sayfa : 4/5

ANALİZ/TEST RAPORU

Rapor No : MAT-17001156

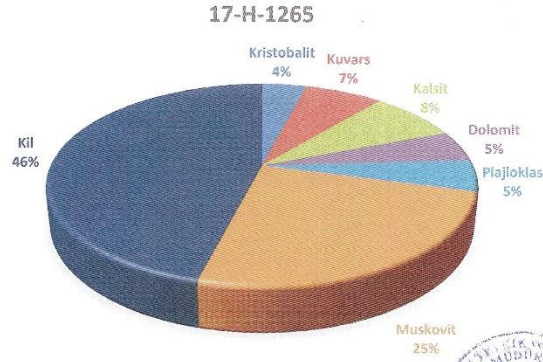
ANALİZ/TEST SONUÇLARI

Numune Kayıt No : 17-H-001265
Numune İşareti : ESENBOĞA CLAY

17-H-001265 no'lu numunenin Cu-X ışın tüplü Panalytical X'Pert Powder XRD analiz cihazı ile (4-70° tüm kayaç, 4-30° detay kil) gerçekleştirilen XRD analizi Panalytical X'Pert Highscore Plus programı ve ICSD veri tabanı kullanılarak Rietveld (1969) metodu ile kantitatif analizi yapılmıştır. Bu analiz sonucuna göre numune;

Tüm Kayaç:

Kuars: % 7,
Kristobalit: % 4,
Kalsit: % 8,
Dolomit: % 5,
Plajiyoklas: % 5,
Muskovit: % 25,
Kil: % 46.



Bülent BAŞARA
Jeoloji Yüksek Mühendisi
Analiz/Test Sorumlusu

Dr. Ebru KAVUĞCU
Birim Yöneticisi
Jeoloji Yüksek Müh.



Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İzinsiz ve muhtarsız rapor geçersizdir.
Sonuçlar sadece analize testi yapılan numuneye aittir.

KY.FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013

Figure B.1 : XRD analysis results for Esenboğa Clay (1).



T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Üniversiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

Müşteri : Odtü İnşaat Mühendisliği Bölümü
Geoteknik Bilim Dalı

Rapor Tarihi : 09/06/2017
Sayfa : 5/5

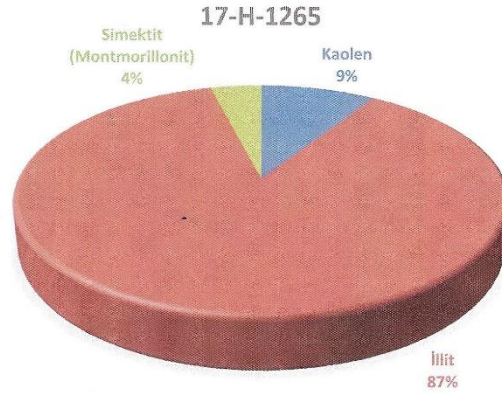
ANALİZ/TEST RAPORU

Rapor No : MAT-170001156

ANALİZ/TEST SONUÇLARI

Detay Kil:

Tüm kayaç analizinde % 46 olarak tespit edilen kil minerallerinin oranı;
Simektit (Montmorillonit): % 4,
Kaolen: % 9,
İllit: % 87.



Not: Yapılan bu XRD analizleri; petrografik, XRF ve SEM analizleri ile desteklenmektedir. Sonuçlar sadece analiz/testi yapılan numuneye aittir.

Numunelere ait XRD grafikleri .jpeg formatında ekte sunulmuştur.

Bülent BAŞARA
Jeoloji Yüksek Mühendisi
Analiz/Test Sorumlusu

Dr. Ebru KAYIKÇU
Birim Yöneticisi
Jeoloji Yüksek Müh.



Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İmzasız ve mühürlü rapor geçersizdir. Sonuçlar sadece analiz/testi yapılan numuneye aittir.

KY.FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013

Figure B.2 : XRD analysis results for Esenboğa Clay (2).



Üniversiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Müşteri : Odtü İnşaat Mühendisliği Bölümü
Geoteknik Bilim Dalı

Başvuran Kişi : İnci Ünver
Başvuru Tarihi : 10/03/2017
Rapor Tarihi : 09/06/2017
Sayfa : 1/5
Sayı : 91817988.302.03/

ANALİZ/TEST RAPORU

Rapor No : MAT-17001156

NUMUNE KABUL BİLGİLERİ

Numune Kayıt No : 17-H-001264 – 17-H-001265
Numune Kayıt Tarihi : 10/03/2017
Numunenin Tanımı : Toprak
Numune Sayısı : 2
Numunenin Mühür Durumu : ☐ Mühürlü ☒ Mühürsüz

LABORATUVAR BİLGİLERİ

Analiz/Test Tarihi : 19/04/2017 – 04/05/2017
Şahit Numune : ☒ Alındı ☐ Alınmadı
Şahit Numune Saklama Süresi :
Proje Kodu : 2017-20

NUMUNE HAZIRLAMA VE ANALİZ/TEST METOTLARI

Analiz/Test Kodu	Kod Açıklaması	Numune Sayısı
35-30-MP-32	Kil örneklerinin standart (2 ² -70° arası), normal (2 ² -30° arası), etilen glikol (2 ² -30° arası), 300°C ısı (2 ² -30° arası) ve 550°C ısı (2 ² -30° arası) işlem çekimleri yapılarak, ASTM Standartlarına uygun olarak ayrıntılı mineralojik tanımlamaları ile uygun yazılım ve veri tabanları kullanılarak kantitatif mineral analizi yapılır. (IV) 35-30-MP-32 analiz koduna ait Rietveld Metoduyla kantitatif XRD analiz başvurusu için gerekli planlamaların yapılabilmesi sebebi ile başvuru yapılmadan önce Numune Kabul Birimi aracılığı ile Mineraloji-Petrografi Araştırmaları Birimi ile temasa geçilmesi gerekmektedir. Bu metot sadece Kil Grubu minerallerce zengin numunelerde uygulanmakta olup, başvuru öncesi uygulanan metot ve numune içeriği hakkında ilgili Birim ile mütaka görüşülmelidir.	2
35-30-MP-18	Kil örneklerinin standart (2 ² -70° arası), normal (2 ² -30° arası), etilen glikol (2 ² -30° arası), 300°C ısı (2 ² -30° arası) ve 550°C ısı (2 ² -30° arası) işlem çekimleri yapılarak, difraktogramlar bilgisayar çıktısı olarak veya elektronik ortamda verilir.	2

AÇIKLAMALAR

Analiz/Test sonuçları bu raporun tamamlayıcı kısmı olan takip eden sayfalarda verilmiştir.

Onaylayan
Dr. Mustafa ALBAYRAK
Mineraloji-Petrografi Araştırmaları
Koordinatörü



Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İmzasız ve mühürsüz rapor geçersizdir.
Sonuçlar sadece analiz/testi yapılan numuneye aittir.

KY.FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013

Figure B.3 : XRD analysis results for Esenboğa Clay (3).

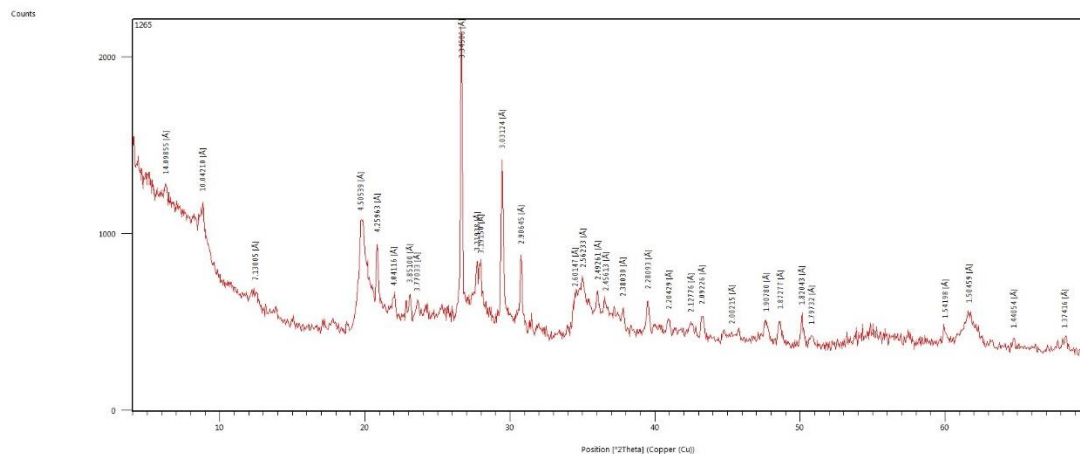


Figure B.4 : X-Ray diffractograms of Esenboğa Clay for bulk sample.

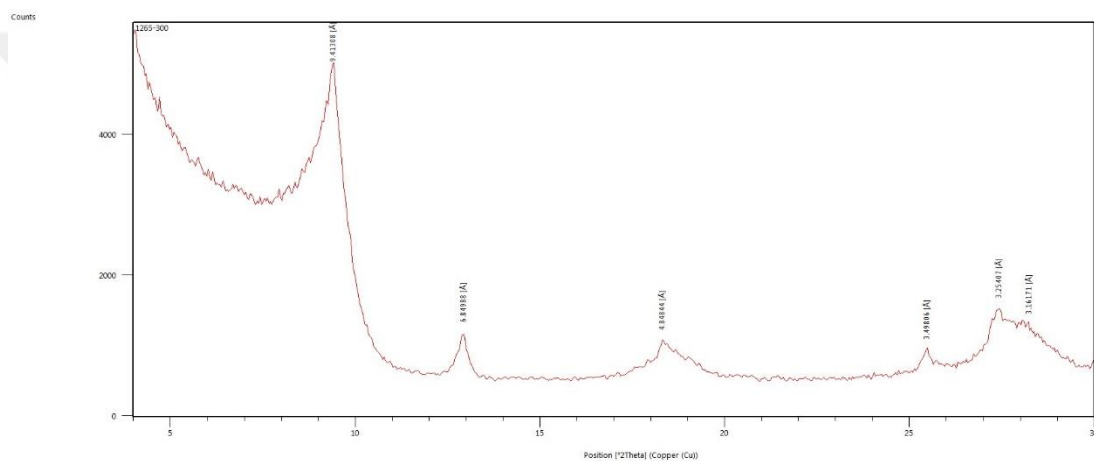


Figure B.5 : XRD patterns for clay fraction in Esenboğa Clay (1).

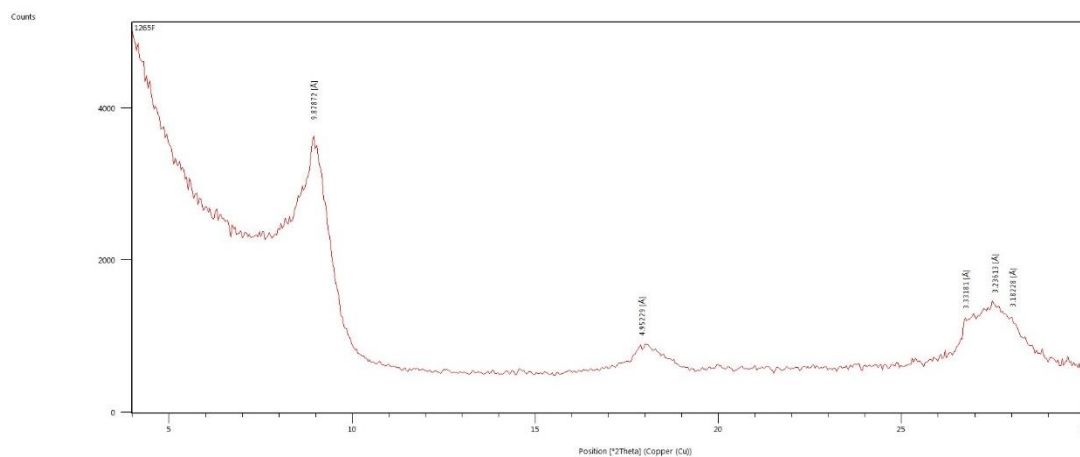


Figure B.6 : XRD patterns for clay fraction in Esenboğa Clay (2).

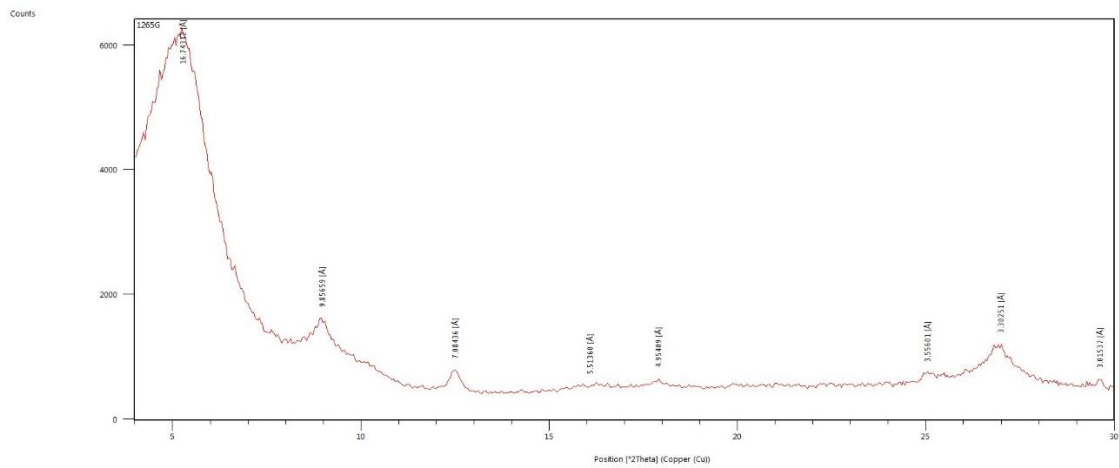


Figure B.7 : XRD patterns for clay fraction in Esenboğa Clay (3).

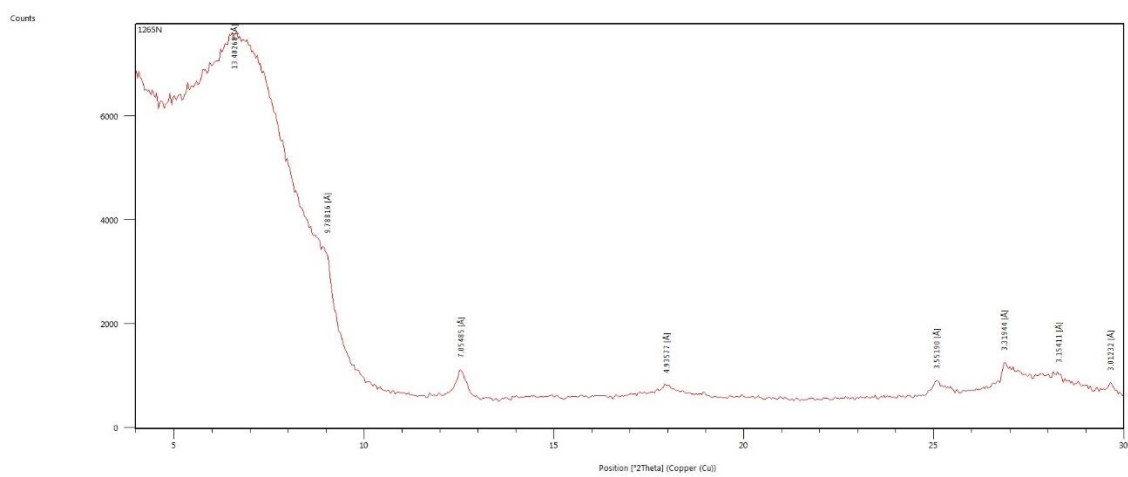


Figure B.8 : XRD patterns for clay fraction in Esenboğa Clay (4).



T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Müşteri : Odtü İnşaat Mühendisliği Bölümü
Geoteknik Bilim Dalı

Üniversiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

Rapor Tarihi : 09/06/2017
Sayfa : 2/5

ANALİZ/TEST RAPORU

Rapor No : MAT-17001156

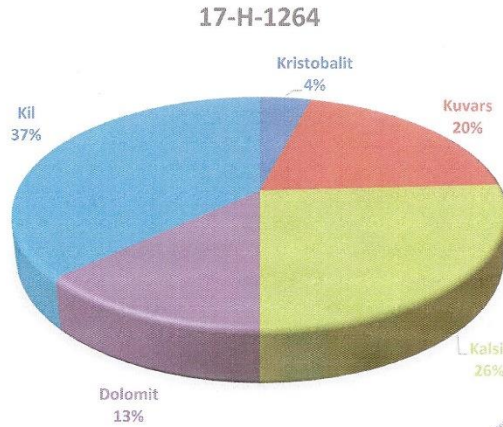
ANALİZ/TEST SONUÇLARI

Numune Kayıt No : 17-H-001264
Numune İşareti : BİLKENT CLAY

17-H-001264 no'lu numunenin Cu-X ışın tüplü Panalytical X'Pert Powder XRD analiz cihazı ile (4-70° tüm kayaç, 4-30° detay kil) gerçekleştirilen XRD analizi Panalytical X'Pert Highscore Plus programı ve ICSD veri tabanı kullanılarak Rietveld (1969) metodu ile kantitatif analizi yapılmıştır. Bu analiz sonucuna göre numune;

Tüm Kayaç:

Kuars: % 20,
Kristobalit: % 4,
Kalsit: % 26,
Dolomit: % 13,
Kil: % 37.



Bülent BAŞARA
Jeoloji Yüksek Mühendisi
Analiz/Test Sorumlusu

Dr. Ebru KAVUKÇU
Birim Yöneticisi
Jeoloji Yüksek Müh.



Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İmzasız ve mühürlü rapor geçersizdir.
Sonuçlar sadece analiz testi yapılan numuneye aittir.

KY.FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013

Figure B.9 : XRD analysis results for Bilkent Clay (1).



T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Müşteri : Odtü İnşaat Mühendisliği Bölümü
Geoteknik Bilim Dalı

Üniversiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

Rapor Tarihi : 09/06/2017
Sayfa : 3/5

ANALİZ/TEST RAPORU

Rapor No : MAT-170001156

ANALİZ/TEST SONUÇLARI

Detay Kil:

Tüm kayaç analizinde % 37 olarak tespit edilen kil minerallerinin oranı;
Paligorskit: % 100.

17-H-1264



Paligorskit
100%

Not: Yapılan bu XRD analizleri; petrografik, XRF ve SEM analizleri ile desteklenmektedir. Sonuçlar sadece analiz/testi yapılan numuncye aittir.

Numunelere ait XRD grafikleri .jpeg formatında ekte sunulmuştur.

Bülent BAŞARA
Jeoloji Yüksek Mühendisi
Analiz/Test Sorumlusu

Dr. Ebru KAVUKÇU
Birim Yöneticisi
Jeoloji Yüksek Müh.



Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İmzasız ve mühürlü rapor geçersizdir. Sonuçlar sadece analiz/testi yapılan numuncye aittir.

KY.FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013

Figure B.10 : XRD analysis results for Bilkent Clay (2).



T.C.
ENERJİ VE TABİİ KAYNAKLAR BAKANLIĞI
MADEN TETKİK VE ARAMA GENEL MÜDÜRLÜĞÜ
Maden Analizleri ve Teknolojisi Dairesi Başkanlığı

Müşteri : Odtü İnşaat Mühendisliği Bölümü Geoteknik
Bilim Dalı

Üniversiteler Mahallesi Dumlupınar Bulvarı
No: 139 06800 Çankaya/ANKARA
Tel: 0312 201 10 00 pbx Faks: 0312 287 54 09
http://www.mta.gov.tr numune@mta.gov.tr

Rapor Tarihi : 14/06/2017
Sayfa: 2 / 2

ANALİZ/TEST RAPORU

Rapor No : MAT-17002853

ANALİZ/TEST SONUÇLARI

Numune Kayıt No : 17-H-003805
Numune İşareti : BİLKENT CLAY

Analiz / Test Adı: X-Işını Kırınımı (XRD) Yöntemi ile Kalitatif Detay Kil Analizi

Mineral Adı: Kuvars
Kalsit
Dolomit
Paligorskit

Bülent BAŞARA
Jeoloji Yüksek Müh.
Analiz/Test Sorumlusu

6 Haziran 2017

Dr. Ebru KAVUKÇU
Jeoloji Yüksek Müh.
Birim Yöneticisi
Mühür/Tarih

Rapor tamamen veya kısmen çoğaltılamaz/yayınlanamaz. İmzasız ve mühürlü rapor geçersizdir.
Sonuçlar sadece analiz/testi yapılan numuneye aittir.

KY FR.5.10.2/1 Rev.No/Tarih: 05/13.05.2013



Figure B.11 : XRD analysis results for Bilkent Clay (3).

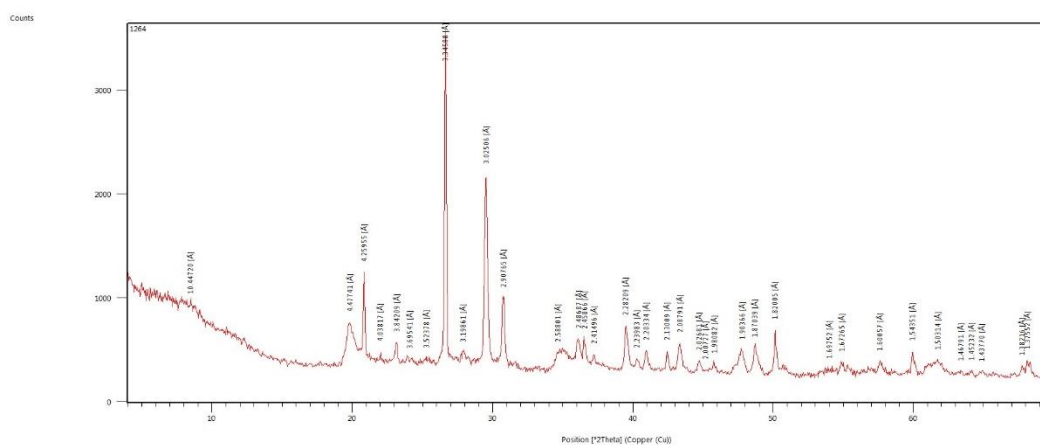


Figure B.12 : X-Ray diffractograms of Bilkent Clay for bulk sample.

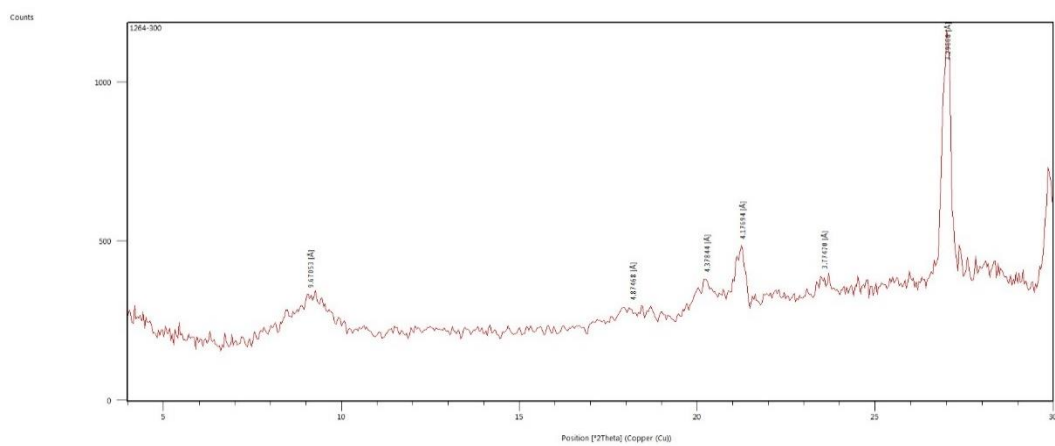


Figure B.13 : XRD patterns for clay fraction in Bilkent Clay (1).

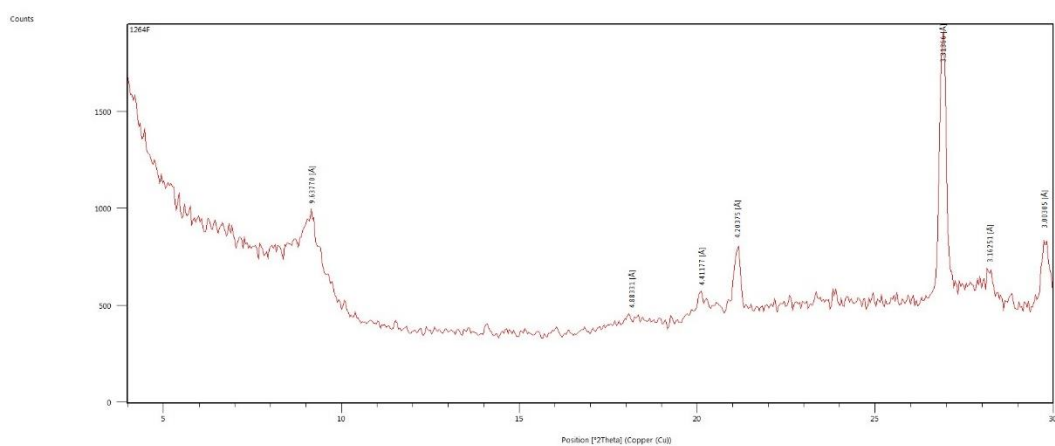
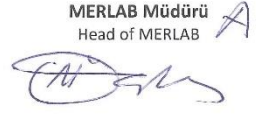


Figure B.14 : XRD patterns for clay fraction in Bilkent Clay (2).

APPENDIX C

		ORTA DOĞU TEKNİK ÜNİVERSİTESİ MERKEZ LABORATUVARI AR-GE EĞİTİM VE ÖLÇME MERKEZİ Üniversiteler Mah. Dumlupınar Blv. No:1, 06800 Çankaya Ankara/TÜRKİYE Tel: +90 312 210 6421 Fax: +90 312 210 6425 e-posta:merlab@metu.edu.trhttp://www.merlab.odtu.edu.tr	
DENEY RAPORU ANALYSIS REPORT			
Müşterinin Adı/Adresi Customer name/address		Orta Doğu Teknik Üniversitesi İnşaat Mühendisliği	
Evrak Kayıt No Record No		19324	
Numune Bilgileri Name and identity of sample		3 adet numune	
Numunenin Kabul Tarihi The date of the receipt of sample		19 / 01 / 2017	
Laboratuvar Adı Name of laboratory		Kimyasal Analiz Laboratuvarı (KAL)	
Deney Metodu Analysis method		X-Işını Floresans Spektrometresi (XRF)	
Deneyin Yapıldığı Tarih Date of analysis		20 / 01 / 2017	
Açıklamalar Remarks		Örnekler 65°C'de 1 gece 105°C'de 2 saat kurutulduktan sonra binder eklenmeden pellet olarak hazırlandı ve analiz edilmiştir. Bor-Uranyum aralığında çalışılan örneklerin sonuçları oksit formunda eklenmiştir.	
Raporun Sayfa Sayısı Number of pages of the report		1	
Ekler (ve sayfa sayıları) Attachments (and their page numbers)		10 sayfa cihaz çıktısı	
ODTÜ Merkez Laboratuvarı AR-GE Eğitim ve Ölçme Merkezi'ne gönderilen numune(ler) üzerinde istenilen deneyler yapılmıştır. Deney ve/veya ölçüm sonuçları ve genişletilmiş ölçüm belirsizlikleri (olması halinde) bu raporun takip eden sayfalarında verilmektedir. The requested analyse(s) were performed on the sample(s) sent to the METU Central Laboratory R&D Training and Measurement Center. The test and/or measurement results and the uncertainties (if applicable) with confidence level are given on the following pages which are a part of this report.			
Tarih Date 20 / 01 / 2017		Deney Sorumlusu Person in charge of test Salih Kaan Kırdeciler	
		MERLAB Müdürü Head of MERLAB 	

Bu rapor, MERLAB'ın yazılı izni olmadan kısmen de olsa kopyalanıp çoğaltılamaz. Sonuçlar sadece deneyi yapılan numuneye aittir. İmzasız raporlar geçersizdir. This report can not be partially copied or reproduced without official permission of MERLAB. The results only belong to the analysed sample. The reports without signature are invalid.
 FRM-NKB-03Rev.No/Tarih: 00/-

1/2

Figure C.1 : XRF analysis results.

Peak Identification Result					
Sample : 19324-1Esenboga			Measured date : 2017- 1-20 09:24		
Application : EZS101XNV			File : 19324-1Esenboga		
Spectrum	No.	Peak position (deg)	Peak int. (kcps)	BG int. (kcps)	Element line
Heavy	1	11.086	2.383	14.257	Ba-KA
	2	15.573	23.585	26.592	Rh-KB1
	3	16.375	24.759	30.608	Rh-KB1-Compton
	4	17.554	122.950	34.590	Rh-KA
	5	18.387	91.049	50.831	Rh-KA-Compton Nb-KB1
	6	20.045	4.650	20.468	Zr-KB1
	7	21.363	3.457	15.440	Nb-KA
	8	22.497	33.474	13.292	Zr-KA Sr-KB1
	9	23.755	4.511	11.429	Rb-KB1
	10	25.137	58.823	9.821	Sr-KA
	11	26.601	8.079	8.598	Rb-KA
	12	33.940	1.587	4.066	As-KA
	13	41.785	2.528	1.994	Zn-KA
	14	43.702	0.740	1.656	Ni-KB1
	15	45.020	1.262	1.518	Cu-KA
	16	48.662	2.195	1.208	Ni-KA
	17	51.742	126.990	2.182	Fe-KB1
	18	57.511	657.250	3.151	Fe-KA Mn-KB1
	19	62.963	7.445	0.760	Mn-KA Cr-KB1
	20	69.328	1.073	0.392	Cr-KA V -KB1
	21	77.024	0.850	0.272	V -KA
	22	77.241	2.402	0.277	Ti-KB1
	23	79.261	0.268	0.213	
	24	86.132	13.694	0.100	Ti-KA
Ca-KA	1	113.113	393.403	1.876	Ca-KA
K -KA	1	135.084	4.467	1.626	K -SKA4
	2	136.664	199.762	1.612	K -KA
Cl-KA	1	92.900	0.723	0.212	Cl-KA
S -KA	1	110.780	2.185	0.203	S -KA
	2	113.148	0.046	0.169	
P -KA	1	139.057	0.352	0.345	P -SKA3
	2	141.131	8.048	0.340	P -KA
Si-KA	1	109.060	1044.145	6.814	Si-KA
Al-KA	1	142.242	29.345	12.429	Al-SKA3
	2	144.766	465.965	5.639	Al-KA
Mg-KA	1	38.233	53.498	1.319	Mg-KA
Na-KA	1	46.438	7.182	0.239	Na-KA
F -KA	1	74.665	0.068	0.047	F -KA
O -KA	1	52.869	24.536	0.564	O -KA
N -KA	1	49.887	0.094	0.413	

Rigaku

Figure C.2 : XRF analysis results for Esenboğa Clay (1).

Rigaku

Figure C.3 : XRF analysis results for Esenboğa Clay (2).

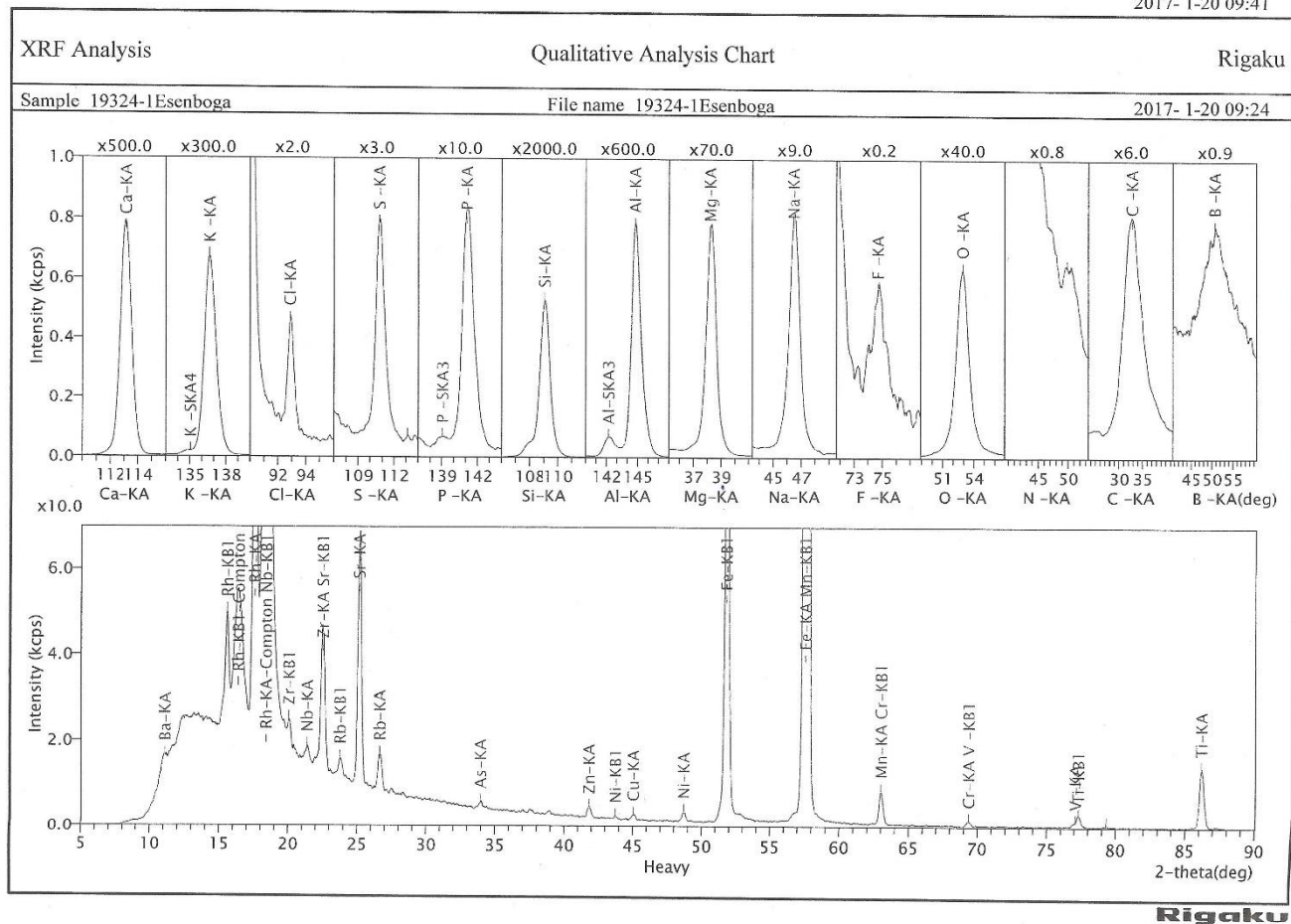


Figure C.4 : XRF analysis results for Esenboga Clay (3).

SQX Calculation Result							
Sample : 19324-1Esenboga				Date analyzed : 2017- 1-20 09:24			
Application : EZS101XNV				Balance :			
Model : Bulk				Matching library:			
				File : 19324-1Esenboga			
No.	Component	Result	Unit	Det.limit	El.line	Intensity	w/o normal
1	SiO ₂	48.7	wt%	0.01655	Si-KA	1044.1450	48.7789
2	Al ₂ O ₃	16.7	wt%	0.01061	Al-KA	465.9650	16.7577
3	CO ₂	11.0	wt%	0.18658	C -KA	4.2653	11.0018
4	Fe ₂ O ₃	7.05	wt%	0.02117	Fe-KB1	126.9902	7.0658
5	CaO	6.57	wt%	0.00371	Ca-KA	393.4030	6.5766
6	MgO	4.30	wt%	0.01051	Mg-KA	53.4983	4.3022
7	K ₂ O	2.59	wt%	0.00263	K -KA	199.7622	2.5960
8	Na ₂ O	1.53	wt%	0.01225	Na-KA	7.1825	1.5306
9	TiO ₂	0.946	wt%	0.00623	Ti-KA	13.6935	0.9478
10	P ₂ O ₅	0.231	wt%	0.00218	P -KA	8.0485	0.2309
11	MnO	0.115	wt%	0.00346	Mn-KA	7.4452	0.1150
12	SO ₃	0.0753	wt%	0.00209	S -KA	2.1854	0.0755
13	SrO	0.0658	wt%	0.00089	Sr-KA	58.8231	0.0659
14	Cl	0.0399	wt%	0.00291	Cl-KA	0.7234	0.0400
15	Cr ₂ O ₃	0.0260	wt%	0.00394	Cr-KA	1.0727	0.0261
16	NiO	0.0146	wt%	0.00185	Ni-KA	2.1952	0.0146
17	ZnO	0.0097	wt%	0.00141	Zn-KA	2.5280	0.0097
18	Rb ₂ O	0.0094	wt%	0.00088	Rb-KA	8.0787	0.0095
19	CuO	0.0065	wt%	0.00162	Cu-KA	1.2624	0.0065
20	Nb ₂ O ₅	0.0033	wt%	0.00097	Nb-KA	3.4573	0.0034

Rigaku

Figure C.5 : XRF analysis results for Esenboğa Clay (4).

Peak Identification Result					
Sample : 19324-2Bilkent			Measured date : 2017- 1-20 09:41		
Application : EZS101XNV			File : 19324-2Bilkent		
Spectrum	No.	Peak position (deg)	Peak int. (kcps)	BG int. (kcps)	Element line
Heavy	1	15.572	23.426	26.848	Rh-KB1
	2	16.404	26.579	30.056	Rh-KB1-Compton
	3	17.556	125.787	35.291	Rh-KA
	4	18.388	91.480	51.427	Rh-KA-Compton
	5	20.020	4.779	21.137	Nb-KB1
	6	21.334	3.401	15.404	Zr-KB1
	7	22.088	3.130	14.299	Nb-KA
	8	22.485	40.932	13.413	Sr-KB2
	9	23.748	4.411	11.865	Zr-KA
	10	25.136	84.365	10.338	Rb-KB1
	11	26.601	8.189	8.726	Sr-KA
	12	33.947	1.239	4.163	Rb-KA
	13	41.795	1.691	2.192	As-KA
	14	45.024	1.288	1.472	Zn-KA
	15	48.652	1.795	1.178	Cu-KA
	16	51.743	84.943	1.400	Ni-KA
	17	56.693	1.589	0.954	Fe-KB1
	18	57.508	449.820	0.980	Mn-KB1
	19	62.961	5.195	0.598	Fe-KA
	20	69.351	0.840	0.370	Mn-KA
	21	77.247	1.807	0.402	Cr-KA
	22	79.196	0.285	0.191	Ti-KB1
	23	86.126	10.313	0.197	Ti-KA
Ca-KA	1	113.115	979.667	3.433	Ca-KA
K -KA	1	135.105	2.567	1.328	K -SKA4
	2	136.665	119.067	1.311	K -KA
Cl-KA	1	92.902	0.987	0.257	Cl-KA
S -KA	1	110.783	3.978	0.243	S -KA
P -KA	1	139.047	0.094	0.226	P -SKA3
	2	141.134	1.858	0.222	P -KA
	3	143.341	0.058	0.218	
Si-KA	1	109.059	939.070	6.249	Si-KA
Al-KA	1	142.242	22.875	9.574	Al-SKA3
	2	144.764	357.777	4.422	Al-KA
Mg-KA	1	38.232	49.916	1.125	Mg-KA
Na-KA	1	46.436	4.480	0.214	Na-KA
F -KA	1	74.540	0.033	0.040	F -KA
O -KA	1	52.862	20.590	0.481	O -KA
C -KA	1	32.515	8.306	0.648	C -KA
B -KA	1	50.706	0.339	0.378	B -KA

Rigaku

Figure C.6 : XRF analysis results for Bilkent Clay (1).

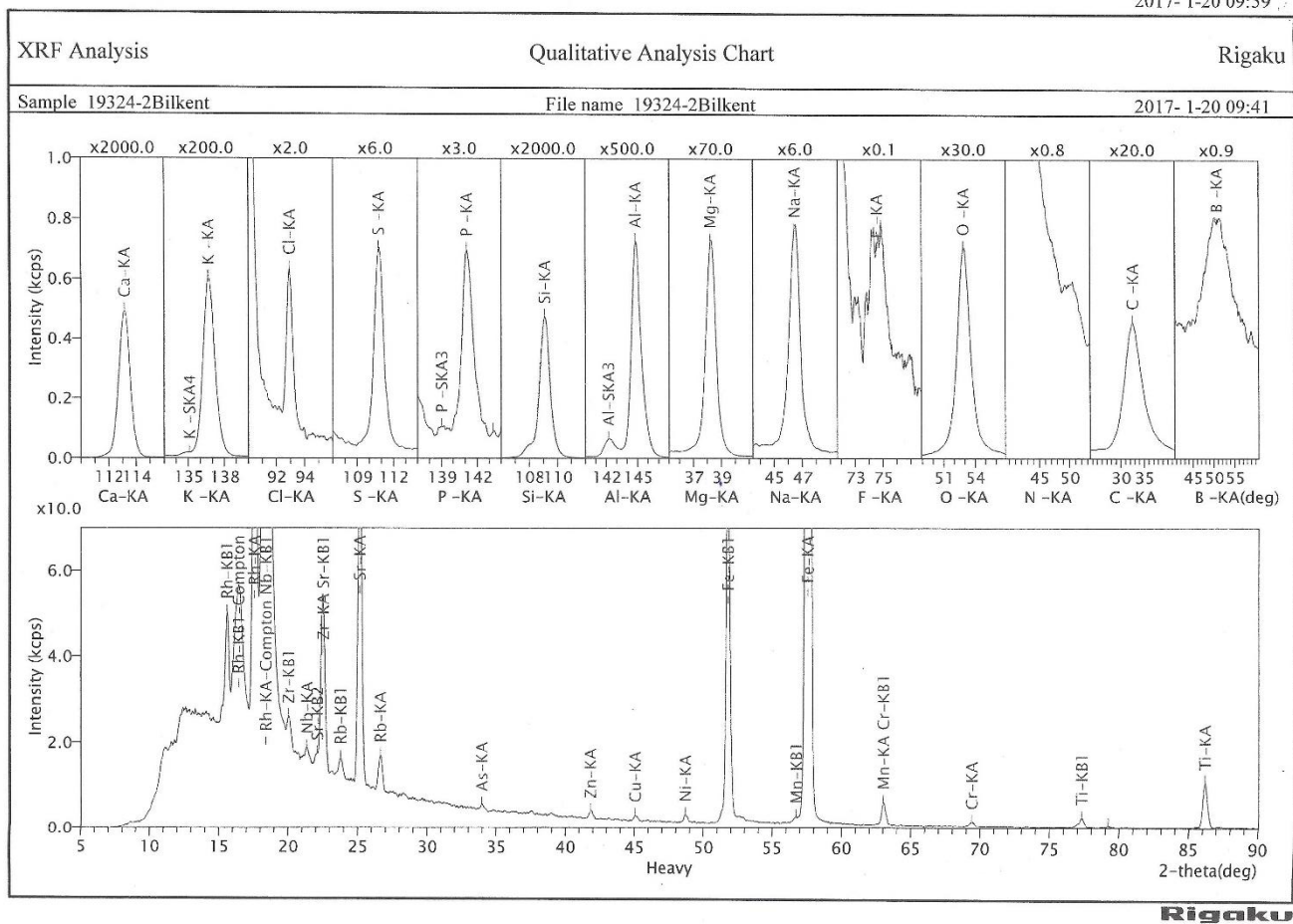


Figure C.7 : XRF analysis results for Bilkent Clay (2).

271

Peak Identification Result					
Sample : 19324-3Kireç			Measured date : 2017- 1-20 09:59		
Application : EZS101XNV			File : 19324-3Kireç		
Spectrum	No.	Peak position (deg)	Peak int. (kcps)	BG int. (kcps)	Element line
Heavy	1	15.567	22.566	17.465	Rh-KB1
	2	16.387	17.532	20.473	Rh-KB1-Compton
	3	17.553	125.239	20.524	Rh-KA
	4	18.388	61.359	34.553	Rh-KA-Compton
	5	22.397	5.561	10.092	Sr-KB1
	6	25.137	24.350	7.745	Sr-KA
	7	48.657	0.812	0.886	Ni-KA
	8	57.505	2.545	0.365	Fe-KA
	9	76.911	0.232	0.137	V -KA
	1	113.112	5250.576	16.293	Ca-KA
K -KA	1	136.643	2.016	1.601	K -KA
Cl-KA	1	92.869	0.765	0.485	Cl-KA
S -KA	1	110.782	41.454	0.467	S -KA
P -KA	1	139.146	0.073	0.217	P -SKA3
	2	141.113	0.803	0.235	P -KA
Si-KA	1	109.067	7.921	0.147	Si-KA
Al-KA	1	142.158	0.237	0.190	Al-SKA3
	2	144.769	3.874	0.158	Al-KA
Mg-KA	1	38.229	19.664	0.409	Mg-KA
Na-KA	1	46.435	0.085	0.110	Na-KA
	2	47.924	0.025	0.086	
F -KA	1	74.864	0.032	0.032	F -KA
O -KA	1	52.798	10.318	0.260	O -KA
C -KA	1	32.425	15.687	0.896	C -KA
B -KA	1	50.611	0.420	0.455	B -KA

Rigaku

Figure C.9 : XRF analysis results for lime (1).

Figure C.11 : XRF analysis results for lime (3).

		ORTA DOĞU TEKNİK ÜNİVERSİTESİ MERKEZ LABORATUVARI AR-GE EĞİTİM VE ÖLÇME MERKEZİ Üniversiteler Mah. Dumlupınar Blv. No:1, 06800 Çankaya Ankara/TÜRKİYE Tel: +90 312 210 6421 Fax: +90 312 210 6425 e-posta:merlab@metu.edu.trhttp://www.merlab.odtu.edu.tr	
DENEY RAPORU ANALYSIS REPORT			
Müşterinin Adı/Adresi Customer name/address		ODTÜ	
Evrak Kayıt No Record No	16916		
Numune Bilgileri Name and identity of sample	16916-1sivasBU; 16916-2sivasFU 16916-3somaBU; 16916-4somaFU		
Numunenin Kabul Tarihi The date of the receipt of sample	01/04/2016		
Laboratuvar Adı Name of laboratory	KAL		
Deney Metodu Analysis method	X-Işını Floresans Spektrometresi		
Deneyin Yapıldığı Tarih Date of analysis	01/04/2016		
Açıklamalar Remarks	2 adet uçucu kül örneği 105°C'de 2 saat süreyle kurutulduktan sonra herhangi bir binder eklenmeden pellet olarak hazırlandı. 30 mm örnek tutucu yardımıyla analiz gerçekleştirildi. Analiz hem Bor-Uranyum hem de Flor-Uranyum aralığında yapıldı. Ekteki sonuçlar hem oksit hem de metal formunda hazırlanmıştır.		
Raporun Sayfa Sayısı Number of pages of the report	1		
Ekler (ve sayfa sayıları) Attachments (and their page numbers)	20 sayfa cihaz çıktısı		
ODTÜ Merkez Laboratuvarı AR-GE Eğitim ve Ölçme Merkezi'ne gönderilen numune(ler) üzerinde istenilen deneyler yapılmıştır. Deney ve/veya ölçüm sonuçları ve genişletilmiş ölçüm belirsizlikleri (olması halinde) bu raporun takip eden sayfalarında verilmektedir. The requested analyse(s) were performed on the sample(s) sent to the METU Central Laboratory R&D Training and Measurement Center. The test and/or measurement results and the uncertainties (if applicable) with confidence level are given on the following pages which are a part of this report.			
Tarih Date	Deney Sorumlusu Person in charge of test	MERLAB Müdürü Head of MERLAB	
01/04/2016	Salih Kaan Kırdeciler	A.	

Bu rapor, MERLAB' ın yazılı izni olmadan kısmen de olsa kopyalanıp çoğaltılamaz. Sonuçlar sadece deneyi yapılan numuneye aittir. İmzasız raporlar geçersizdir. This report can not be partially copied or reproduced without official permission of MERLAB. The results only belong to the analysed sample. The reports without asiganture are unvalid.
FRM-NKB-03Rev.No/Tarih: 00/-

1/1

Figure C.12 : XRF analysis results for fly ash (1).

Peak Identification Result					
Sample : 16916somaBU			Measured date : 2016- 4- 1 14:20		
Application : EZS101XNV			File : 16916somaBU		
Spectrum	No.	Peak position (deg)	Peak int. (kcps)	BG int. (kcps)	Element line
Heavy	1	11.043	6.059	14.931	Ba-KA
	2	15.578	23.121	26.571	Rh-KB1
	3	16.400	23.628	29.696	Rh-KB1-Compton
	4	17.557	124.859	34.451	Rh-KA
	5	18.405	88.097	49.452	Rh-KA-Compton
	6	20.037	5.019	20.897	Zr-KB1
	7	21.349	3.978	15.424	Y -KB1
	8	21.993	2.158	14.216	Sr-KB2
	9	22.509	37.599	13.251	Zr-KA Sr-KB1
	10	23.780	8.548	11.268	Y -KA Rb-KB1
	11	25.138	50.607	9.813	Sr-KA
	12	26.601	10.476	8.535	Rb-KA
	13	28.247	2.597	7.132	Pb-LB1
	14	30.414	1.437	5.984	As-KB1
	15	33.968	8.662	4.271	As-KA Pb-LA
					Ga-KB1
	16	37.518	1.381	3.174	Zn-KB1
	17	38.902	1.637	2.754	Ga-KA
	18	41.789	6.134	2.227	Zn-KA
	19	45.034	1.754	1.625	Cu-KA
	20	48.657	1.849	1.189	Ni-KA
	21	51.746	68.444	1.636	Fe-KB1
	22	57.512	358.875	1.965	Fe-KA Mn-KB1
	23	62.968	2.191	0.538	Mn-KA Cr-KB1
	24	69.329	1.065	0.360	Cr-KA V -KB1
	25	77.007	1.440	0.206	V -KA
	26	77.254	2.533	0.198	Ti-KB1
	27	79.225	0.615	0.178	Ba-LB1
	28	86.136	12.441	0.161	Ti-KA
	29	87.127	0.476	0.258	Ba-LA
Ca-KA	1	113.065	731.755	2.711	Ca-KA
K -KA	1	136.618	124.485	1.706	K -KA
S -KA	1	110.737	57.607	0.563	S -KA
P -KA	1	139.106	0.442	0.580	P -SKA3
	2	141.095	11.749	0.530	P -KA
Si-KA	1	109.032	1042.007	7.059	Si-KA
Al-KA	1	142.229	55.736	23.736	Al-SKA3
	2	144.769	886.985	10.019	Al-KA
Mg-KA	1	38.193	17.677	1.312	Mg-KA
Na-KA	1	46.418	2.579	0.195	Na-KA

Rigaku

Figure C.13 : XRF analysis results for fly ash (2).

[illegible]

Figure C.14 : XRF analysis results for fly ash (3).

SQX Calculation Result							
Sample : 16916somaBU				Date analyzed : 2016- 4- 1 14:20			
Application : EZS101XNV		Model : Bulk		Balance :		Matching library:	
				File :		16916somaBU	
No.	Component	Result	Unit	Det.limit	El.line	Intensity	w/o normal
1	SiO2	45.8	wt%	0.01577	Si-KA	1042.0072	43.5030
2	Al2O3	26.6	wt%	0.01188	Al-KA	886.9850	25.2810
3	CaO	11.6	wt%	0.00423	Ca-KA	731.7551	11.0016
4	CO2	5.14	wt%	0.15333	C -KA	2.3830	4.8890
5	Fe2O3	4.38	wt%	0.02130	Fe-KB1	68.4441	4.1627
6	SO3	1.79	wt%	0.00306	S -KA	57.6071	1.6985
7	K2O	1.54	wt%	0.00269	K -KA	124.4853	1.4622
8	MgO	1.20	wt%	0.00892	Mg-KA	17.6766	1.1434
9	TiO2	1.02	wt%	0.00849	Ti-KA	12.4405	0.9715
10	Na2O	0.454	wt%	0.00967	Na-KA	2.5791	0.4313
11	P2O5	0.299	wt%	0.00247	P -KA	11.7490	0.2842
12	V2O5	0.0718	wt%	0.00736	V -KA	1.4401	0.0682
13	SrO	0.0594	wt%	0.00094	Sr-KA	50.6071	0.0565
14	MnO	0.0392	wt%	0.00344	Mn-KA	2.1907	0.0373
15	Cr2O3	0.0289	wt%	0.00478	Cr-KA	1.0649	0.0275
16	ZnO	0.0249	wt%	0.00156	Zn-KA	6.1335	0.0237
17	NiO	0.0130	wt%	0.00197	Ni-KA	1.8493	0.0124
18	Rb2O	0.0129	wt%	0.00092	Rb-KA	10.4764	0.0122

Rigaku

Figure C.16 : XRF analysis results for fly ash (5).

Figure C.17 : XRF analysis results for fly ash (6).

Peak Identification Result					
Sample : 16916somaFU			Measured date : 2016- 4- 1 14:36		
Application : EZS101XNV			File : 16916somaFU		
Spectrum	No.	Peak position (deg)	Peak int. (kcps)	BG int. (kcps)	Element line
Heavy	1	11.047	6.627	14.523	Ba-KA
	2	15.576	22.784	26.830	Rh-KB1
	3	16.381	24.201	29.844	Rh-KB1-Compton
	4	17.559	124.053	33.926	Rh-KA
	5	18.402	89.372	48.640	Rh-KA-Compton Nb-KB1
	6	20.032	4.678	21.439	Zr-KB1 U -LB1
	7	21.351	3.876	15.855	Nb-KA
	8	22.503	37.606	13.254	Zr-KA Sr-KB1
	9	23.764	8.647	11.404	Rb-KB1
	10	25.137	50.346	9.725	Sr-KA
	11	26.072	0.989	8.938	U -LA
	12	26.604	10.633	8.490	Rb-KA
	13	27.431	1.385	7.793	
	14	28.227	2.943	7.123	Pb-LB1
	15	30.421	1.525	5.739	As-KB1
	16	33.966	8.350	4.373	As-KA Pb-LA
					Ga-KB1
	17	37.506	1.663	3.070	Zn-KB1
	18	38.884	1.234	2.841	Ga-KA
	19	41.792	6.727	2.040	Zn-KA
	20	45.034	1.801	1.527	Cu-KA
	21	48.658	1.941	1.186	Ni-KA
	22	51.746	69.654	1.208	Fe-KB1
	23	57.512	357.453	1.937	Fe-KA Mn-KB1
	24	62.958	2.194	0.608	Mn-KA Cr-KB1
	25	69.322	0.913	0.395	Cr-KA V -KB1
	26	76.993	1.440	0.231	V -KA
	27	77.236	2.338	0.225	Ti-KB1
	28	79.219	0.608	0.241	Ba-LB1
	29	86.135	12.194	0.174	Ti-KA
	30	87.139	0.472	0.250	Ba-LA
Ca-KA	1	113.066	728.451	2.531	Ca-KA
	2	115.047	0.742	2.837	
K -KA	1	136.622	125.560	1.713	K -KA
Cl-KA	1	92.797	0.097	0.201	Cl-KA
S -KA	1	108.292	0.075	0.578	Pb-MA
	2	110.737	57.605	0.496	S -KA
P -KA	1	139.073	0.471	0.574	P -SKA3
	2	141.091	11.885	0.549	P -KA
Si-KA	1	109.031	1041.363	6.923	Si-KA

Rigaku

Figure C.18 : XRF analysis results for fly ash (7).

Rigaku

Figure C.19 : XRF analysis results for fly ash (8).

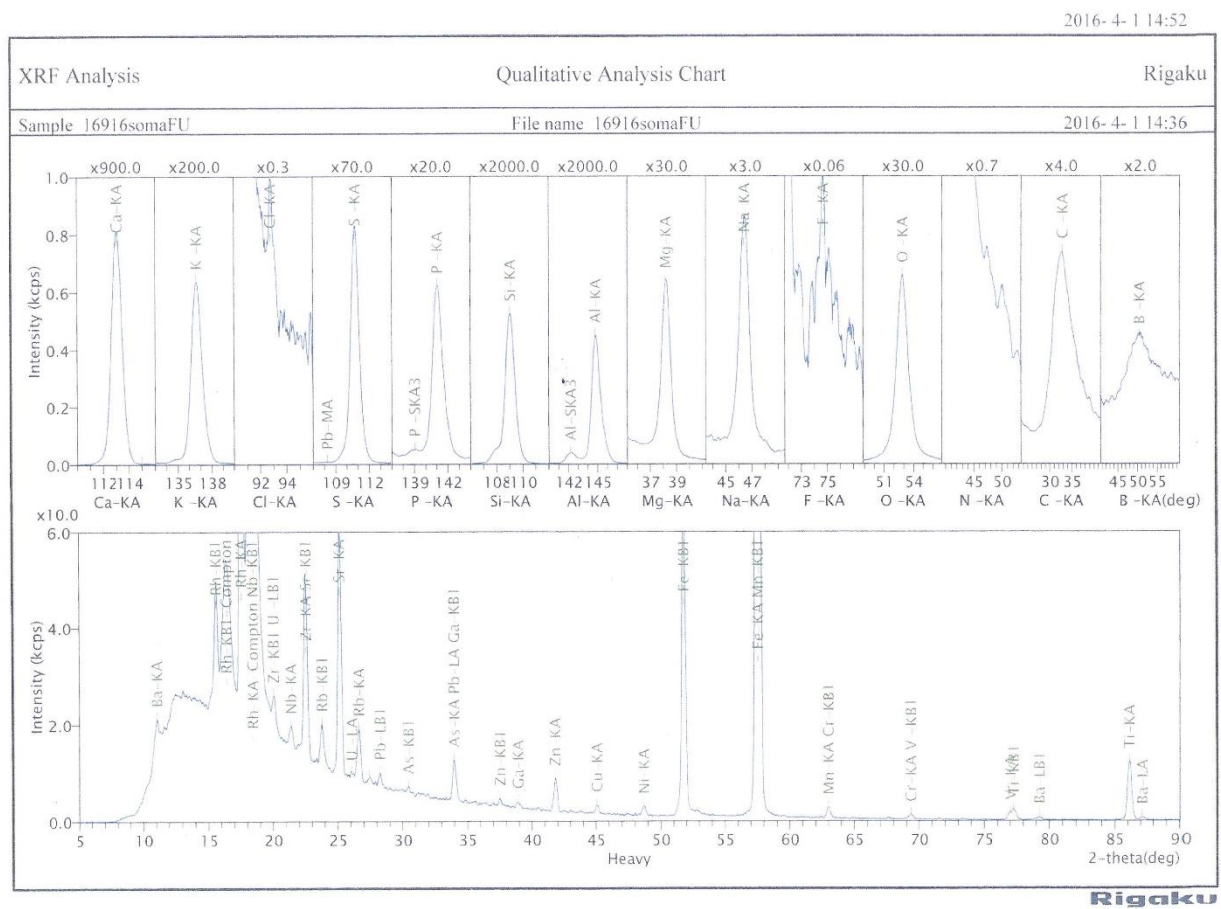


Figure C.20 : XRF analysis results for fly ash (9)

284

Figure C.22 : XRF analysis results for fly ash (11).

APPENDIX D

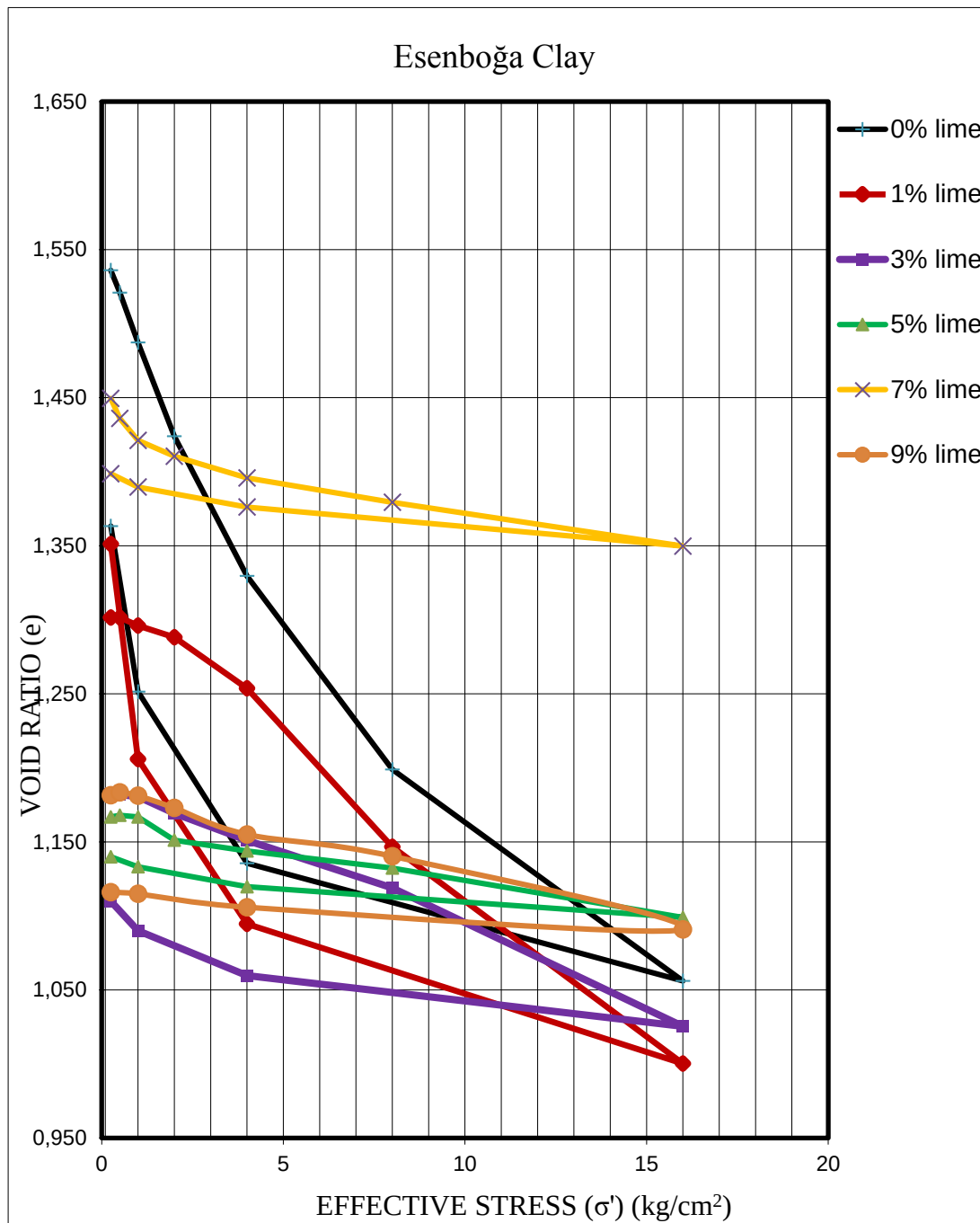


Figure D.1 : e - σ' relationship for natural and lime-treated soil samples without cure for Esenboğa Clay.

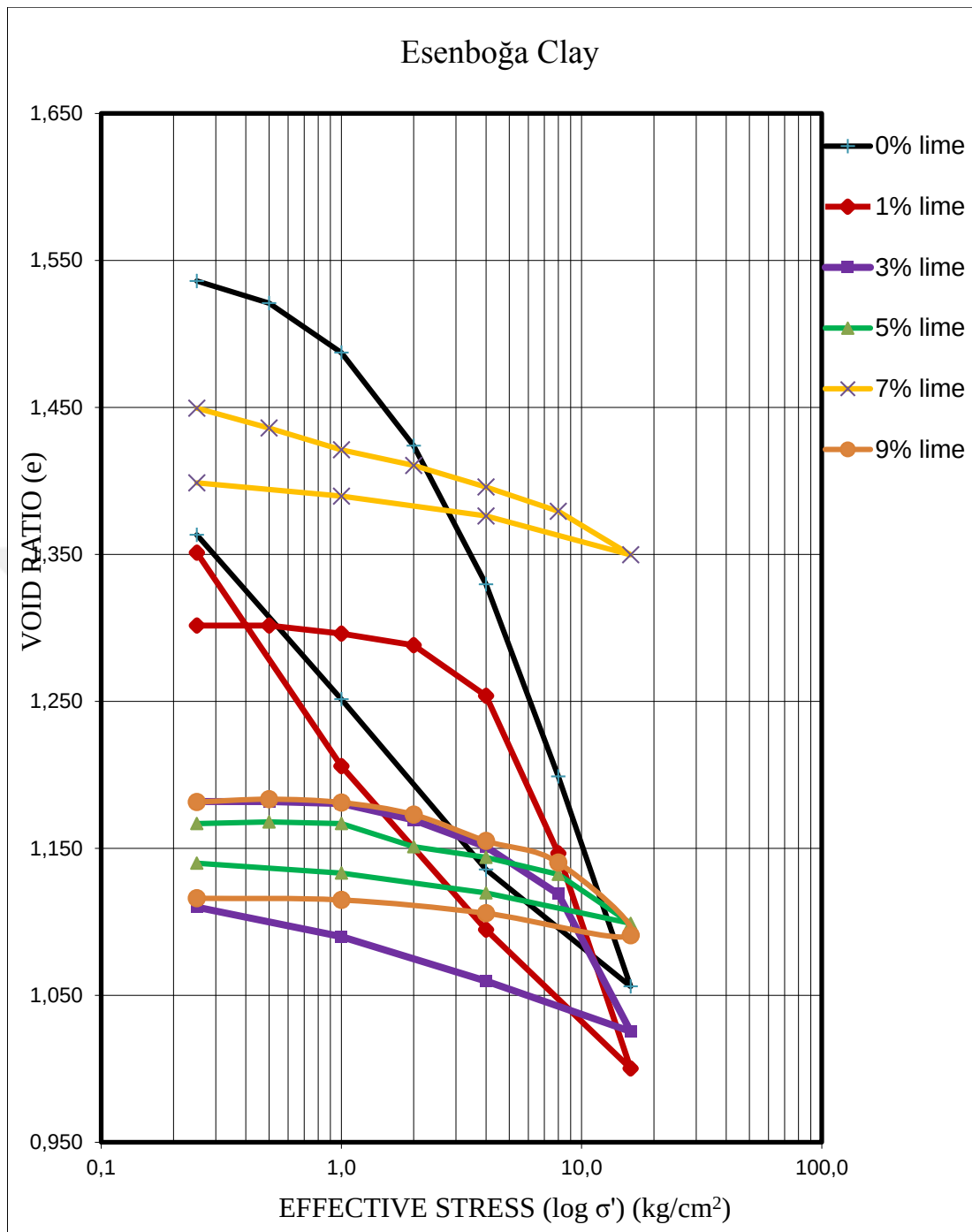


Figure D.2 : e-log σ' relationship for natural and lime-treated soil samples without cure for Esenboğa Clay.

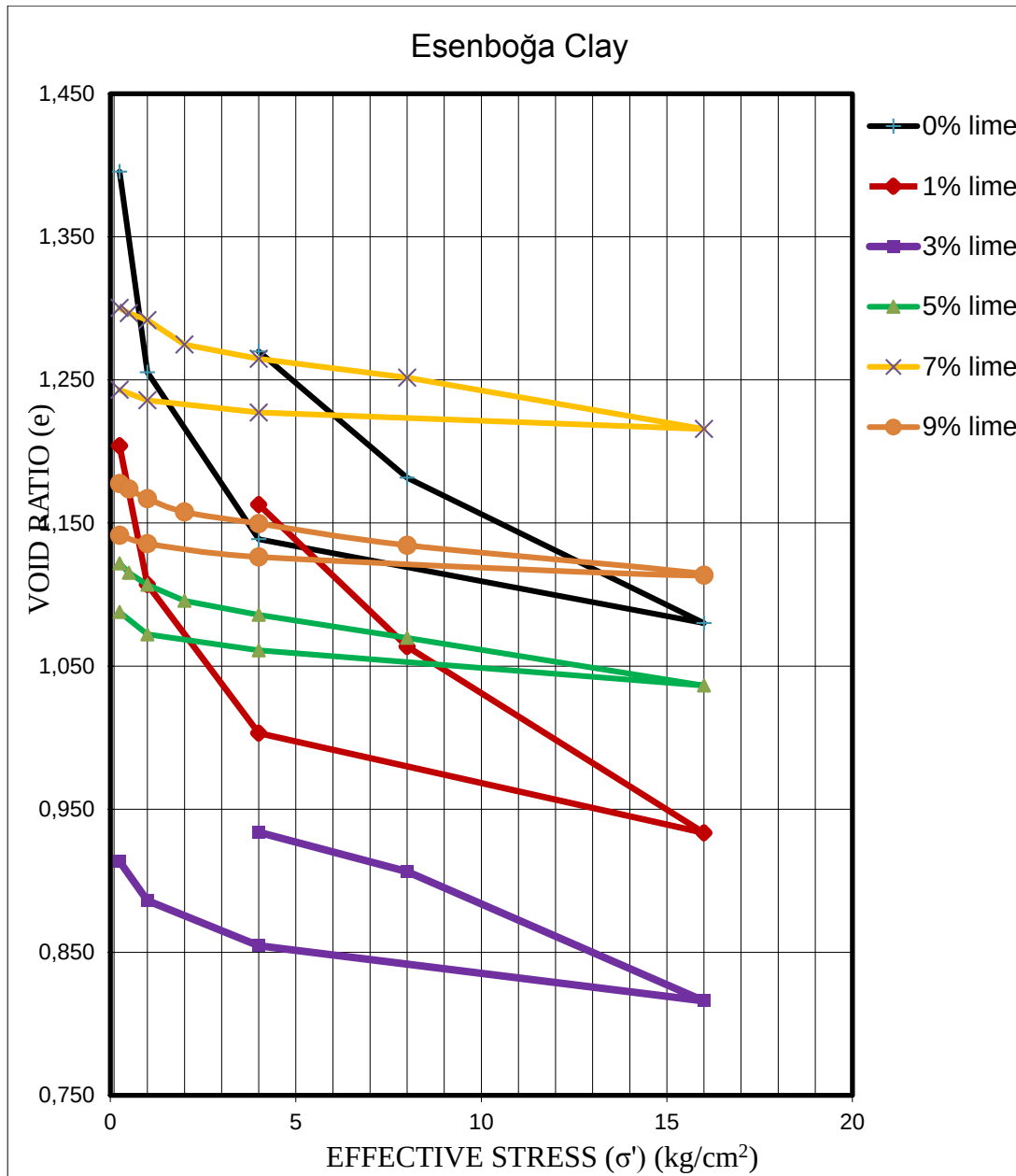


Figure D.3 : e - σ' relationship for natural and lime-treated soil samples for 7 days cure period for Esenboğa Clay.

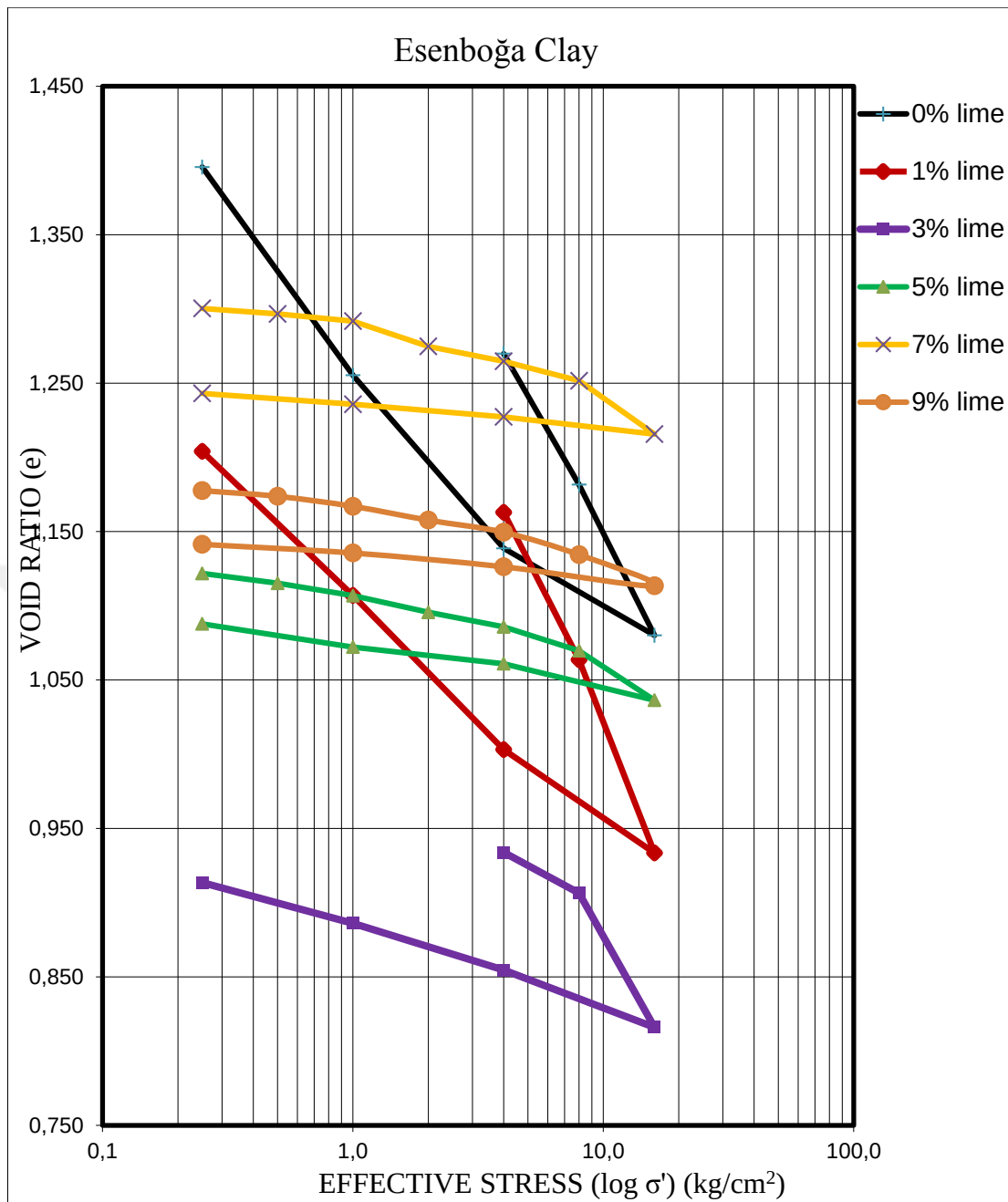


Figure D.4 : e-log σ' relationship for natural and lime-treated soil samples for 7 days cure period for Esenboğa Clay.

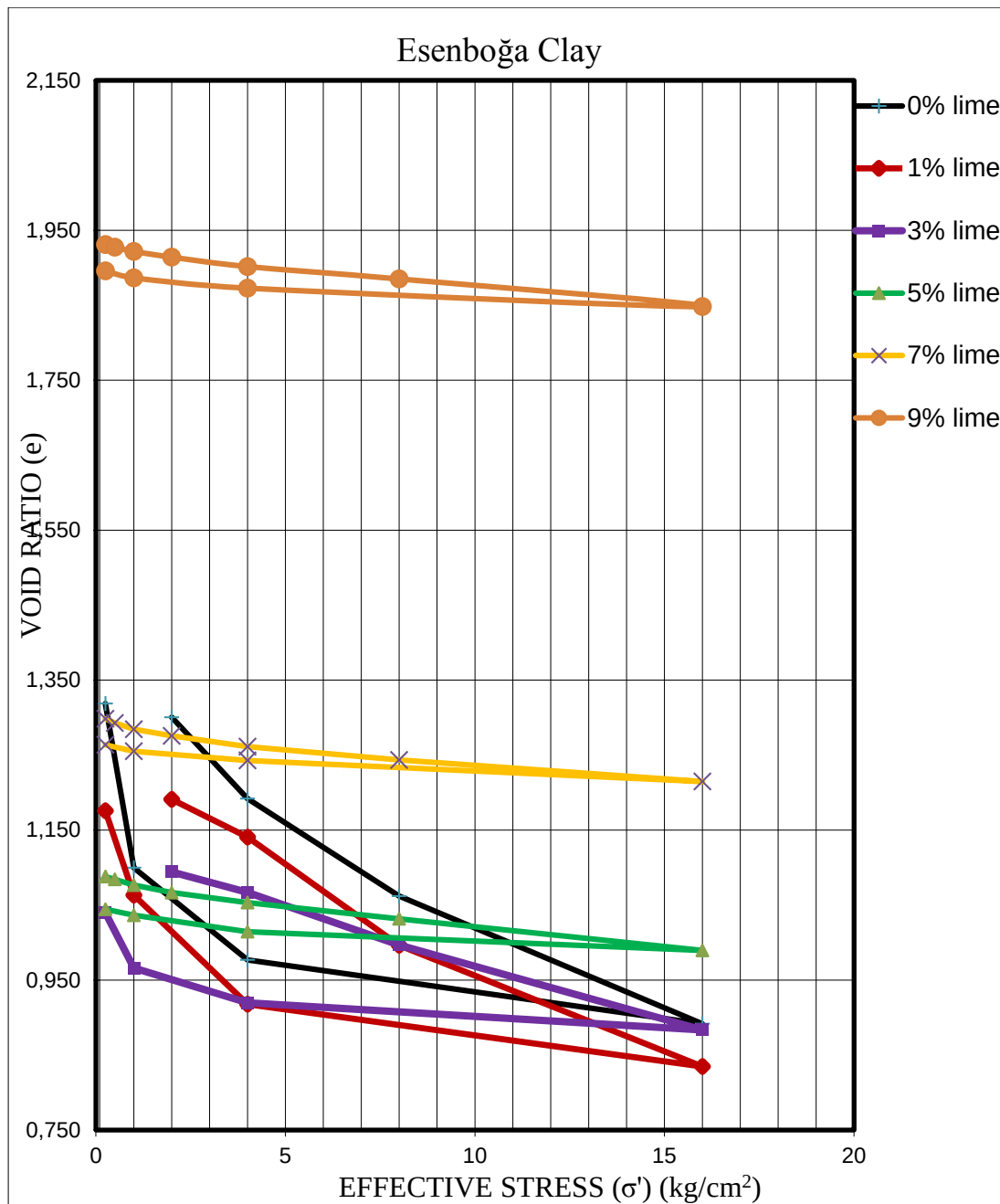


Figure D.5 : e - σ' relationship for natural and lime-treated soil samples for 28 days cure period for Esenboğa Clay.

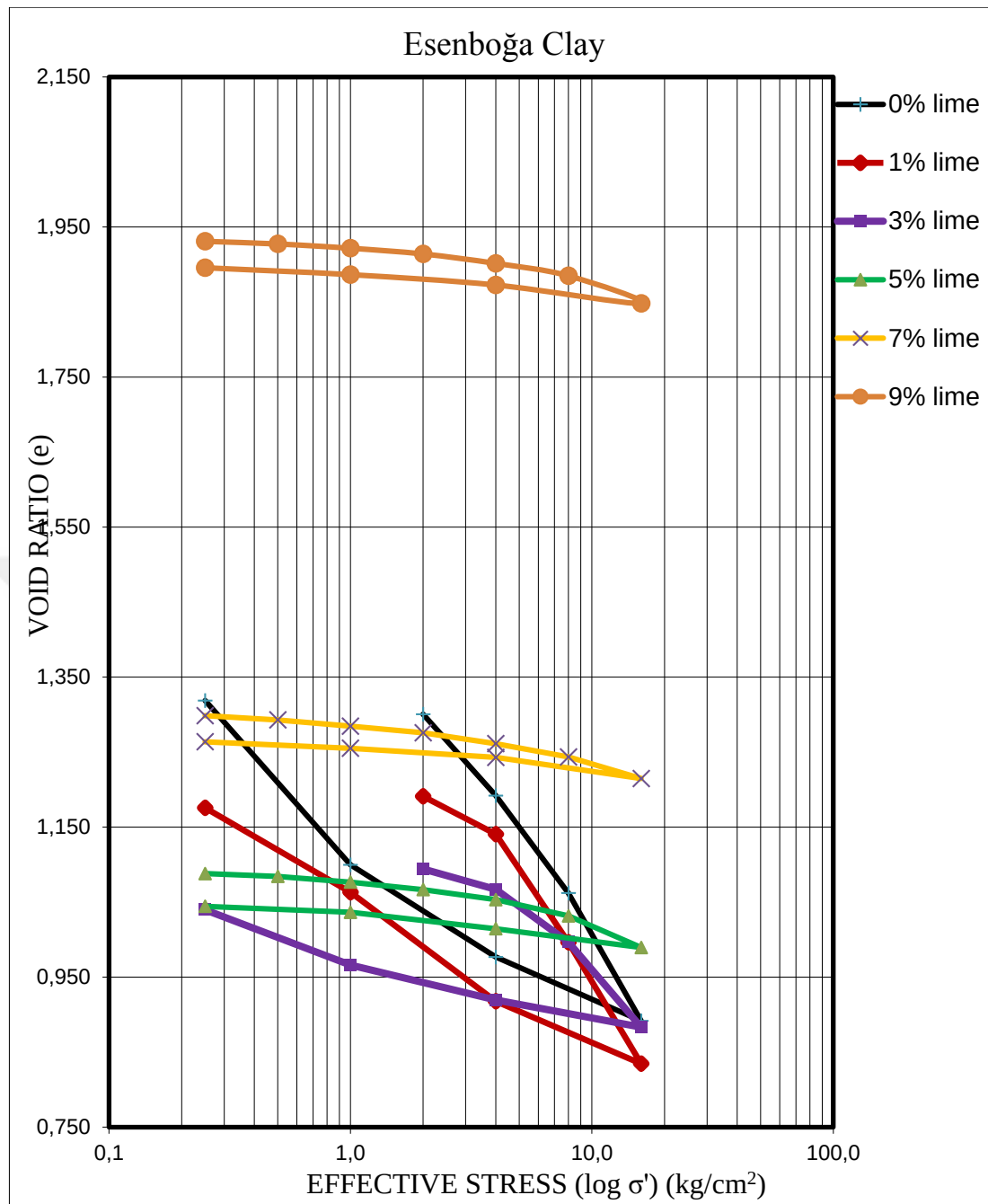


Figure D.6 : e - $\log \sigma'$ relationship for natural and lime-treated soil samples for 28 days cure period for Esenboğa Clay.

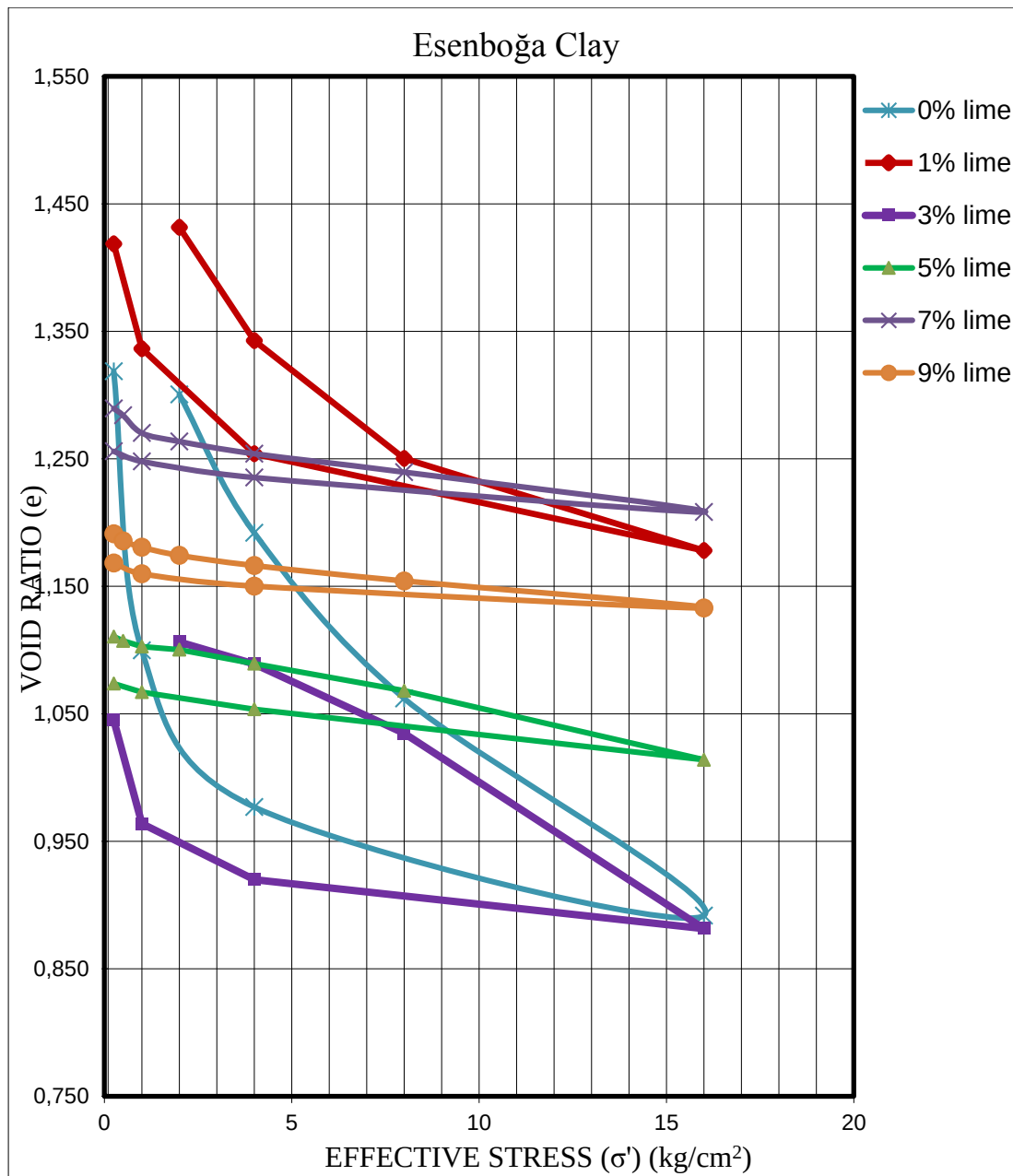


Figure D.7 : e - σ' relationship for natural and lime-treated soil samples for 56 days cure period for Esenboğa Clay.

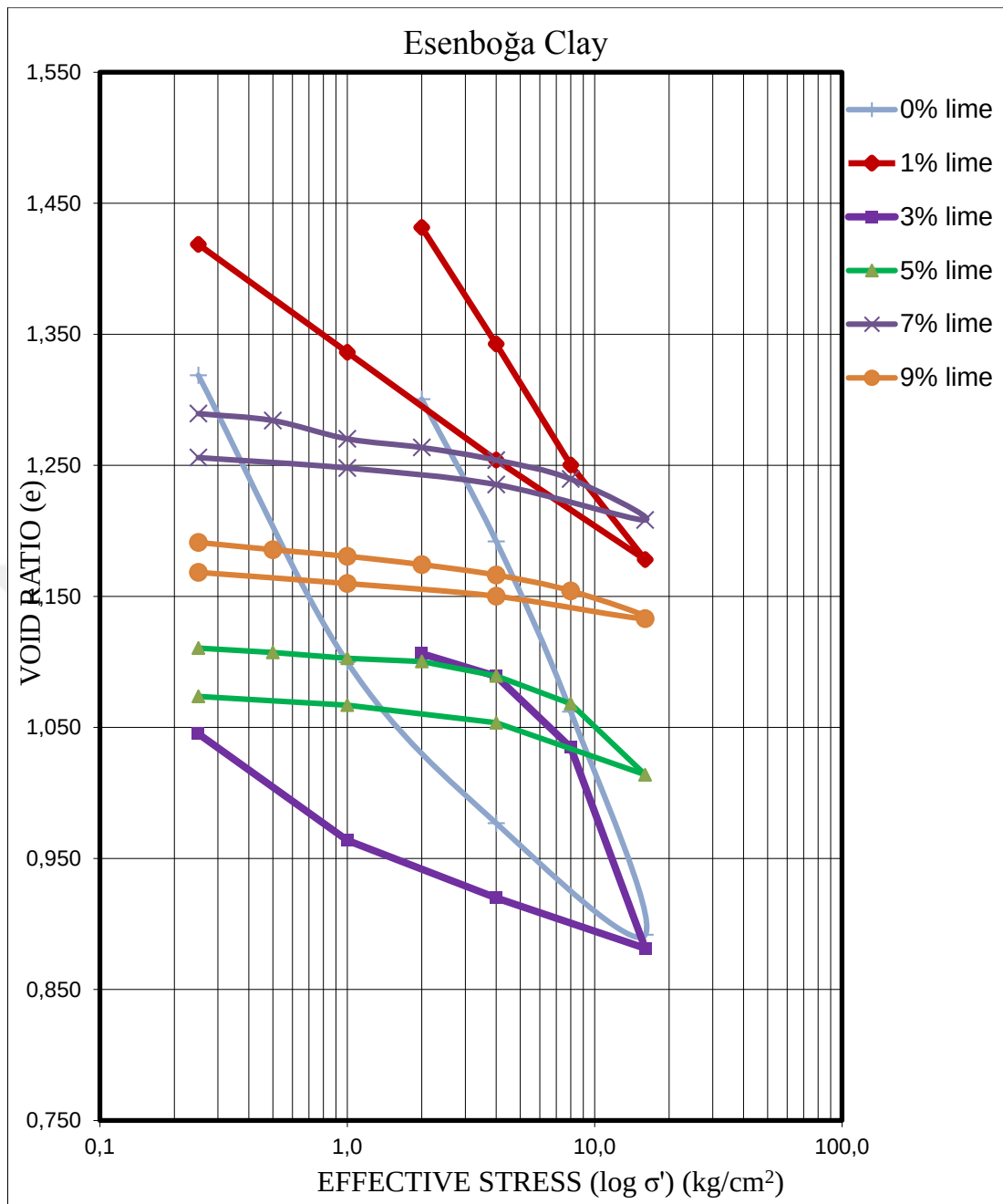


Figure D.8 : e-log σ' relationship for natural and lime-treated soil samples for 56 days cure period for Esenboğa Clay.

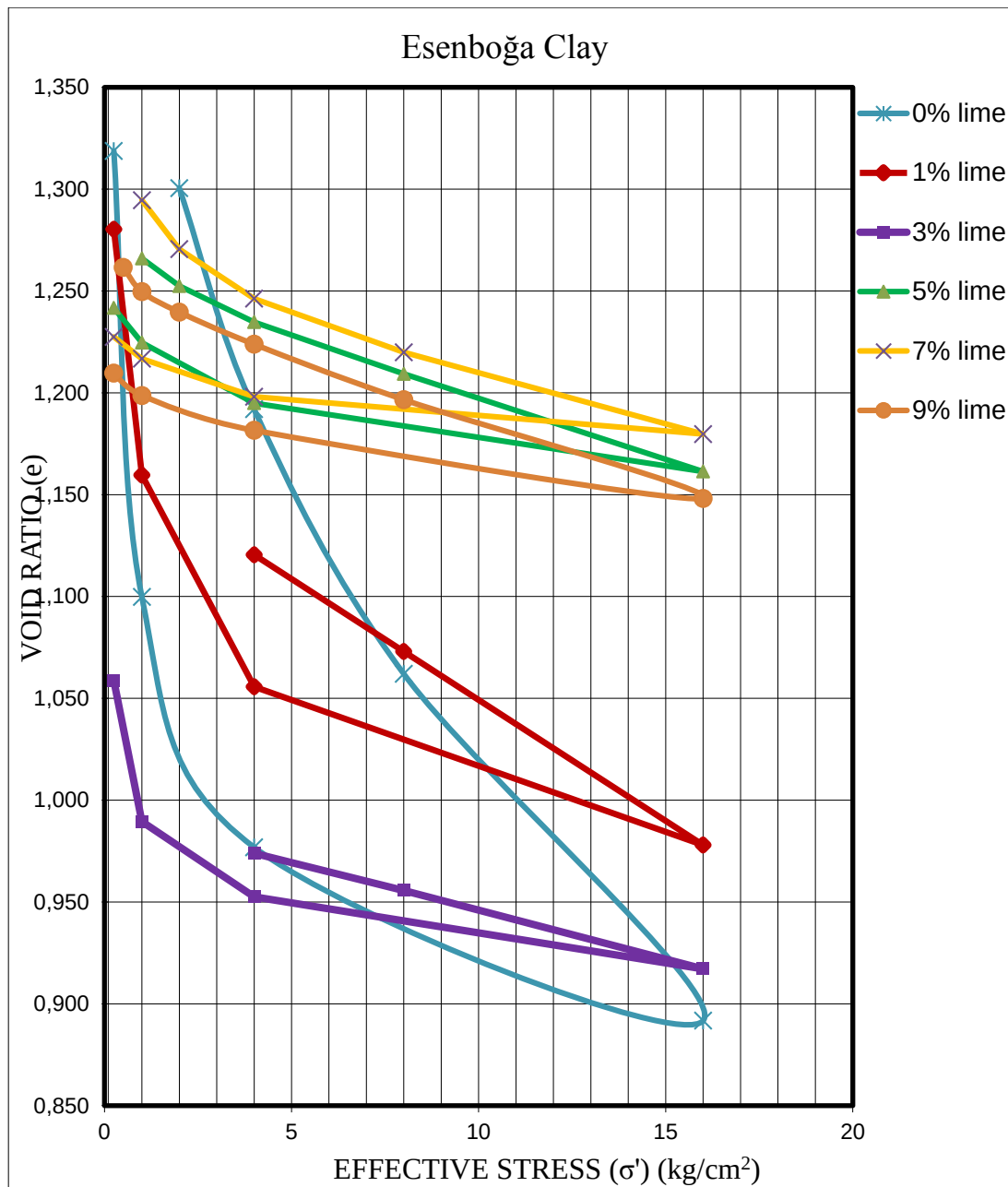


Figure D.9 : e - σ' relationship for natural and lime-treated soil samples for 90 days cure period for Esenboğa Clay.

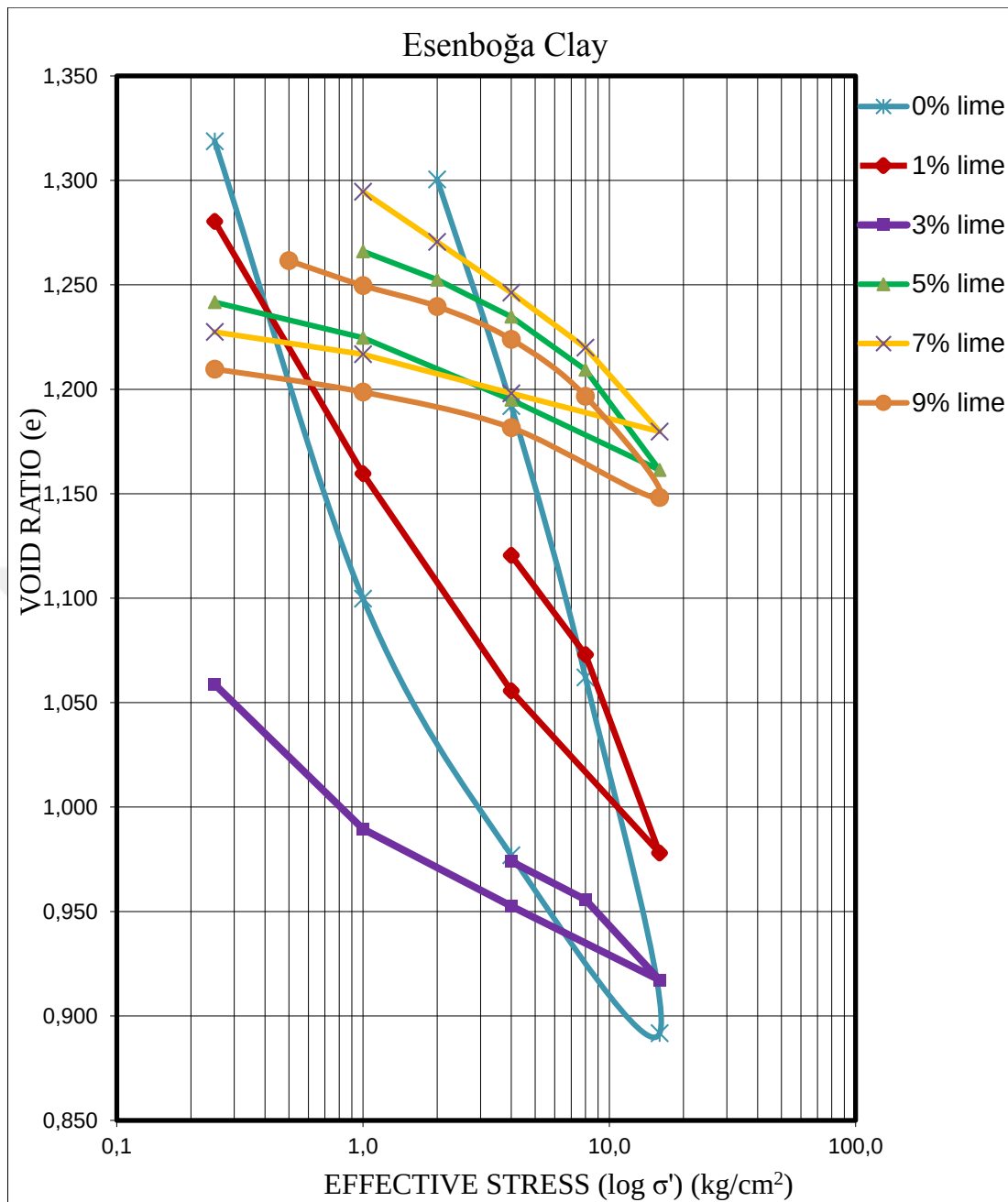


Figure D.10 : e-log σ' relationship for natural and lime-treated soil samples for 90 days cure period for Esenboğa Clay.



CURRICULUM VITAE

Name Surname : İnci Ünver
Place and Date of Birth : Ankara 04.06.1979
E-Mail : incisut79@hotmail.com

EDUCATION :

- **B.Sc.** : 2002, Selçuk University, Faculty of Engineering-Architecture, Department of Civil Engineering
- **M.Sc.** : 2006, Gazi University, Faculty, of Engineering-Architecture, Department of Civil Engineering

PROFESSIONAL EXPERIENCE AND REWARDS:

- 2003-2005 AŞTİ – Çayyolu 1st Stage Metro Construction, Prokon-Optim-Obermayer Joint Venture (Consultant) Task: Site Quality Control/Assurance Engineer (Anchored bored pile walls)
- 2005- 2006 Prokon Engineering and Consult. Co. Inc. Task : Civil Engineer (Design activities)
- 2006-2008 MNG Zemtaş Soils and Foundation Eng. Construction Co. Inc. Task : Civil Engineer (Chief of Technical Office)
- 2008-2009 GEOTEM Geotechnical Consult. Eng. Construction Ltd. Task: Ankara Gimat Intersection Underpass construction – Site coordination and geotechnical engineering design activities
- 2009-2010 ATAC Eng. Consult. Ltd. Task: Project Engineer
- 2013- Atılım University - Department of Civil Engineering, Research Assistant

PUBLICATIONS, PRESENTATIONS AND PATENTS ON THE THESIS:

- **Ünver İnci Süt**, Lav M.Ayşen, Çokça E. (2016), "Improvement of an Expansive Soil with Hydrated Lime", 12th International Congress on Advances in Civil Engineering, ACE2016, Boğaziçi University, İstanbul, Turkey.
- **Ünver İnci Süt**, Lav M.Ayşen, Çokça E. (2018), "Effect of Lime on the Compressibility Characteristics of a Highly Plastic Clay", International Journal of Applied Science and Technology, Vol.8, No:2, doi: 10.30845/ijast.v8n2p3, pp.19-23.
- **Ünver İnci Süt**, Lav M.Ayşen, Çokça E. (2018), "Effect of Fly Ash on Swell Behaviour of a Very Highly Plastic Clay", China-Europe Conference on Geotechnical Engineering, University of Natural Resources and Life Sciences, Vienna, Austria.
- **Ünver İnci Süt**, Lav M.Ayşen, Çokça E. (2018), "Methylene Blue Adsorption and Swelling Characteristics of a Lime-treated Expansive Clay", 13th International Congress on Advances in Civil Engineering, ACE2018, Ege University, İzmir, Turkey.

OTHER PUBLICATIONS, PRESENTATIONS AND PATENTS:

- Ünver A.Mengüç, **Ünver İnci Süt** (2012), "The Back Analysis of a 25m Deep Excavation Supported by Anchored Bored Pile Walls", 10th International Congress on Advances in Civil Engineering, ACE2012, Middle East Technical University, Ankara, Turkey.
- **Ünver İnci Süt**, Soygür Ünsal (2012), "Effects of Stabilization of Cohesive Soils by Using Lime on CBR Value", 11th International Congress on Advances in Civil Engineering, ACE2014, İstanbul Technical University, İstanbul, Turkey.
- **Ünver İnci Süt**, Soygür Ünsal (2014), "Kireç ile İyileştirilen Şişebilen Zeminlerin Statik ve Dinamik Özellikleri", Zemin Mekaniği ve Temel Mühendisliği Onbeşinci Ulusal Kongresi, ZM15, Orta Doğu Teknik Üniversitesi, Ankara.
- Ünver A.Mengüç, **Ünver İnci Süt** (2016), "A Preliminary Design of Ground Improvement by Grouted Stone Columns for a Shopping Centre", 5th International Conference on Geotechnical and Geophysical Site Characterisation, ISC5, Gold Coast, Queensland, Australia.