APRIL 2019 M.Sc. in Civil Engineering ISLAM TABUR

HASAN KALYONCU UNIVERSITY

GRADUATE SCHOOL OF

NATURAL & APPLIED SCIENCES

AN EXPERIMENTAL STUDY ON UNIT SOCKET RESISTANCE OF GAZIANTEP LIMESTONE FOR DRY AND FULLY SATURATED CONDITIONS

M.Sc. THESIS

IN

CIVIL ENGINEERING

BY

ISLAM TABUR

APRIL 2019

An Experimental Study On Unıt Socket Resistance Of Gaziantep Limestone For Dry And Fully Saturated Conditions

M.Sc. Thesis

in

Civil Engineering

Hasan Kalyoncu University

Supervisor

Assist. Prof. Dr. Volkan KALPAKCI

Islam TABUR

April 2019

	GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCESINSTITUTE M.Sc. ACCEPTANCE AND APPROVAL FORM

Civil Engineering Department, M.Sc. (Master of Science) programme student **İslam TABUR** prepared and submitted the thesis titled **" An Experimental Study On Unit Socket Resistance Of Gaziantep Limestone For Dry And Fully Saturated Conditions"** defended successfully on the date of 18/04/2019 and accepted by the jury as a M.Sc. thesis.

Position	**Title, Name and Surname** **Department/University**	**Signature**
Supervisor	Assist. Prof. Dr. Volkan KALPAKCI Civil Engineering Department/ Hasan Kalyoncu University	
Jury Member	Prof. Dr. Hanifi ÇANAKÇI Civil Engineering Department/ Gaziantep University	
Jury Member	Assist. Prof. Dr. Bülent HAZNEDAR Computer Engineering Department/ Hasan Kalyoncu University	

This thesis is accepted by the jury members selected by the institute management board and approved by the institute management board.

Prof. Dr. Mehmet KARPUZCU
Director

I here by declare that all in formation in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Islam TABUR

ABSTRACT

AN EXPERIMENTAL STUDY ON UNIT SOCKET RESISTANCE OF GAZIANTEP LIMESTONE FOR DRY AND FULLY SATURATED CONDITIONS

TABUR, Islam
M.Sc. in Civil Engineering
Supervisor:Assist. Prof. Dr. Volkan KALPAKCI
April 2019, 43 pages

The primary objective of this study was to investigate the correlation between the uniaxial compressive strength and unit socket resistance of Gaziantep limestone experimentally. Since the uniaxial compressive strength of Gaziantep limestone was known to decrease significantly upon saturation, the experiments were conducted in the laboratory for dry and fully saturated conditions on sample limestone blocks taken from a quarry site in Gaziantep. The results revealed that, both the unit socket resistance and uniaxial compressive strength of Gaziantep limestone was decreasing significantly due to saturation. The results were also compared with the methods given in the literature which correlate the unit socket resistance of rocks with their uniaxial compressive strength. These evaluations revealed that the linear correlations significantly overestimated the measured socket resistance values both for dry and fully saturated samples. On the other hand, the non-linear methods generally overestimated the unit socket resistance under fully saturated conditions but provided a reasonable estimation for dry samples. A linear correlation and upper bound and lower bound curves as a non-linear correlation range for estimating the unit socket resistance of Gaziantep limestone from its uniaxial compressive strength were also suggested within this study. It was concluded that care should be taken while using these suggested methods for estimation of the unit socket resistance of Gaziantep limestone especially for places prone to saturation. For such cases, using some of the lower bound solutions discussed in this study may be a safer option for desing purposes.

Key Words: Rock, Compressive Strength, Socket Resistance, Limestone

ÖZET

GAZİANTEP KİREÇTAŞININ KURU VE TAMAMEN DOYGUN DURUMLARDA BİRİM ÇEVRE SÜRTÜNMESİ ÜZERİNE BİR DENEYSEL ÇALIŞMA

TABUR, Islam
Yüksek Lisans, Inşaat Mühendisliği Bölümü
Tez Yöneticisi:Dr. Öğr. ÜyesiVolkanKALPAKCI
Nisan 2019, 43 sayfa

Bu çalışmanın ana amacı, Gaziantep kireçtaşının birim çevre sürtünmesi ile tek eksenli basınç dayanımı arasındaki ilişkinin deneysel olarak incelenmesidir. Gaziantep kireçtaşının tek eksenli basınç dayanımının ıslanma durumunda ciddi oranda düştüğü önceki çalışmalardan bilindiğinden, deneyler Gaziantep'te bir sahadan alınan numuneler üzerinde tamamen ıslak ve tamamen kuru durumlar için gerçekleştirilmiştir. Yapılan deneyler sonucunda, Gaziantep kireçtaşında ıslanma durumunda hem birim çevre sürtünmesinin hem de tek eksenli basınç dayanımının ciddi şekilde düştüğü gözlemlenmiştir. Deney sonuçları literatürde bu iki parametreyi birbiriyle ilişkilendiren çeşitli metotlarla da karşılaştırılmıştır. Bu karşılaştırmalar neticesinde incelenen doğrusal ilişkilerin hem kuru hem de ıslak durum için birim çevre sürtünmesini ciddi şekilde yüksek tahmin ettiği görülmüştür. Doğrusal olmayan metotlar ise kuru durumlar için makul tahminler sunarken genellikle ıslak durumlarda ölçülen değerlerden daha yüksek değerler tahmin etmişlerdir. Bu çalışma kapsamında Gaziantep kireçtaşının birim çevre sürtünmesi tahmininde kullanılmak üzere bir doğrusal korelasyon bir de doğrusal olmayan tahmin aralığı önerilmiştir. Yapılan bu çalışma neticesinde, literatürde önerilen metotların Gaziantep kireçtaşı için özellikle ıslanmaya maruz kalabilecek bölgelerde kullanılmasının gerçek durumdan çok daha yüksek çevre sürtünmeleri tahmin edilmesine yol açabileceği görülmüştür. Bu sebeple bu çalışma içerisinde bahsedilen bazı metotların alt limitlerinin kullanılması tasarım için daha güvenli olacaktır.

Anahtar Kelimeler: Kaya, Basınç Dayanımı, Çevre Sürtünmesi, Kireçtaşı.

To my lovely family…

ACKNOWLEDGEMENTS

I sincerely acknowledge Asst. Prof. Dr. Volkan Kalpakcı for his continuous support and contributions during this study. I also thank to Prof. Dr. Hanifi Canakci for his valuable contributions especially during the experimental studies. Finally, I specially thank to my mother and my family for their support and patience during my studies.

TABLE OF CONTENTS

Page

LIST OF TABLES

Page

LIST OF FIGURES

Page

LIST OF SYMBOLS/ ABBREVIATION

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
ISRM	International Society for Rock Mechanics
MTA	Turkey General Directorate of Mineral Research and Exploration
MEP	Multi Expression Programming
GEP	Gene Expression Programming
LGP	Linear Genetic Programming
UCS	Uniaxial Compressive Strength
CFEM	Canadian Faundation Engineering Manual
LRFD	Load and Resistance Factor Design
LVDT	Linear Variable Differential Transformer
γ_{dry}	Dry Unit Weight
γ_{sat}	Saturated Unit Weight
W_g	Water Absorption by Weight
q_{dry}	Compressive Strength, Dry
q_{sat}	Compressive Strength, Saturated
σ_{dry}	Tensile Strength, Dry
σ_{sat}	Tensile Strength, Saturated
USP ν_{dry}	USP Velocity, Dry
E_{dry}	Modulus of Elasticity, Dry
q_s	Unit Socket Resistance

P_a	Reference Pressure
q_u	Uniaxial Compressive Strength of Rock
b	Empirical Coefficient
p_d	Dry Density
m_d	Dry Weight
B_v	Bulk Volume
P_s	Saturated Density
m_s	Saturated Weight
p_n	Natural Density
m_n	Natural Weight
n	Porosity
p_v	Pore Volume
W	Water Content
W_w	Pore Water Weight
f_{ck}	Characteristic Compressive Strength
R	Correlation Coefficient

CHAPTER 1

INTRODUCTION

1.1 General

In ancient times, people used natural stones to meet their housing needs. Hittites, ancient Egyptians, ancient Greeks, Romans, Ottomans have made use of natural stones in many civilizations. In ancient civilizations, a wide variety of structures such as palaces, temples and statues were built with natural stones. Nowadays, although the usage area has narrowed compared to the past, natural stones are mainly used as filling material and aggregate in the construction sector.

Limestones between natural stones has created an industry branch that is open to development due to its use in many areas. It was used as a building material in the previous periods before being used in concrete structures. In parallel with the development of the limestone industry which was used as mortar binder in construction, the main usage area is 40-70% construction sector. Limestone is used as aggregate in concrete mortar in this sector, as filling material in road construction, as stone in railway ballast. The second major use of limestone is cement construction. A major part of the main raw material of the cement is limestone. Besides, it is used as agricultural fertilizer and in various industrial applications.

Located in South Eastern Anatolia Region of Turkey Gaziantep is one of the important central points of transition from Anatolia to Syria and from there to Mesopotamia.(Figure 1.1). Gaziantep limestone has a clayey and calcareous structure and has a more porous structure than other limestones. In appearance, the clayey limestone is whitish, gray cream, dirty yellow color and medium thin layer. Calcareous limestone is gray, yellowish gray color, medium thick layer. There may also be fossils in some parts of the Gaziantep limestone. (Marangoz,2005).

Limestone with chemical formula $CaCO_3$ is found in many parts of Gaziantep. In addition to the existence of houses made of adobe in rural areas of Gaziantep, limestone has been used as building material in the center of Gaziantep for many years.

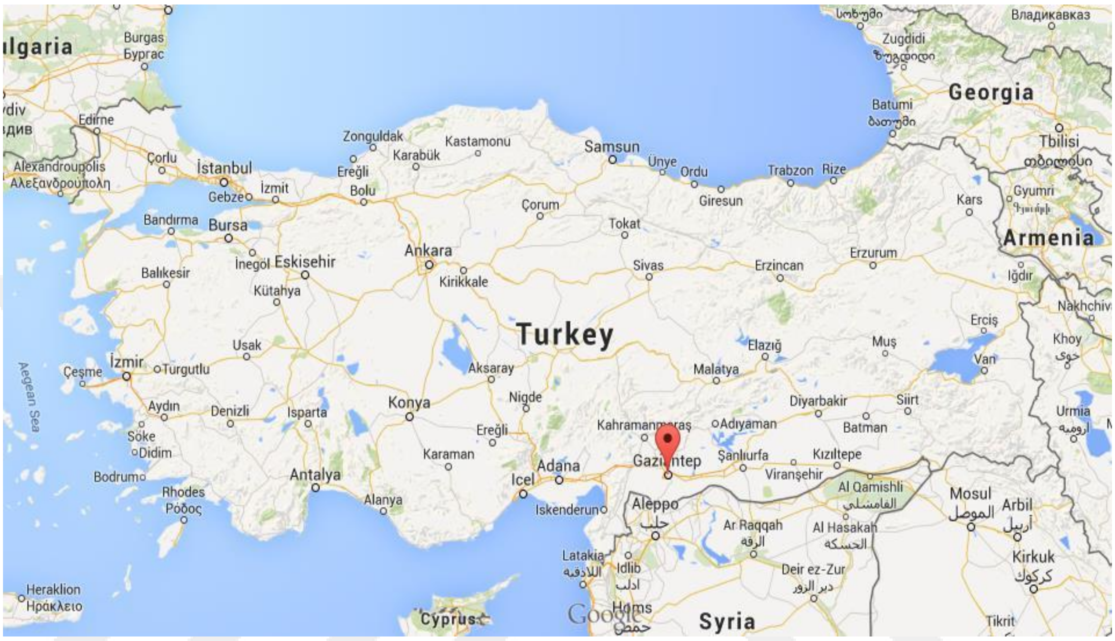

Figure 1.1 Location of Gaziantep (World Easy Guides)

With the rapid development of the city in recent years, deep excavations in limestone units with rock-bolt or anchorage systems or high-rise structures have to be supported by piles socketed into limestone. One of the main parameters used in the design of these structural elements is the unit side friction of the rock. Because this value is an important parameter to determine the axial capacity of the structural element when the unit side firction is multiplied by the socket region surface area of the structural element. In majority of the methods given in the literature, the uniaxial compressive strength of the rock is used in calculating the unit side friction of the rocks. It shol also be stated that, in previous studies, it has been determined that the uniaxial compressive strength of Gaziantep limestone in wet conditions decreased by 40% - 60% compared to the dry state. (Canakcı, 2007)

In this study, samples taken from Gaziantep limestone were cutten as blocks and cylindrical core samples were taken from the midpoints of these blocks. Two blocks from each sample were cut and one sample was tested in dry condition while the orher was tested after full saturation. Firstly the core samples taken from the blocks obtained from the same sample was tested under uniaxial pressure test while one was wet and the other was in dry condition and the wet and dry situation uniaxial compressive strength of these samples were determined. Then, concrete grout was placed in the spaces inside the samples, after the concrete hardened the unit side friction of the samples were determined by a testing equipment deisgned for this thesis. After this step, the results are compared with the methods proposed in the literature and the results are presented in the thesis.

1.2 Thesis Layout

In the thesis, the detailed literature review on the subject after the introduction section presented in this chapter is given in Chapter 2. The details of the material and method related to the experimental study is given in Chapter 3 and the results of the tests are presented in Chapter 4 together with discussions. In the light of the findings,conclusions are given in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Limestone is an extrusive sedimentary rock. In this section, firstly general information about sedimentary rocks and especially limestone is given. Then, the results obtained from the studies about the Gaziantep limestone used in the experimental studies are summarized. Finally, used in the comparisons made in this study the unit environmental friction calculation methods are given in detail.

2.2 Sedimentary Rocks

Sediments from a relatively thin surface layer of the Earth's crust, covering the underlying igneous and metamorphic rocks (Marangoz, 2005).

This sedimentary cover is discontinuous and of varying thickness; it avarages about 0.8 km in thickness but locally reaches over 12 km in long narrow belts, the sites of former geosynclines. It has been estimated that sediments constitute only about 5 per cent of the crustal rocks (to a depth of 16 km), in which the proportions of the three main types are approximately: shales and clays, 4 per cent; sandstones, 0.75 per cent; limestones, 0.25 per cent. Sedimentary rocks also include varieties which are composed of the remains of organisms, such as certain limestones and coals, and others which are formed by chemical deposition (De Freitas, 1979).

Accumulations of loose sand, for example, derived from the breakdown of older rocks in ways described earlier, and brought together and sorted by water and wind, have become hardened rocks such as sandstone and quartzite. Pore spaces in the original sands have been partly or completely filled with mineral matter brought by percolating water and deposited as coatings on the sand grains, thus acting as a cement to bind them together. These processes are known as cementation. In muddy sediments, the very small particles of silt and clay of which they are mainly composed have been pressed together by the weight of sediment; interstitial water has been squeezed out and in course of time the mud has become a coherent mass of clay, shale, ormudstone.(De Freitas, 1979).

Compaction of this kind of affects the muddy sediments to a greater degree than the sands, and during the compaction process much of the pore-contained water in an original mud is pressed out. Some of the water, with its dissolved salts, may remain in the sediment after its compaction, and is known as connate water. The general term diagenesis is used to denote the compaction of a sediment into a sedimentary rock, and includes the processes outlined above and also chemical processes such as re-crystallization and replacement (Marangoz, 2005).

When rock come again into the zone of weathering, after a long history, soluble substances are removed and insoluble matter is released, to begin a new cycle of sedimentation in rivers and the sea. The broad groupings used in the Table of Sedimentary Rocks are:

1) Detrital sediments (mechanically sorted), e.g. gravels, sandstones, clays and shales.

2) Chemical and biochemical (organic), e.g. limestones, coals, centimeter(De Freitas, 1979).

2.3 Limestone

Limestones consist essentially of calcium carbonate, with which there is generally some magnesium carbonate, and siliceous matter such as quartz grains. The average of over 300 chemical analyses of limestones showed 92 per cent of $CaCO_3$ and $MgCO_3$ together, and 5 per cent of SiO_2; the proportion of magnesium carbonate is small except in dolomite and dolomitic limestones. Limestones are bedded rocks often containing many fossils; they are readily scratched with a knife, and effervesce on the addition of cold dilute hydrochloric acid. The distance between bedding-planes in limestones is commonly 30 to 60 cm, but varies from a couple of centimeters or less in thin-bedded rocks to over 6 cm in some limestones (Marangoz, 2005).

Calcium carbonate is present in the from of crystals of calcite or aragonite, as amorphous calcium carbonate, and also as the hard parts of organisms (fossils) such as shells and calcareous skeletons, or their broken fragments. Thus, a consolidated shell-sand is a limestone by virtue of the calcium carbonate of which the shells are made. On the other hand, chemically deposited calcium carbonate builds limestones under conditions where water of high alkalinity has a restricted circulation, as in a shallow sea or lake. Non-calcereous constituents commonly present in limestones include clay, silica in colloidal from or as quartz grains or as parts of siliceous organisms, and other hard detrital grains. Though usually grey or white in colour, the rock may be tinted, e.g. by iron compounds or finely divided carbon, or by bitumen. The types listed in the table are now described (De Freitas, 1979).

Chalk is a soft white limestone largely made of finely divided calcium carbonate, much of which has been shown to consist of minute plates, 1 or 2 microns in diameter. These plates are derived from the external skeletons of calcareous algae, and are known as coccoliths. The Chalk also contains many foraminifera, which differ in king and abundance in different part of the formation; and other fossils, such as the shells of brachiopods and sea-urchins. The foraminifera are minute, very primitive jelly-like organisms (protozoa) with a hard globular covering of carbonate of lime; they float at the surface of the sea during life, and then sink and accumulate on the sea floor. Radiolaria are similar organisms which have siliceous frameworks, often of a complicated and beautiful pattern; these too are found in Chalk but are not so numerous as the foraminifera. Parts of the rock contain about 98 per cent $CaCO_3$ and it is thus almost a pure carbonate rock. It was probably formed at moderate depths (round about 180 m) in clear water on the continental shelf (Marangoz, 2005).

2.4 Geology of Gaziantep

General geology of the study area Turkey General Directorate of Mineral Research and Exploration (MTA) is made by is the study of the geological features of the province Gaziantep was utilized. The observed units in the study area are located at respectively Eocene Hoya formation and Plio-quaternary unit. Figure 2.1 in shows the geological map made by MTA. The unit consisting of carbonates was named by Sungurlu (1974).

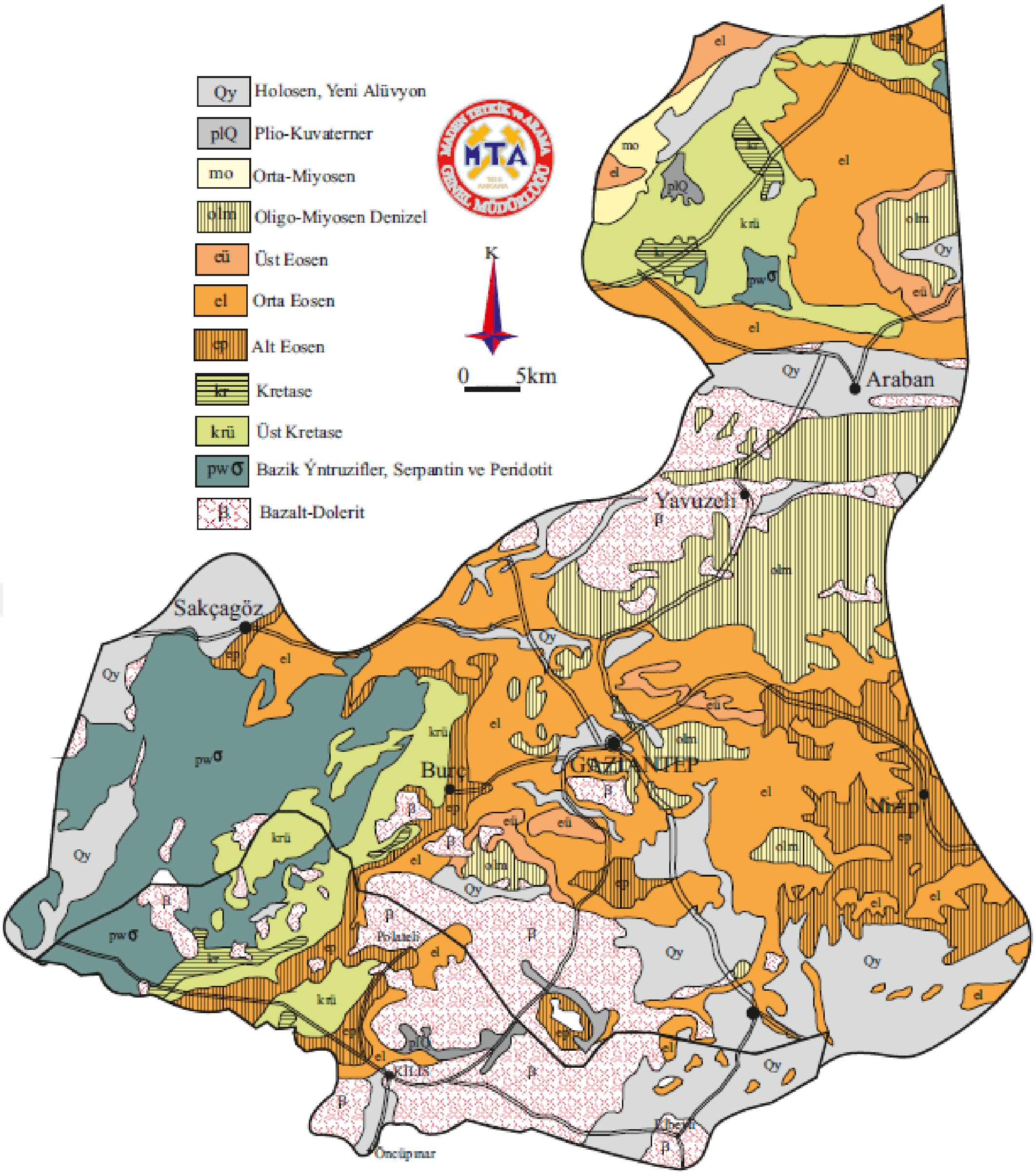

Figure 2.1 Geological map of Gaziantep (Kılıc, 2015)

The dominant rock type of the formation is limestone. It starts with gravelly limestone at the bottom. Gray, beige, some places red colored, thick-very thick bedded limestones, passes to the top towards the limestone. These limestones are creamy, dirty white, light gray colored, medium thick bedded, some places without any bedded, some of them have fossiliferous and plenty of cracks. In the upper surfaces of the unit are observed cherty tubers. The limestone carbonate flatness micro-facies environment and open platform micro-facies environment with is deposited (Kılıc, 2015). The age of the unit was determined as Middle (Upper Lutetian) - Upper (Priabonian) Eocene (Terlemez et al., 1997).

2.5 Some Basic Features of Gaziantep Limestone

In the study presented in the details at the Canakcı (2007); in order to investigate the possible causes of the collapse of the limestone caves in the Karakabir and Hamdi Kutlar regions of Gaziantep, some of the basic properties of samples of Gaziantep Limestone have been determined. The rock core samples were tested for uniaxial compressive strength (both in oven dry and saturated conditions), tensile strength, density (dry and saturated surface dry) water absorption and ultrasonic pulse velocity. In order to investigate the effect of saturation on the compressive and the tensile strength of the core samples, they were left in the water for 30 days. Sample sizes were adjusted in accordance with ISRM for each test. All tests were performed in accordance with the procedures given in ISRM (Brown 1981). Dry compressive strengths of the limestone were 25.51 and 10.20 MPa for Karakabir and Hamdi Kutlar, while saturated compressive strengths were 11.53 and 5.36 MPa, respectively. The tensile strengths of the limestone in dry condition for the former and the latter caves were 3.12 and 2.41 MPa, which reduced to 0.65 and 0.31 MPa upon saturation, respectively. Water absorption values of the limestone were 24 and 11% for Karakabir and Hamdi Kutlar, respectively. The test results are given in Table 2.1.

Canakcı et al. (2007) in another study by multi expression programming (MEP), gene expression programming (GEP) and linear genetic programming (LGP) known as a series of genetic programming techniques using the pressure and tensile strength of the Gaziantep limestone was tried to be estimated.

Table 2.1. Properties of limestone obtained from collapsed caves

Property	Karakabir Region	Hamdi Kutlar Region
γ_{dry}(kN/m^3)	16.76	18.64
γ_{sat}(kN/m^3)	20.79	20.60
W_g(%)	24	11
q_{dry}(MPa)	25.51	10.2
q_{sat}(MPa)	11.53	5.36
σ_{dry} (MPa)	3.12	2.41
σ_{sat} (MPa)	0.65	0.31
USP v_{dry} (m/s)	2906	2656
E_{dry}(GPa)	11.3	4.45

Canakcı et al.(2007) by in this study carried out performing uniaxial pressure tests on a total of 116 cores taken from Gaziantep limestone the results were analyzed by a series of genetic programming methods described above. For uniaxial pressure tests, samples of 60mm diameter and 150mm height were prepared in accordance with ISRM (1981) standards. After ovendry the samples, experiments were carried out with a loading rate of 0.5 MPa /s.

According to the results of this study, the mean value for uniaxial compressive strength (UCS) of Gaziantep limestone is 10.7 MPa and the standard deviation is 9.6 MPa. The results obtained is changed range between 3.7 MPa and 67.4 MPa. When the frequency histogram was examined, it was found that the majority of the samples tested had UCS $\leq$ 15 MPa.Based on the results of this study, the strength of limestone samples used in this study can generally be classified as very weak rock to weak rock according to ISRM (1981).Similarly, the uniaxial compressive strength of the samples indicates that the samples of Gaziantep limestone, which are tested on the basis of the range of values recommended by Ramamurthy and Arora (1993), are mostly in the low-strength rocks.

Table 2.2 Strength classification of intact and jointed rocks (Ramamurthy and Arora, 1993)

Class	Description	UCS (MPa)
A	Very high strength	> 250
B	High strength	100-250
C	Moderate strength	50-100
D	Medium strength	25-50
E	Low strength	5-25
F	Very low strength	<5

2.6 Methods for Determining Side Friction from UCS

The side friction that will be mobilized along the interface between the structural members and rockshas been studied by various resaerchers for long years. This is because the ultimate bearing capacity of structural members embedded in rocks (rock-socketed piles, rock bolts and etc…) will depend on the side friction value.

The mechanism that takes place during mobilization of side friction along the embedded surface is complex and dependent on several factors like the friction and cohesion along the interface,changes in the normal stress distribution on the shaft surface and etc… Hence, utilization of empirical data is essential for estimating the side friction alongthese surfaces as it was discussed in Serrano and Olalla (2004).

In the methodsfor estimating the side friction along the shaft-rock interfaces, generally an empirical relation based on the UCS of the rock is recommended. Some of the methods given in the literature directly relate the UCS of the rock with the side friction while some others also consider the effect of rock mass structure. Since intact samples were used during the experiments of this study, the methods which do not consider the effect of rock mass structure were of concern and these methods are summarized below.

A well-known method is suggested in (CFEM, 2006) as:

$$\frac{q_s}{Pa} = b(\frac{q_u}{Pa})^{0.5} \qquad (2.1)$$

Where; P_a is reference pressure (100 kPa), q_u is the UCS of rock and b is an empirical coefficient.

The "b" coefficient was proposed as 0.63 by Carter and Kulhawy (1988) as a lower bound value while it was suggested as 0.63 - 0.94 for service limit design by Horvath et al (1983). Also, "b" value was given as 1.41 for allowable stress method in the study of Rowe and Armitage (1984).

Other methods given in the literature may be generalized in two categories as linear and power relationships.

In the linear relationships, the side friction is assumed to change linearly by the UCS of the rock. Some of these methods are summarized below in the historical order:

$q_s = 0.3q_u$ (Reynolds and Kaderabek, 1981) (2.2)

$q_s = 0.2q_u$ (Gupton and Logan, 1984) (2.3)

$q_s = 0.15q_u$ (Reese and O'neill, 1988) (2.4)

$q_s = 0.25q_u$ (Toh et al., 1989) (2.5)

As it can be seen above, the proposed methods estimate a skin friction changing between 0.15 to 0.30 of the UCS of the rock.

On the other hand, some of the power relationships may be listed as follows:

$q_s = 0.34q_u^{0.51}$ (Rosenberg and Journeaux, 1976) (2.6)

$q_s = 0.21q_u^{0.5}$ (Horvath and Kenny, 1979) (2.7)

(as suggested in AASHTO LRFD, 2007)

$q_s = 0.22q_u^{0.6}$ (Meigh and Wolski, 1979) (2.6)

$q_s = 0.21q_u^{0.5}$ (Williams et al., 1980) (2.7)

$q_s = 0.41q_u^{0.57}$ (Rowe and Armitage, 1984) (2.10)

In the study of Rowe and Armitage (1987), the avaliable shaft resistance data found in the database was collected. After analyzing all of the avaliable data, the proposed equation was dependent on surface roughnessas follows:

$q_s =(0.45\text{-}0.6)\ q_u^{0.5}$ (Rowe and Armitage, 1987) (2.11)

The lowerbound value was taken into consideration for our study since the socket interface was clean and smooth in all experiments.

As it can be seen from the listed power relationships, in estimating the side friction from UCS of the rock, the coefficients vary between 0.21 – 0.44 while the power is in range of 0.34 – 0.6. It should be noted that some of the methods given in the literature for different unit systems are converted for "q_u" in units of [MPa].

In McVay et al. (1992), the site data for Florida limestone was compared with the avaliable methods in the literature to estimate side friction and it was seen that among linear relationships the best fit to the analyzed data was obtained from Gupton and Logan (1984).

Later; Kulhawy and Phoon (1993) combined the database of Rowe and Armitage (1987) which included approximately 80 load tests from 30 different sites with the databases of Bloomquist et al. (1991) and McVay et al. (1992) in which the results of 47 load tests from 13 different sites of Florida limestone were presented. Based on this study, Kulhawy and Phoon (1993) had suggested the following equation:

$$\frac{q_s}{Pa} = c\left(\frac{q_u}{2Pa}\right)^{0.5} \quad (2.12)$$

where; c = 1 for lowerbound solution, c = 2 represents the mean and c = 3 for upper bound solution (for artifically roughened surfaces).

In the study of Gunnink and Kiehne (2002), the side friction in Burlington limestone was investigated through field tests and the results were compared with the methods suggested by Williams et al. (1980) and Rowe and Armitage (1984). The predicted values were significantly lower than the observed side resistances for two of the experiments while the inverse was valid for the other load test.

In another study by Rezazadeh and Eslami (2017), most of the avaliable methods were analyzed by a large data base (combining almost all of the avaliable data in the literature). The results have revealed that the linear relationships yielded to overestimated side friction values while the power relationships had performed better for limestones. The following equations were proposed for limestones (Eq. 2.13) and for general (Eq. 2.14) respectively.

$q_s = 0.4014 q_u^{0.3411}$ (For limestone, Rezazadeh and Eslami, 2017) (2.13)

$q_s = 0.36 q_u^{0.36}$ (For general use, Rezazadeh and Eslami, 2017) (2.14)

It should here be stated that, some of the above listed methods may not be fully compatible with the tested rock type in this study. However, since most of the methods are utilized for a generalized side friction estimation in the literature, it was decided to compare the results of this study with all of the listed methods.

The bond strength between rock bolts and the embedded rock surface is another area where side friction value is used. The side friction value was given as 300 – 400 kPa for limestones in the study of Elias and Juran (1991) as it can be seen in Table 2.3.

Table 2.3 Estimated bond strength of soil nails in soil and rock

Material	**Construction Method**	**Soil/Rock Type**	**Ultimate Bond Strength, q_s (kPa)**
Rock	Rotary Drilled	Marl limestone	300 - 400
		Phyllite	100 - 300
		Chalk	500 - 600
		Soft dolomite	400 - 600
		Fissured dolomite	600 - 1000
		Weathered sandstone	200 - 300
		Weathered shale	100 - 150
		Weathered schist	100 - 175
		Basalt	500 - 600
		Slate Hard shale	300 - 400

CHAPTER 3

RESEARCH PROGRAM AND TEST PROCEDURES

Samples were taken from a quarry site in Karatas region of Gaziantep as shown in Figure 3.1. The blocks were then shaved and 24 blocks having dimensions of 30x30x15cm (WidthxLengthxHeight) were obtained (Figure 3.2).

Figure 3.1 Quarry site in Karatas region

Figure 3.2 Limestone block

Figure 3.3 Two adjacent limestone blocks (15×30×15 cm)

The experiments of this study were conducted in the Civil Engineering laboratory of Hasan Kalyoncu University. As the first step of the experimental study, 24 limestone blocks having dimensions of 300x300x150mm (WidthxLengthxHeight) were taken from different places of a quarry site in Gaziantep as discussed above. After assigning a number to each block; the blocks were cut into two adjacent parts having equal dimensions of 150x300x150mm (WidthxLengthxHeight) making a total of 48 limestone blocks (Figure 3.3). In this way, it was aimed to have a dry and a fully saturated test result for each block. Then, by a carrot sampler (Figure 3.4) a NX size cylindrical sample (D = 54.7mm) was extracted from the middle of each of the 48 blocks (Figure 3.5 and 3.6). All the tests conducted on limestone samples were done in accordance with the suggested methods in ISRM (2007). The cylindrical samples were first weighed and then oven-dried at least for 24 hours (until constant weight) at 105±3^{0}C (Figure 3.7). After the drying procedure, uniaxial compression tests were done for the dry samples of each block by UTEST UTR-0550 type testing machine which has a loading capacity of 42000 kN with a sensitivity of 0.01 MPa (Figure 3.8). The height/diameter ratio of the samples wereH/D = 150/54.7 = 2.74 and the loading rate was selected as 0.7 MPa/s from the range given in ISRM (2007) as 0.5 – 1.0 MPa/s. As a result, the uniaxial compressive strength of each dry sample from each block was determined.

Figure 3.4 Carrot sampler

Figure 3.5 A limestone block after sample taken from the middle

Figure 3.6 A view of some of the limestone blocks

Figure 3.7 Drying of the cylindrical samples in the laboratory oven

Figure 3.8 Uni-axial compression testing machine

The rest of the samples were cured under water (Figure 3.9) and weighed each day until constant weight (to nearest 0.01g) by UTest UTW-0644 type weigher with 6000g capacity (Figure 3.10), to ensure the fully saturation of the samples. After full saturation, the uniaxial compressive strengths of these samples were determined experimentally applying the same testing procedure described for dry samples. As a result, the uniaxial compressive strength of each sample from each block was determined for fully saturated condition. Additionally, some basic properties of the tested samples like porosity, water absorption capacity, dryand fully saturated unit weights were also determined during these tests in accordance with ISRM (2007).

Upon completion of the uniaxial compression tests, the holes at the middle of the blocks which were drilled during the extraction of the cylindrical samples were filled with a concrete having 28 days characteristic compressive strength of 30MPa. CEM I 42.5 cement class was used for concrete production and the characteristic compressive strength was determined according to ASTM C39 (ASTM, 2018) standard (Figure 3.11). All the rock blocks were cured under water for 28 days to obtain the target compressive strength of the infilled concrete. The upper and lower ends of the cylindrical concrete core was levelled with a suitable cutter before testing in order to obtain a plane loading surface. After curing procedure, the unit side resistance tests were executed for samples under fully saturated conditions while the remaining blocks were tested after completely dried in the oven (Figure 3.12). The infilled concrete was intentionally selected to have a higher compressive strength than the highest uniaxial compressive strength value obtained for the tested limestone samples in order to ensure the failure of the surface between concrete and rock to be controlled by the side resistance of rock but not by the side cohesion of the concrete as it was also discussed in Carter and Kulhawy (1992), Kulhawy et al. (2005) and Salgado (2008).

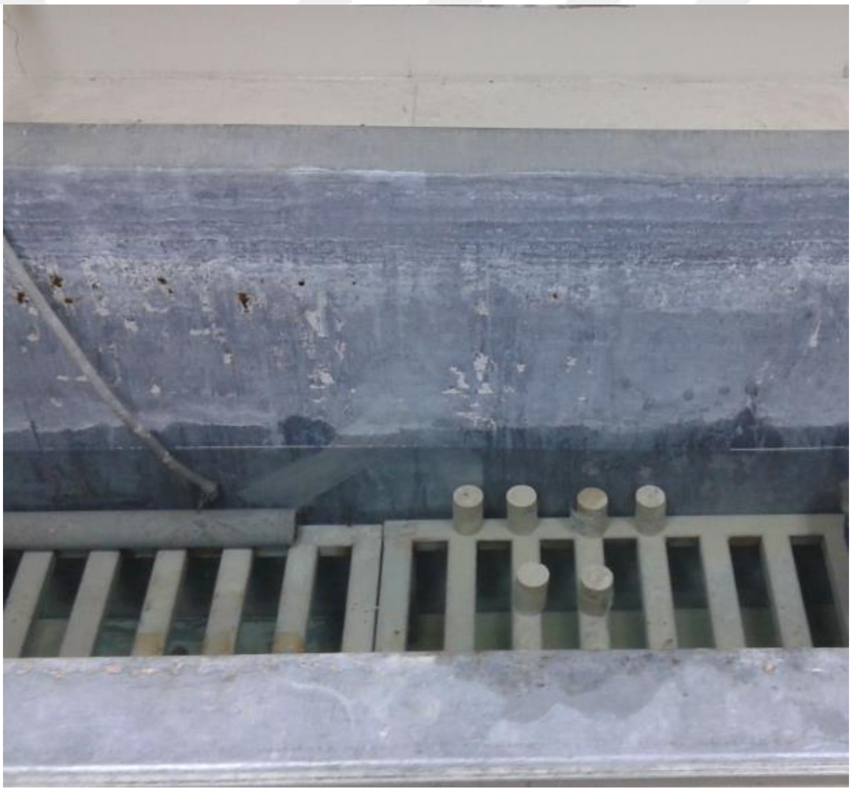

Figure 3.9 Curing of some of the cylindrical samples

Figure 3.10 Laboratory weigher

Figure 3.11 Uniaxial compression test of infilled concrete sample

Since the cylindrical sampler had a side wall thickness of 3.00mm, the diameter of the holes opened to take NX size samples and later infilled with concrete were D_{IC} = 60.7 mm while the heights of the holes were equal to the heights of the blocks (H = 150mm). An unconfined compression testing machine (UTEST UTS-0860) as shown in Figure 3.13 was modified for determination of the unit side resistance of limestone blocks. A circular loading piston having a slightly smaller diameter (D_{LP} = 60mm) than that of the infilled concrete core (D_{IC} = 60.7 mm) was mounted to the loading system (Figure 3.14) to load the concrete core in the rock block axially without any friction. Also, a stiff steel box with a hole in the middle having a dimeter slightly wider than that of the concrete core (D_h = 65mm) was put under the block to allow the slip displacement of the concrete cylinder in the rock block (as illustrated in Figure3.15). The steel box had a slightly larger surface(WidthxLength = 170x340mm) than the base of the rock block (WidthxLength = 150x300mm) to prevent any motion of the rock block during testing. The axial loads were recorded by a 100kN capacity load cell with a sensitivity of 0.001%. A displacement-controlled testing procedure was applied during the experiments. In order to determine the suitable displacement rate, tests were conducted on control blocks for both dry and fully saturated conditions for displacement rates changing between 0.001 – 0.1 mm/s. Since the obtained side resistance values were changing within a very narrow band (≈±5%) for the tested displacement rate range, the displacement rate was selected as 0.01 mm/s. This rate was both slow enough to observe the experiment and fast enough to complete it in a reasonable duration. The displacement of the rock block was also measured by a LVDT having 25mm axial displacement capacity. The unit side resistance of each block was determined by dividing the ultimate failure load to the inner surface contact area of each hole at the time of failure. The test setup and a sample view of the test is given in Figure 3.16.

(a) (b)

Figure 3.12 Some of the infilled limestone blocks (a) under curing (b) drying

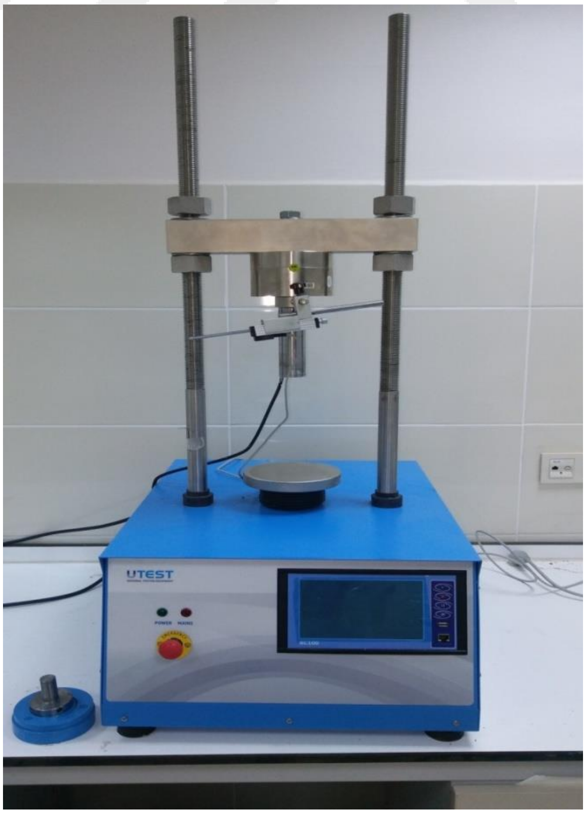

Figure 3.13 Sliding testing machine

Figure 3.14 Loading piston

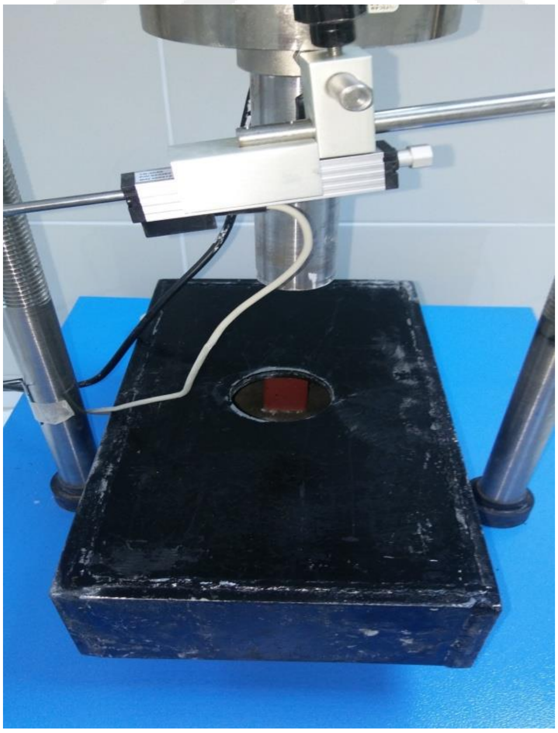

Figure 3.15 Steel box

(a)

(b)

Figure 3.16 (a) Test setup (b) A view during an experiment

CHAPTER 4

RESULTS AND DISCUSSIONS

The results of the experimental study, details of which was described in the previous chapter, were presented in the first part of this chapter. After that, the results obtained during the experiments were compared with the available literature data and discussions were made based on these comparisons. Also linear and non-linear relationsships for estimating the unit socket resistance (q_s) of Gaziantep limestone from uni-axial compressive strength (q_u) were recommended.

4.1 Results of the Experimental Study

4.1.1 Basic Properties of Gaziantep Limestone

As the first step of the experimental study, 48 cylindrical samples were taken from 24 adjacent limestone block as described in the previous chapter. Since half of the samples were going to be tested under fully saturated conditions and the other half as completely dry samples, determination of the suitable time for sample curing under water for fully saturation was so important. The drying and curing procedure was done according to the ISMR (1981) testing procedure and the basic properties of the tested blocks (unit weight, porosity and etc…) were determined according to the recommended equations in ISMR (1981) as summarized below in equations 4.1 - 4.5.

$$\rho_d = \frac{m_d}{B_v} \tag{4.1}$$

$$\rho_s = \frac{m_s}{B_v} \tag{4.2}$$

$$\rho_n = \frac{m_n}{B_v} \tag{4.3}$$

$$n = \frac{P_v}{B_v} \times 100(\%) \tag{4.4}$$

$$W = \frac{W_w}{m_d} \times 100(\%) \tag{4.5}$$

The curing tests were done for first set of samples and the results are presented below in Table 4.1. As it can be seen from Table 4.1 the minimum time needed for fully saturation of samples was determined as one week and after this stepall of the fully saturated samples were cured under water for a minimum period of one week before any further testing.

Table 4.1 Results of the Saturation Test

	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday	Monday	Tuesday
Weight (g)	Completely Dry	Saturated	Saturated	Saturated	Saturated	Saturated	Saturated	Saturated
Block 1	613,87	737,78	740,68	742,51	744,03	744,82	744,82	744,82
Block 2	608,67	730,95	734,42	734,45	734,50	736,56	737,32	737,33
Block 3	752,61	838,65	840,52	840,98	841,57	842,13	842,17	842,18
Block 4	755,25	836,45	838,06	838,82	840,45	840,79	840,87	840,87
Block 5	602,08	726,61	730,10	731,73	732,18	733,96	734,16	734,17
Block 6	597,27	721,57	725,33	726,73	728,08	729,84	730,00	730,00

Based on the data obtained during these tests, some basic properties of the Gaziantep limestone for tested samples were determined. The dry unit weight of the samples were changing between γ_{dry} = 16,62 – 21,02 kN/m^3, while the saturated unit weight of the samples were between 20,32 – 23,40 kN/m^3. The porosity of the tested samples were in the range of 24 – 38% while the water absorption capacity was between 11 – 22% by weight. These results were within a comparable range with the results presented in Canakcı (2007) and given in Table 2.1 of this study.

4.1.2 Uniaxial Compressive Strength of Gaziantep Limestone

As described previously, 48 cylindrical samples were obtained for uniaxial compression test. The testing procedure was applied in accordance with the testing procedure of ISRM (1981) as discussed in the previous chapter. 24 samples were tested in completely dry condition while the other 24 samples were tested under fully saturated conditions.

The uniaxial compressive strength of the dry samples were within the range of q_u = 15,65 – 22,07 MPa while that of the fully saturated samples were betweenq_u = 10,00 – 12,97 MPa. The reduction in the uniaxial compressive strength of Gaziantep limestone between dry and fully saturated conditions were in the range of 36,10 – 42,85% which may be assumed as 40% for any practical purpose. The results of the experiments are summarized in Table 3.2.Sample photographs taken during the experiments are given in Figure 4.1 and 4.2.

Table 4.2 Results of the Uniaxial Compression Tests

		q_u (Mpa)	Reduction in q_u (%)
Block 1	Dry	18.32	39.08
	Saturated	11.16	
Block 2	Dry	19.29	39.92
	Saturated	11.59	
Block 3	Dry	16.95	38.05
	Saturated	10.50	
Block 4	Dry	16.19	37.74
	Saturated	10.08	
Block 5	Dry	18.12	38.58
	Saturated	11.13	
Block 6	Dry	22.07	41.23
	Saturated	12.97	
Block 7	Dry	20.12	40.61
	Saturated	11.95	
Block 8	Dry	16.55	39.27
	Saturated	10.05	
Block 9	Dry	17.34	38.58
	Saturated	10.65	
Block 10	Dry	18.62	37.97
	Saturated	11.55	
Block 11	Dry	21.78	41.14
	Saturated	12.82	
Block 12	Dry	15.65	36.10
	Saturated	10.00	
Block 13	Dry	17.93	38.76
	Saturated	10.98	
Block 14	Dry	17.33	38.37

	Saturated	10.68	
Block 15	Dry	19.19	40.28
	Saturated	11.46	
Block 16	Dry	16.56	38.71
	Saturated	10.15	
Block 17	Dry	21.21	42.86
	Saturated	12.12	
Block 18	Dry	19.89	38.01
	Saturated	12.33	
Block 19	Dry	17.49	40.25
	Saturated	10.45	
Block 20	Dry	16.71	37.88
	Saturated	10.38	
Block 21	Dry	19.71	37.95
	Saturated	12.23	
Block 22	Dry	18.28	40.65
	Saturated	10.85	
Block 23	Dry	20.25	39.26
	Saturated	12.30	
Block 24	Dry	18,11	38.10
	Saturated	11.21	

Figure 4.1 Dry sample in UCS test

Figure 4.2 Wet sample in UCS test

4.1.3 Uniaxial Compressive Strength of Infilled Concrete

CEM I 42.5 cement was used for the experiments. The samples were tested based on the testing procedure proposed by ASTM C39 (ASTM Standards, 2012) to determine the compressive strength. The characteristic compressive strength of the samples were obtained as $f_{c,k}$ = 30 MPa. So, the side friction values obtained during the experiments were guaranteed to not to be controlled by the side cohesion of the infill material. A photograph taken during the experiments is given in Figure 4.3.

Figure 4.3 Concrete sample in UCS test

4.1.4 Unit Skin Resistance of Gaziantep Limestone

The unit skin resistance between concrete and Gaziantep limestone was investigated by the experimental setup established for this purpose, the details of which was given in the previous parts of this study. 48 blocks were tested during the study. Half of the samples were completely dry while the other 24 samples were in fully saturated condition. The unit skin resistance of the samples were obtained by dividing the ultimate axial load to the inner surface area of the samples.

The experimental results have revealed that the unit skin resistance of the completely dry samples were in betweenq_s = 0.78 – 1.45MPa for a rock uniaxial compressive strength range of q_u = 15.65 – 22.07 MPa. As it was expected at the beginning of this study, the unit skin resistance of the fully saturated samples was observed to decrease significantly as compared to that of dry samples. The unit skin resistance of the fully saturated samples were varying in the range of q_s = 0.36 – 0.80MPa for a rock uniaxial compressive strength range of q_u = 10.00 – 12.97 MPa.

Although the percent reduction in the uniaxial compressive strength of Gaziantep limestone between dry and fully saturated conditions were in a narrow band of 36.10 – 42.86%, the percent reduction in the unit skin resistance was changing between 23.96 – 64.23%. This fact is attributed to the changes in the inner surface structure of each tested block. Since the porosity and clay content was variable for each hole, the obtained skin resistance reduction was not directly comparable with the approximate 40% reduction in the uniaxial compressive strength of Gaziantep limestone. Nevertheless, the obtained skin resistance values have revealed an interesting behavior which is described in detail in the following parts of this chapter.

The obtained skin resistance values for each block is given in Table 4.3.

Table 4.3 Unit skin resistance values for tested samples

		q_s (Mpa)	Reduction in q_s (%)
Block 1	Dry	1.45	44.83
	Saturated	0.80	
Block 2	Dry	0.88	48.86
	Saturated	0.45	
Block 3	Dry	1.16	46.55
	Saturated	0.62	
Block 4	Dry	0.78	53.85
	Saturated	0.36	
Block 5	Dry	1.00	52.00
	Saturated	0.48	
Block 6	Dry	1.23	60.16
	Saturated	0.49	
Block 7	Dry	1.01	38.61
	Saturated	0.62	
Block 8	Dry	0.95	50.53
	Saturated	0.47	
Block 9	Dry	0.83	54.22
	Saturated	0.38	
Block 10	Dry	1.03	36.89
	Saturated	0.65	
Block 11	Dry	1.33	41.35
	Saturated	0.78	
Block 12	Dry	0.82	29.27
	Saturated	0.58	
Block 13	Dry	0.96	23.96
	Saturated	0.73	
Block 14	Dry	1.02	36.27
	Saturated	0.65	
Block 15	Dry	1.28	56.25
	Saturated	0.56	
Block 16	Dry	1.03	61.17
	Saturated	0.40	
Block 17	Dry	1.24	54.84

	Saturated	0.56	
Block 18	Dry	1.28	51.56
	Saturated	0.62	
Block 19	Dry	1.10	56.36
	Saturated	0.48	
Block 20	Dry	1.28	56.25
	Saturated	0.56	
Block 21	Dry	1.10	36.36
	Saturated	0.70	
Block 22	Dry	0.92	43.48
	Saturated	0.52	
Block 23	Dry	1.40	63.57
	Saturated	0.51	
Block 24	Dry	1,23	64.23
	Saturated	0.44	

4.2 Discussions of the Results

As it was stated in the previous parts of this document, the main aim in this study was to investigate the change of unit skin resistance of Gaziantep limestone with its uniaxial compressive strength under dry and fully saturated conditions. The relationships given in the literature which correlate the uniaxial compressive strength of rocks (q_u) with the unit skin resistance (q_s) may be divided into two main groups as linear and non-linear relationships as it was discussed in the literature review part. In this part of the study, the experimental results were compared with suitable linear and non-linear correlations and discussions are made based on these comparisons.

4.2.1 Comparison of Experimental Results with Linear Relationships

As it was given in the literature review, the proposed linear correlations of Reynolds and Kaderabek, (1981), Gupton and Logan (1984), Reese and O'neill (1988) and Toh et al. (1989) estimate a unit skin friction changing between 0.15 to 0.30 of the UCS of the rock (see Equation 2.2 – 2.5).

These linear relationships were plotted on Figure 4.4, together with the data points obtained from this study. As it can be seen on this figure, even the method with the least linear coefficient (q_s = 0,15*q_u) significantly overestimated unit skin resistance of Gaziantep limestone. This fact was also reported by Rezazadeh and Eslami (2017) for a large database of unit skin resistance values obtained for different limestones from different places of Earth. So, it may be said that care should be taken while using the linear correlations for estimating the unit skin friction of limestones. The estimated values may significantly be higher than the real values which may lead to an unsafe design.

As an alternative to the proposed methods, a new linear correlation was suggested in this study for estimating the unit skin resistance of Gaziantep limestone from uniaxial compressive strength (Equation 4.1). As it is also plotted on Figure 4.4, the proposed correlation yields to an almost unbiased estimation with a correlation coefficient of R^2 = 0,77.

$$q_s=0.056q_u \tag{4.6}$$

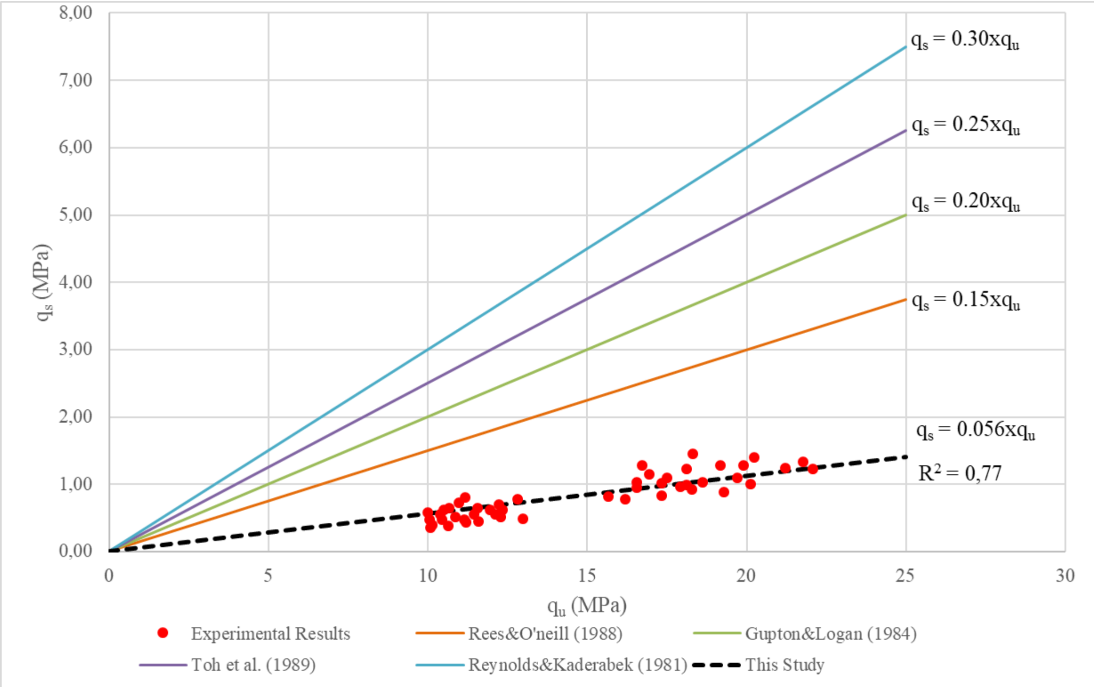

Figure 4.4 Comparison of Test Results with Linear Correlations

4.2.2 Comparison of Experimental Results with Non-Linear Relationships

In this part of the thesis, the results obtained in this study were compared with the suitable non-linear methods selected from the avaliable literature and from related manuals and design codes.

In this manner, the results were firstly compared with the methods suggested in AASHTO LRFD (2007) and CFEM (2006). As it was discussd earlier, for rocks with a uniaxial compressive strength greater than q_u> 1,9 MPa, the correlation suggested by Horvath and Kenny (1979) was recommended (Equation 2.8) by AASHTO LRFD

(2007) for estimation of unit skin resistance from uniaxial compressive strength of rock. On the other hand, the correlations suggested by Carter and Kulhawy (1988) and Rowe and Armitage (1984) were recommended as lower and upper bound values for unit skin resistance in CFEM (2006). The test result were compared with these correlations in Figure 4.5.

As it can be seen in Figure 4.5, the suggestion of AASHTO LRFD (2007) and lower bound solution of CFEM (2006) give a very close estimation of unit skin resistance. Both methods provides a reasonable lower bound estimation for dry samples. However, the unit skin resistance of fully saturated samples were mostly overetimated by the suggested methods.

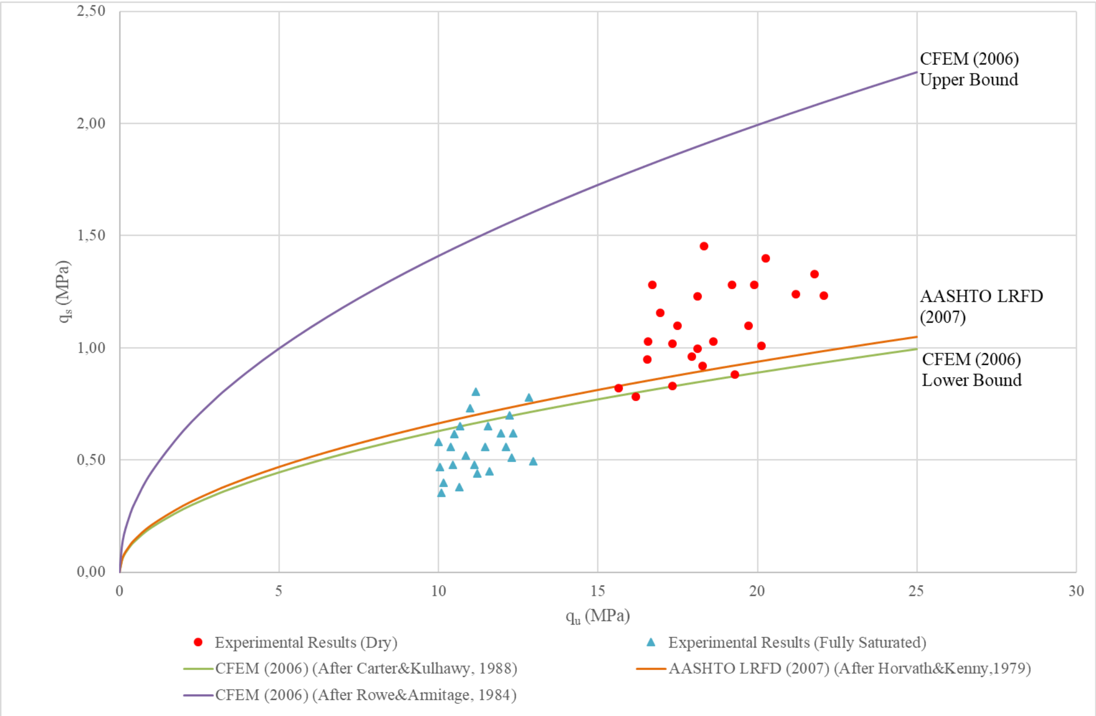

Figure 4.5 Comparison of Test Results with AASHTTO LRFD (2007) and CFEM (2006)

In a more recent correlation suggested by Kulhawy and Phoon (1993) a lowerbound, mean and upper bound solution was suggested for estimation of unit skin resistance. It should here be stated that the database utilized in this study contained a significant amount of data obtained from limestones. As it can be observed from Figure 4.6, the lower bound solution of the method suggested by Kulhawy and Phoon (1993) successfully covers even the lowest data obtained for fully saturated samples. On the other hand, the curve for “mean” values seem to provide a more realistic estimation for unit skin resistance of dry samples.

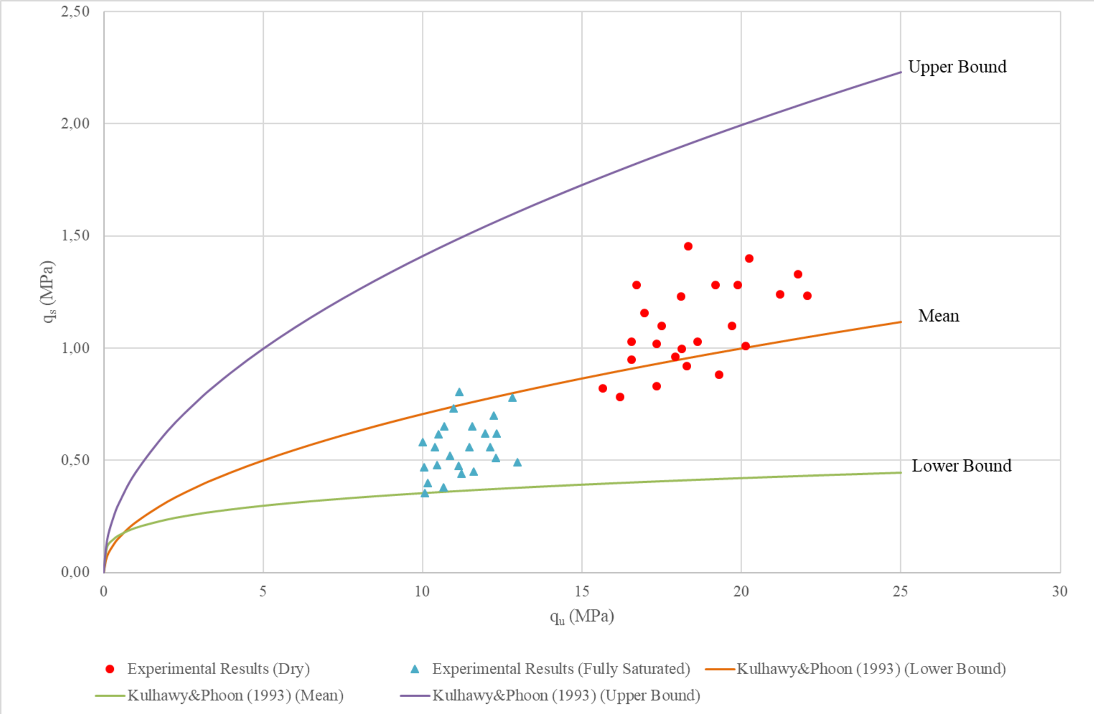

Figure 4.6 Comparison of Test Results with Kulhawy and Phoon (1993)

In the study of Rezazadeh and Eslami (2017), the skin resistance data for various types of rocks obtained from the previous studies of various researchers were analyzed in a combined manner and best-fit curves were suggested for different rock types. Among these correlations; the ones suggested for limestones and for general use which were obtained by generating a best-fit to the data only for limestones and to the combined data respectively were utilized for evaluation. Both curves give a reasonable “mean” estimation for unit skin resistance under dry conditions as it can seen in Figure 4.7. However, the unit skin resistance of Gaziantep limestone for fully saturated samples was overestimated by the suggested correlations.

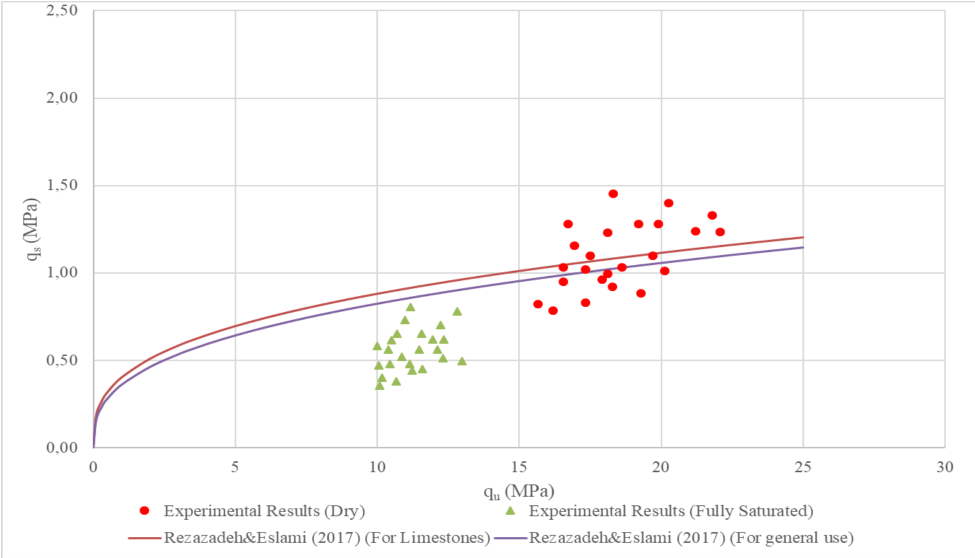

Figure 4.7 Comparison of Test Results with Rezazadeh and Eslami (2017)

Finally, a non-linear correlation range was suggested in this study, for estimation of the unit skin resistance of Gaziantep limestone from its uniaxial compressive strength (Equation 4.2 and Equation 4.3). It is suggested to use these curves as upper and lower bound solutions (Figure 4.8). It should also be emphasized that care should be taken while using the upper bound solutions especially for areas prone to saturation.

$$q_s = 0.35x\sqrt{q_u} \text{ (Upper bound solution for Gaziantep Limestone)} \tag{4.7}$$

$$q_s = 0.10x\sqrt{q_u} \text{ (Lower bound solution for Gaziantep Limestone)} \tag{4.8}$$

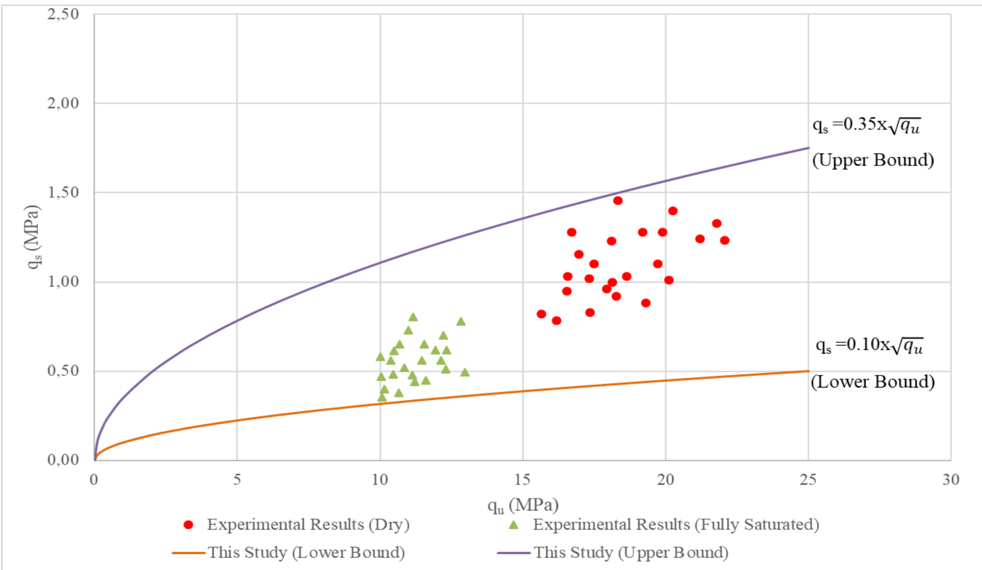

Figure 4.8 Correlation suggested for Gaziantep Limestone

CHAPTER 5

CONCLUSIONS

In this study, the unit skin resistance of Gaziantep limestone was investigated experimentally under dry and fully saturated conditions and the results were correlated with the unconfined compressive strength of the corresponding test block.

In order to conduct the test program, 24 rock blocks were collected from a rock quarry site in Gaziantep. Each block was divided into two adjacent parts to have a fully saturated and dry sample from each block. As a result, 48 unconfined compressive strength tests and 48 skin resistance tests were performed at the Civil Engineering laboratory of Hasan Kalyoncu University besides the previously described saturation and concrete strength tests.

The results have revealed that the reduction of the uniaxial compressive strength of Gaziantep limestone upon full saturation was on the order of 36.10 – 42.86% (≈40%) with respect to the completely dry case. The reduction was also observed for the unit skin resistance of Gaziantep limestone upon saturation in a wider range (23.96 – 64.23%). This fact is attributed to the difference in the the porosity and clay content of each hole. As a result of these variabilities, the obtained skin resistance reduction was not directly comparable with the approximate 40% reduction in the uniaxial compressive strength of Gaziantep limestone.

The unit skin resistance of rocks was generally correlated with the uniaxial compressive strength of the corresponding rock in the literature. The methods which recommend a correlation between the unit skin resistance and uniaxial compressive strength may be grouped in two broad categories as linear and non-linear correlations. The results of this study were compared with both suitable linear and non-linear correlations.

The comparison with the linear relationships have revealed that the unit skin resistance of Gaziantep limestone was significantly overestimated by the linear correlations for both dry and fully saturated conditions. Alternatively a new linear correlation was suggested for Gaziantep limestone with a reasonable correlation coefficient.

Among the non-linear correlations that may be found in the literature, the results of this study were compared with the methods either suggested by generally accepted standards or with the ones during development of which a data base containing data from different limestone measurements was utilized.

The first comparison was made by the methods recommended in AASHTO LRFD (2007) and CFEM (2006). Themethod recommended in AASHTO LRFD (2007) and lower bound solution of CFEM (2006) had given a very close estimation of unit skin resistance. Both methods provided a reasonable lower bound estimation for dry samples. However, the unit skin resistances of fully saturated samples were mostly overestimated by the suggested methods.

The second comparison was made by the method suggested by Kulhawy and Phoon (1993). In their method, these researchers had given three curves as loer bound, mean and upper bound solutions. Interestingly, the lower bound solution of this method was the only method that did not overestimate the unit skin resistance of Gaziantep limestone for fully saturated conditions. Even the lowest data point was within the limits of the lower bound solution. On the other hand, the curve for “mean” values seemed to provide a more realistic estimation for unit skin resistance of dry samples.

Another comparison was made by the curves recommended in Rezazadeh and Eslami (2017). The curves utilized in this comparison were suggested for limestones and for general use which were obtained by generating a best-fit to the data only for limestones and to the combined data respectively. Both curves have given a reasonable “mean” estimation for unit skin resistance under dry conditions. However, the unit skin resistance of Gaziantep limestone for fully saturate samples was overestimated by the suggested correlations.

Finally, a lower and upper bound solution was suggested in this study which may be used in calculating the ultimate skin resistance of Gaziantep limestone. It should here be stated that both the uniaxial compressive strength and the unit skin resistance of Gaziantep limestone is highly dependent on the degree of saturation. So, care should be taken while using such formulations for rock socket design in this and other similar formations, especially for places prone to saturation. It is reasonable to use the lower bound solutions of Kulhawy and Phoon (1993) and this study for such areas.

REFERENCES

AASHTO LRFD. American Association of State Highway and Transportation Officials Load and Resistance Factor Desing. 2007. Bridge Design Specifications. Washington, D.C.

ASTM. American Society for Testing and Materials. 2012. Standart Guide for the Preparation of a Binary Chemical Compatibility Chart, ASTM International, West Conshohocken, PA

ASTM. American Society for Testing and Materials. 2018. Standart Guide for the Preparation of a Binary Chemical Compatibility Chart, ASTM International, West Conshohocken, PA

Bloomquist, D., Townsend, F. C., Parra, F. 1991. Development of in situ equipment for capacity determinations of deep foundations in Florida limestone [Final report for Florida Department of Transportation]. Department of Civil Enginering, University of Florida.

Brown, E. T. 1981. *Rock characterization testing and monitorng* ISRM suggested methods. Royal School of Mines. Imperial College of Science and technology, London, England.

Canakcı, H., Baykasoğlu A, Güllü H, Özbakır L. (2007). Prediction of Compressive and Tensile Strength of Limestone via Genetic Programming. Expert Systems with Applications **35**:111-123

Canakcı, H. (2007). Collapse of Caves at Shallow Depth in Gaziantep City Center, Turkey: A Case Study, *Environmental Geology*, **53** (4), 915-922

Carter, J.P., Kulhawy, F.H. 1988. Analysis and design of foundations socketed into rock [Report EL-5918]. Palo Alto, USA: Electronic Power Research Institute.

Carter, J. P., Kulhawy, F. H. (1992). Analysis of laterally loaded shafts in rock. *Journal of Geotechnical Engineering,* **118**(6), 839-855.

CFEM. 2006. Canadian foundation engineering manual. British Columbia: BiTech Publishers Ltd.

De Freitas, (1979). *A Geology for Engineers.* 6th edition Formerly Reader in Engineering Geology, Imperial College of Science and Technology, London Lecturer in Engineering

Elias, V., Juran, I. 1991. "Soil Nailing for Stabilization of Highway Slopes and Excavations," Publication FHWA-RD-89-198, Federal Highway Administration, Washington D.C.

Gunnink, B., Kiehne, C. (2002). Pile bearing in Burlington limestone. In: Transportation conference proceeding; pp. 145-8.

Gupton, C., Logan, T. (1984). Design guidelines for drilled shafts in weak rocks of South Florida. In: Proceeding of the South Florida annual ASCE meeting. Reston: ASCE.

Horvath, R.G., Kenney, T.C., Kosicki, P. (1983). Method of improving performance of drilled piers in weak rock. *Canadian Geotechnical Journal;* **20**(4):758-72.

Horvath, R.G., Kenney, T.C. (1979). Shaft resistance in rock socketed drilled piers. In: Proceedings of the symposium on deep foundations. Reston: ASCE. p. 182-214.

http://www.worldeasyguides.com/europe/turkey/gaziantep/where-is-gaziantep-on-map-turkey/ (arrival date: 06.01.2019)

Brown E. T. 1981. ISRM. Suggested Methods Rock Characterization, Testing and Monitoring. Pergamon Press, London, 211 s.

Brown E. T. 2007. ISRM.. ISRM Suggested Methods Rock Characterization, Testing and Monitoring. Pergamon Press, London, 211 s.

Kılıc A. M. (2015). Investigation of Blasting Parameters of Gaziantep-Şehit Kamil-Karpuzkaya Area Limestone Quarry and Determination of Post-Blast Particle Size

Distribution, *Çukurova University Journal of the Faculty of Engineering and Architecture*, **30**(2), 217-226.

Kulhawy, F. H., Akbas, S. O., Prakoso, W. A. (2005). Evaluation of capacity of rock foundation sockets. In *Alaska Rocks 2005, The 40th US Symposium on Rock Mechanics (USRMS)*. American Rock Mechanics Association.

Kulhawy, F. H., Phoon, K. K. (1993). Drilled shaft side resistance in clay soil to rock. In: Proceedings of the conference on design and performance of deep foundations: piles and piers in soil and soft rock. Reston: ASCE. pp. 172-83.

Marangoz L. 2005. Correlation of geotechnical properties of limestone with ultrasonic pulse velocity in Gaziantep region. M.Sc. thesis submitted to school of natural and applied science, University of Gaziantep.

McVay, M. C., Townsend, F. C., William, R. C. (1992). Design of socketed drilled shaft in limestone. *Journal of Geotechnical Engineering*. **118**(10), 1626-37.

Meigh, A.C., Wolski, W. (1979). Design parameters for weak rocks. In: Proceeding of the 7th European conference on soil mechanics and foundation engineering. Brighton: British Geotechnical Society; pp. 59-79.

MTA. General Directorate of Mineral Research and Exploration. 1994. Geological Map of The Gaziantep-K24 Quadrangle. Ankara, Turkey.

Ramamurthy, T., Arora, V. K. (1993). A classification for intact and jointed rocks. In A. Anagnostopoulos et al. Geotechnical engineering of hard soils-soft rocks (pp. 235 - 242). Rotterdam: A.A. Balkema.

Rezazadeh, S., Eslami, A. (2017). Empirical methods for determining shaft bearing capacity of semi-deep foundations socketed in rocks. *Journal of Rock Mechanics and Geotechnical Engineering;* **9**(6), 1140-51.

Reese, L.C., O'Neill, M.W. (1988). Drilled shafts: construction procedures and design methods [Publication no. FHWA-HI-88-042]. Washington, D.C.: Federal Highway Administration.

Reynolds, R.T., Kaderabek, T.J. 1981. Miami limestone foundation design and construction. New York: ASCE. p. 859-72.

Rosenberg, P., Journeaux, N.L. (1976). Friction and end bearing tests on bedrock for high capacity socket design. *Canadian Geotechnical Journal;* **13**(3), 324-33.

Rowe, R. K., Armitage, H. H. (1987). A Design method for drilled piers in soft rock. *Canadian Geotechnical Journal;* **24**(1), 126-42.

Rowe, R. K., Armitage, H. H. 1984. Design of piles socketed into weak rock [Geotechnical research report]. London, Ontario, Canada: University of Western Ontario.

Salgado, R. 2008. The engineering of foundations (Vol. 888). New York: McGraw-Hill.

Serrano, A., Olalla, C. (2004). Shaft resistance of a pile embedded in rock. *International Journal of Rock Mechanics and Mining Sciences;* **41**(1), 21-35.

Terlemez I, Sentürk K, Sümengen M, Oral A. (1997). Türkiye jeoloji haritaları. No: 45. Jeoloji etütleri dairesi, Ankara

Toh, C.T., Ooi, T.A., Chiu, H.K., Chee, S.K., Ting, W.N. (1989). Design parameters for bored piles in a weathered sedimentary formation. In: Proceeding of the 12th international conference on soil mechanics and foundation engineering; pp. 1073-8.

Williams, A. F., Johnston, I. W., Donald, I. B. (1980). Design of socketed piles in weak rock. In: Proceeding of international conference on structural foundations on rock; pp. 327-47.