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Characterizing and Modeling of Ultra-Soft Clay Soil, Filter Cake and Drilling Mud

A Dissertation

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By

Aram Mohammed Raheem

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Characterizing and Modeling of Ultra-Soft Clay Soil, Filter Cake and Drilling Mud

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DEDICATION

The sum of the past five years is dedicated to

My wife **Enas Najim**

Her love and support allowed me to rearrange my life to build a new identity.

The loving memory of my father

Mohammed Raheem Kaka Weis

My lovely kids

Ahmed Mohammed and Areen Mohammed

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An Abstract

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Abstract

In this study, ultra-soft soils representing the deepwater seabed offshore, coastal soils, and onshore soils with filter cakes and drilling muds, was characterized using new non-destructive in-situ test methods and modeling of the behavior. The new test methods to characterize the ultra-soft soils included the two-probe electrical method and CIGMAT miniature penetrometer. The clay content in the ultra-soft soils, filter cakes and drilling muds investigated in this study varied from 2% to 10% by weight. The type of clays investigated include montmorillonite (bentonite) and kaolinite. The shear strength of the ultra-soft soils varied from 0.01 kPa to 0.30 kPa using the modified vane shear test. Electrical characterization of the ultra-soft soils identified the soil to be a resistive material. Several modifiers such as lime, polymer, sand, and cement were used to treat the ultra-soft soils. The effect of the modifiers on the shear strength, electrical resistivity, water content, density, and electrical impedance were investigated. The shear strength of the treated ultra-soft soil had the highest value of 6.8 kPa, a change in shear strength of 2167%, with 10% polymer treatment. Electrical resistivity was correlated with the solid content, shear strength, and water content for treated and untreated ultra-soft soils. Experimental, analytical, statistical, and finite element methods were used to model the stress-strain relationship of the ultra-soft soils.

Filter cake formation and fluid loss occur concurrently during various engineering operations including during oil well drilling is influenced by the seepage and consolidation of the cake. A new coupling continuous function with time and depth variables was developed to represent the combined seepage-consolidation phenomenons during the filter cake formation under different pressures and temperatures. The new

continuous function solution was compared with Terzaghi discrete consolidation solution and both solutions were verified using several experimental results. Currently, filter cake is modeled using the API method where the cake properties are assumed to be constant but the cake thickness varies with time. In the new kinetic model developed in this study, variations of fluid loss, porosity, permeability, relative solid content, and cake thickness with time have been included. The new kinetic model also takes into account the effects of both high pressure and high temperature. Also, the new kinetic model has a limit on the maximum amount of the fluid loss, however, the API method predicts the maximum fluid loss to be infinity. The prediction for both API and new kinetic models were verified using several high pressure and high temperature test results from the current study and reported literature.

Drilling mud rheological behavior with and without contamination was investigated under different temperatures using the Herschel-Bulkley and hyperbolic models. Nonlinear models were used to investigate the combined effects of bentonite and salt contamination, and the changes in the temperature on the fundamental properties of the drilling mud such as yield and maximum shear stress, electrical resistivity and other hyperbolic model parameters. Nonlinear model showed that the bentonite content in the drilling mud had the highest effect in decreasing the electrical resistivity, yield and maximum shear stresses compared to salt contamination and temperature in the range of studied variables.

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Chapter 1 - Introduction

1.1. General

Ultra-soft soils are encountered in both offshore and onshore areas (Bo et al. 2007). The ultra-soft soils exhibits high water content and low shear strength (Bartos, 1977; DeMeyer and Mahlerbe, 1987). Due to the modern urbanization of the onshore areas and laying deep water oil pipelines on soft seabed and oil well construction in the offshore areas, the ultra-soft soil characterization has gained great interest (Gallant et al., 2007). Most of the available methods to characterize the soft soils are mainly appropriate for the offshore soil at higher depths with higher strength soils rather than the top soft surface soils. Traditionally, different types of penetrometer such as cone penetration test (CPT), standard penetration test (SPT), T-bar, and ball penetration methods are being used to characterize the in-situ soils. All these types of penetrometers have heavy weights and inappropriate to be used for near surface soft soil characterization. Hence, to better utilize the near surface ultra-soft soils along the coastal areas and the deep water conditions, it is a need for developing in-situ methods to better characterize the near surface ultra-soft soils (Lunne, 2001).

Filter cake builds up over the face of the porous medium during the drilling operation and the filtrate is lost into the formation and affecting the performance of the drilling muds. The filter cake is a thin, soft clayey layer having similar characteristics as soft soil. Filter cake are formed under different conditions such deep oil wells, bored piles, tunneling, slurry dewatering, vertical cutoff walls, and solid–liquid separation during wastewater treatment. Filter cake formation must be characterized with time in terms of fluid loss, cake permeability, cake relative solid content, cake porosity, and cake

thickness. The American Petroleum Institute (API) model is mainly used to characterize the fluid loss with the time for 30 minutes. The API-model was derived based on the assumption that the filter cake permeability and relative solid content are time independent. Hence, there is a need to develop a model taking into account the change in filter cake porosity, permeability, relative solid content and cake thickness with time during the formation of the filter cake.

The ultra-soft soils and filter cakes in onshore or offshore conditions can be contaminated by salt (Jilani et al., 2002). Salt can affect the soft soil properties such as shear strength. Also, salt contamination can affect the rheological properties of the drilling mud, which is the source for the filter cake. Temperature is another important factor that can affect the properties of the soft soil and filter cake. Hence, there is need to quantify the combined effects of both temperature and salt contamination.

1.2.Objectives

The main objectives of this study are listed below:

1. Develop non-destructive in-situ test methods to characterize the behavior of ultra-soft with and without modification.
2. Correlate the properties of ultra-soft soil with and without treatment with non-destructive test results.
3. Characterize the effects of temperature and contamination on the drilling mud.
- 4.Characterize and model the filter cake formation and fluid loss.

1.3.Organization

Chapter two describes the background of the soft soil characterization, including the history of soil classification, different types of penetrometers, and popular soft soil improvement agents. In addition, it summarizes filter cake properties and effect of contamination on the drilling mud behavior. Chapter three summarizes the materials and methods that have been used in this study. Chapter four focuses on soft soil characterization and improvement using CIGMAT penetrometer and electrical resistivity method. Chapter five investigates the filter cake characterization and introduces a new kinetic model to predict the fluid loss behavior. Chapter six concentrates on the effect of both temperature and salt contamination on the behavior of the drilling mud. Finally, the conclusions and recommendations are given in chapter seven.

Chapter 2 - Background

2.1 Soft soil

2.1.1. History of bentonite soft soil

In 1847 the name "montmorillonite" was given to a rose-red, clay-like mineral forming nests in a brown clay at Montmorillon, France (Hauser and Colombo, 1953). The first analysis, reported by Salvétat using H-clay titration method, gave 49.4 % to SiO_2 , 19.7 % to Al_2O_3 , 0.8 % to Fe_2O_3 , 0.27 % to MgO , 1.5 % to CaO , 1.5 % to alkalies, and 25.67 % to H_2O . Much later, a substance that was originally called "taylorite" after William Taylor (1953), who was the first to draw attention to it, was finally designated as "bentonite" because it was first found in the Fort Bemon series of rocks in Montana in the united states. The most characteristic mineral component of bentonite is crystalline and definitely montmorillonite (Hauser and Colombo, 1953).

It was not until 1916 that Leitmeier classified as montmorillonite a yellow clayey mineral found in Bulgaria (Leitmeier, 1916). The sample had lain in the laboratory for three years. When analyzed using titration method, it was found to contain: 50.14% SiO_2 , 19.74% Al_2O_3 , 4.14% Fe_2O_3 , 1.26% CaO , 2.28% MgO and 22.61% water (total: 100.17 percent). Leitmeier (1916) assumed that part of the Fe_2O_3 was actually replacing Al_2O_3 , leaving the balance as well as MgO and CaO as impurities. Based on this assumption, he figured the formula of the pure crystalline material to be $\text{Al}_2\text{O}_3 \cdot 4\text{SiO}_2 \cdot 6\text{H}_2\text{O}$.

Ross and Shannon (1926) gave the formula of montmorillonite as $(\text{MgCa})\text{O} \cdot \text{Al}_2\text{O}_3 \cdot 5\text{SiO}_2 \cdot n\text{H}_2\text{O}$ and state that its physical form is micaceous and its crystal form

orthorhombic. Their analysis, made from material, which had been washed free of impurities, indicated that its composition agreed closely with material from Montmorillon, France. The only real difference was the much higher MgO content than that originally reported by Salvétat.

Ross and Shannon (1926) also analyzed the montmorillonite obtained from Montmorillon, France. and offered the following results: 38.6% SiO₂, 20.03% Al₂O₃, 1.25% Fe₂O₃, 0.16% MnO, 1.72% CaO, 5.24% MgO and 21.52% water. They also offered the following data for the water loss of the same clays: at 100° C, 11.88%; at 140° C, 2.00%; at 450° C, 2.64% and at red heat, 5.0% (total 21.52 %).

In 1897 W. C. Knight described a variety of clay under the name taylorite. After he found that that name was already preoccupied, he proposed the name bentonite, since the clay was originally discovered near Fort Benton, northeast of Great Falls, Montana (Knight, 1898). At that time, colloid science had not yet become sufficiently known and established; therefore, almost no progress had been made in regard to the most important applications of bentonites. Now the situation is entirely different and important contributions have been made, particularly during the second quarter of last century. A considerable amount of basic research has been published on bentonite colloidal analysis (Bradfield, 1927, and 1931 ; Bradley, 1945; Bradley and Grim, 1948; Foshag and Woodford, 1936; di Gleria and Zucker, 1931; Grim, 1942; Hauser, 1945; Hauser and le Beau, 1938, 1939, 1941, and 1952; Hauser et al., 1951; Hauser and Leggett, 1940; Hendricks, 1945; Marshall, 1949; Perkins, 1952; Ross and Hendricks, 1945; and Williams et al., 1953). In Table 2.1, the chemical analysis of different types of bentonite has been identified.

Table 2.1 Chemical analysis (Hauser and Colombo, 1953)

	Wyoming Bentonite (%)	Hectorite (%)	Cherry Lease (%)	Princeton (%)	Kisameet Bay (%)
Water Loss at 110° C	13.75	14.56	13.36	7.86	4.36
SiO₂	58.6	54.28	58.84	67.77	52.52
Al₂O₃	19.59	0.68	18.39	17.13	19.40
Fe₂O₃	3.19	0.10	2.82	2.12	3.77
FeO	-	-	-	-	4.52
CaO	0.37	0.50	2.29	2.92	4.35
MgO	2.51	25.25	2.12	1.84	4.53
Na₂O	1.69	3.19	0.91	0.42	1.57
K₂O	0.32	0.16	0.39	-	3.83
Li₂O	-	1.02	-	-	-
CO₂	-	-	0.36	-	0.16
SO₃	-	-	0.16	-	-
TiO₂	-	-	0.16	-	1.12
Total	100.02	99.74	99.80	100.06	100.13

All montmorillonites and bentonites have the basic structural pattern of mica, namely, two silica sheets that enclose an alumina sheet. One of the most important factors is that the spacing between the lattice units varies with the moisture content in the case of sodium bentonites. This is the reason why they are frequently referred to as the "expanding lattice" group; this group comprises four main minerals, bentonite, saponite, beidellite, and nontronite. In the clay mineral, hectorite and aluminum have been largely replaced by magnesium. In the bentonites, the seat of the net charge is separated from the water layers and from the exchangeable cations by the silica sheets, which in the case of sodium bentonites appear to hinder a close association between the cations and the lattice. Marshall (1949) has already pointed out that in general, the greater dissociation of Na-bentonites than beidellites can be accounted for quantitatively.

All clays of the montmorillonite type fit very well into McBain's (1950) definition of colloidal electrolytes since, in an aqueous suspension, each lattice unit comes to equilibrium with the outer solution. On this basis, it has been assumed that the base-exchange capacity should be almost independent of particle size, and Marshall confirmed this. Hauser and Reed (1936 and 1937) have shown that relatively monodisperse fractions of electrodyalized sodium bentonite all give the same pH-versus-concentration curve, irrespective of particle size. This strongly reinforces the suggestion that bentonites must be classified as colloidal electrolytes.

In recent studies, the Wyoming bentonite that has been used the clay was characterized as having montmorillonite (MMT) (hydrated sodium calcium aluminum magnesium silicate hydroxide $(\text{Na,Ca})_{0.33}(\text{Al,Mg})_2(\text{Si}_4\text{O}_{10})(\text{OH})_2 \cdot n\text{H}_2\text{O}$) (2 θ peaks at 7.51° , 28.12° , 35.10° , 48.02° , 52.31° and 76.20°), kaolinite ($\text{Al}_2\text{Si}_2\text{O}_5(\text{OH})_4$) (2 θ peak at 11.89° and 42.12°), feldspar (Albite) ($\text{NaAlSi}_3\text{O}_8$) (2 θ peaks at 9.81° , 14.32° , 21.03° , 29.40° and 30.01°), beidellite ($(\text{Na}, \text{Ca}_{0.5})_{0.3}\text{Al}_2((\text{Si},\text{Al})_4\text{O}_{10})(\text{OH})_2 \cdot n\text{H}_2\text{O}$) (2 θ peak at 62.05° and 73.88°) and quartz (SiO_2) (2 θ peaks at 32.09° , 50.10° and 68.20°) as shown in Figure 2.1 (Vipulanandan and Mohammed, 2014).

2.1.2. History of kaolinite soft soil

In the south-west coal-field of east Glamorganshire (UK)-especially in the lower coal measures- a white, soft and pulverulent substance was found (Butler, 1911). The sample was observed to consist of a congeries of well-defined crystals of kaolinite. The crystals were chiefed as basal flakes, hexagonal in outline, and 0.02 mm to 0.037 mm in length. Most of them show elongation in one direction, and unequal extension of the thin lamella composing them.

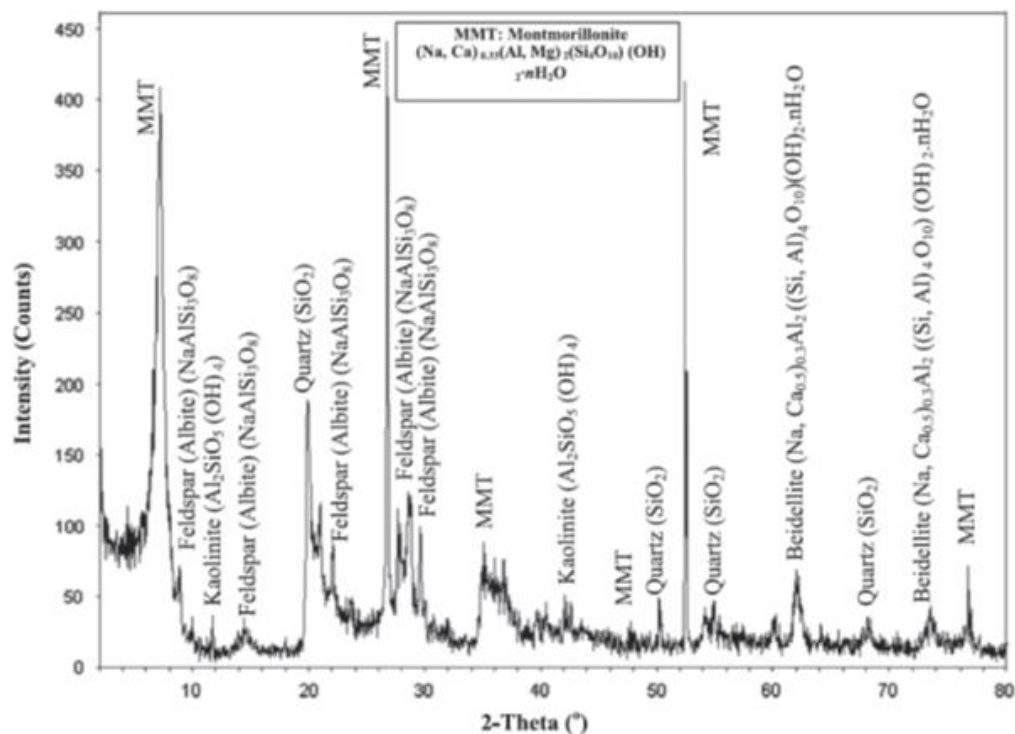


Figure 2.1 XRD pattern of the Wyoming bentonite (Vipulanandan and Mohammed, 2014).

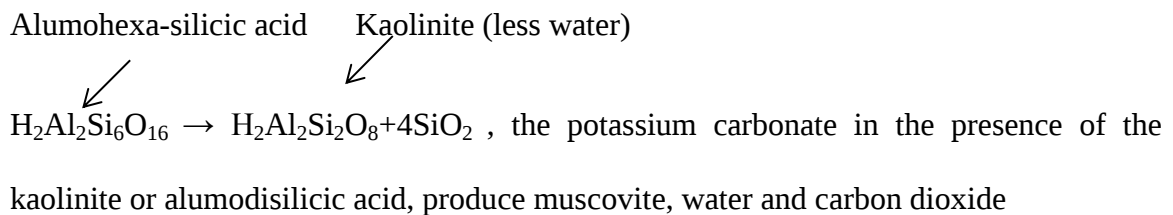
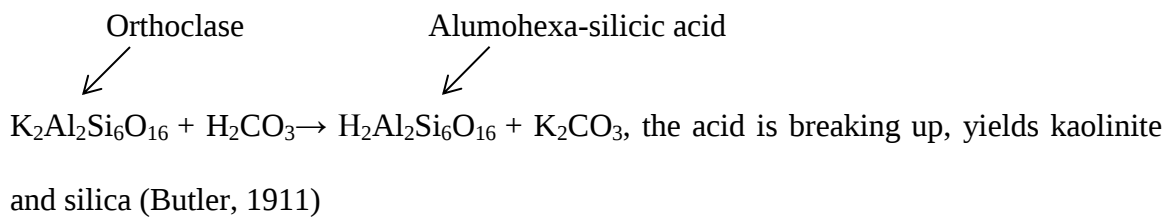
The specimen consists of a white crystalline powder and it was made of small hexagonal plates, which have the characteristic form and appearance of kaolinite. The maximum refractive index is less than 1.57, and the minimum greater than 1.56 (Butler, 1911). In composition, the material is hydrated silicate of aluminum. A determination of water by loss on ignition, in a portion practically free from calcareous and carbonaceous matter, gave 18.78 %.

The kaolinite may be traced as specimens indicate through every accessible bed of rock. It may be realized lying along the cleavage-planes of the coal at a depth of 560 yards from the surface. The most abundant deposits, varying in thickness from that of a slight film to a quarter of an inch, form the central band in vertical fissures in the grits, which are lined with quartz and dolomite or calcite, and may be studded with pyrite and

millerite crystals. Obviously, the kaolinite was the last mineral to occupy the fissures. In places, more particularly along fault planes, it is admixed with muddy and other foreign material. Its usual freedom from impurities and its loose state of aggregation probably point to quiet deposition from suspension in water that percolated through the Coal measures.

It has been suggested that kaolinite, which has the composition $\text{H}_2\text{Al}_2\text{Si}_2\text{O}_8 \cdot \text{H}_2\text{O}$, may, without its water of crystallization, be regarded as the first of a series of acids of which cimolite and pyrophyllite are respectively the second and third members. These have the general formula $\text{H}_2\text{Al}_2\text{Si}_n\text{O}_{2n+4}$, and yield the ions H_2 and $\text{Al}_2\text{Si}_n\text{O}_{2n+4}$. When n exceeds four, the silicic acid is readily broken up. Orthoclase is the potassium salt of aluminohexasilicic acid, $\text{H}_2\text{Al}_2\text{Si}_6\text{O}_{16}$ the fifth member of Morozewicz's series. X-diffraction pattern and SEM micrograph of the untreated kaolinite can be identified in Figure 2.2 (Rios et al., 2007).

First, the conversion of the feldspar into aluminohexasilicic acid and potassium carbonate (Butler, 1911)



Kaolinite (less water)

Muscovite

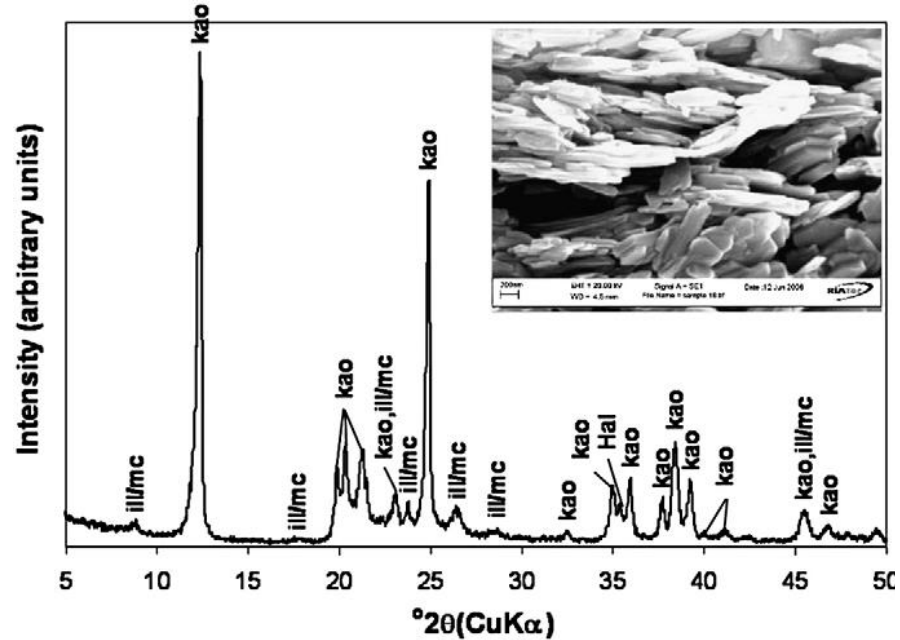


Figure 2.2 X-diffraction pattern and SEM micrograph of the untreated kaolinite (Rios et al., 2007).

2.1.3. General soil classification

In the following section, different tables and figures are going to be displayed from different old studies that normally used for soil classification especially fine grained soils. First, ranges for general soil properties are listed in Table 2.2 (Beata and Imre, 2009). Second, identification of composite clay soils based on plasticity basis can be identified as shown in Table 2.3 (Burmister, 1951). Third, clayey soil consistency can be clearly identified as shown in Figure 2.3 (Das, 2012). Fourth, particle size distribution for different types of soil has been described in Table 2.4 (Beata and Imre, 2009). Fifth, shear strength properties of different types of fine grained soils can be seen in Table 2.5 (Nagaraj et al., 2012).

Table 2.2 General soil properties (Beata and Imre, 2009)

Property	Soil Type	Range
Water content (%)	Sands	~5%
	Clays	20-30%
	Organic soils	100-300%
Void ratio	Dense sandy gravel	0.3
	Loose sand	0.6
	Clays	0.5~1.0
Porosity	Loose soils	~50%
	Compact soils	~30%
	Cohesive soils	50-70%
	Fibrous peat	80-90%
Saturation	Soil under groundwater level	100%
	Sands above groundwater level	20-40%
	Clays above groundwater level	80-90%
Density (gm/cm ³)	Solid granular of solids	2.65-2.85
	Natural density of soils	1.8-2.1
Activity*	Kaolinite	0.25
	Illite	0.4
	Montmorillonite	>=1.25

*Activity (A) of a soil is the PI divided by the percent of clay-sized particles (less than 2 μ m) present.

Table 2.3 Identification of composite clay soils on an overall plasticity basis (Burmister, 1951)

Degree of Overall Plasticity	PI	Identification (Burmister system)	Smallest Diamter of Rolled Threads, mm
Non-plastic	0	Silt	None
Slight	1-5	Clayey silt	6
Low	5-10	Silt and clay	3
Medium	10-20	Clay and silt	1.5
High	20-40	Silty clay	0.8
Very high	>40	Clay	0.4

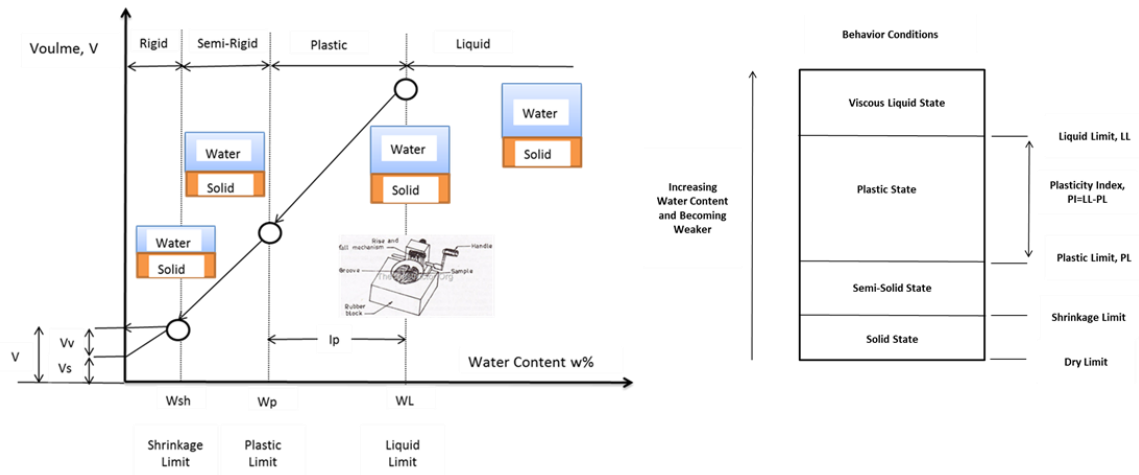


Figure 2.3 Clayey soil consistency (Das, 2012).

Table 2.4 Particle size distribution of different types of soils (Beata and Imre, 2009)

Soil Fractions	Sub-Fractions	Symbols	Particle Sizes (mm)
Very Coarse Soil	Large boulder	LBo	>630
	Boulder	Bo	200 to 630
	Cobble	Co	63 to 200
Coarse Soil	Gravel	Gr	2 to 63
	Coarse gravel	CGr	20 to 63
	Medium Gravel	MGr	6.3 to 20
	Fine Gravel	FGr	2 to 6.3
	Sand	Sa	0.063 to 2
	Coarse sand	CSa	0.63 to 2
	Medium sand	MSa	0.2 to 0.63
Fine Soil	Fine Sand	FSa	0.063 to 2
	Silt	Sl	0.002 to 0.063
	Coarse silt	CSl	0.02 to 0.063
	Medium silt	MSl	0.0063 to 0.02
	Fine silt	FSl	0.002 to 0.0063
	Clay	Cl	≤ 0.002

Sixth, clay sensitivity has been presented in Table 2. 6 (Skempton and Northey, 1952). In Table 2.7, Naval facility engineering command for fine grained soil has been shown (1986).

Table 2.5 Index properties of soils (Nagaraj et al., 2012)

Consistency	Shear Strength (kPa)	Unconfined compression Strength (kPa)	Feel or touch
Soft	< 24.5	<49	Easily Penetrated Several Inches with Thumb
Medium stiff	24.5-49	49-98	Moderate Effort Penetrates Several Inches with Thumb
Stiff	49-98	98-196	Readily Indented with Thumb but Penetrated Only with Great Efforts
Very stiff	98-196	196-392	Readily Indented by Thumbnail Only
Hard	>196	>392	Indented with Difficulty by Thumbnail

Table 2. 6 Clay sensitivity* (Skempton and Northey, 1952)

Type	Sensitivity Value
Non-Sensitive	2-4
Sensitive	4-8
Highly Sensitive	8-16
Quick	>16

*Sensitivity of clays is defined as the ratio of their undisturbed and remoulded strengths, and varies from about 1 for heavily over-consolidated clays to values of over 100 for the so-called extra-sensitive or “quick” clays.

Table 2.7 Guide of consistency of fine-grained (cohesive) soils (NAVFAC DM 7.1, 1986)

SPT, Penetration N Value (blows/foot)	Estimated Consistency	Estimated Range of Unconfined Compressive Strength (kPa)
<2	Very soft (extruded between fingers when squeezed)	Less than 26.91
2-4	Soft (molded by light finger pressure)	26.91-53.82
4-8	Medium (molded by strong finger pressure)	53.82-107.64
8-15	Stiff (readily indented by thumb but penetrated with great effort)	107.64-215.28
15-30	Very stiff (readily indented by thumbnail)	215.28-430.56
>30	Hard (indented with difficulty by thumbnail)	>430.56

The identification and characterization of special weak materials according to NAVAC DM 7.1 has been shown in Table 2.8 while the cohesion and adhesion

properties of different interface materials can be seen in Table 2.9. Identification of fine-grained soil fraction from manual tests can be seen in Table 2.10.

Some other classifications are based on stickiness of fine grained materials as follows (Das, 2012) (Figure 2.4)

- Non-Sticky: little or no soil adheres to fingers after release of pressure.
- Slightly Sticky: soil adheres to both fingers after release of pressure with little stretching on separation of fingers.
- Moderately Sticky: soil adheres to both fingers after release of pressure with some stretching on separation of fingers.
- Very Sticky: soil adheres firmly to both fingers after release of pressure with stretches greatly on separation of fingers.

Non-
Sticky



Slightly-
Sticky



Very-
Sticky



Figure 2.4 Different stickiness conditions (Das, 2012).

In addition, different important criterion for soil strength, soil dilatancy, soil toughness and organic content are reported. For instance, criterion for soil strength

evaluation has been shown in Table 2.11 while soil dilatancy criterion has been given in Table 2. 12 (Das, 2012). Moreover, criterion for soil toughness and identification of inorganic fine grained soils can be seen in Table 2.13 and Table 2.14 respectively (Das, 2012). In Figure 2.5, the extended plasticity chart with tested Bentonite properties can be seen.

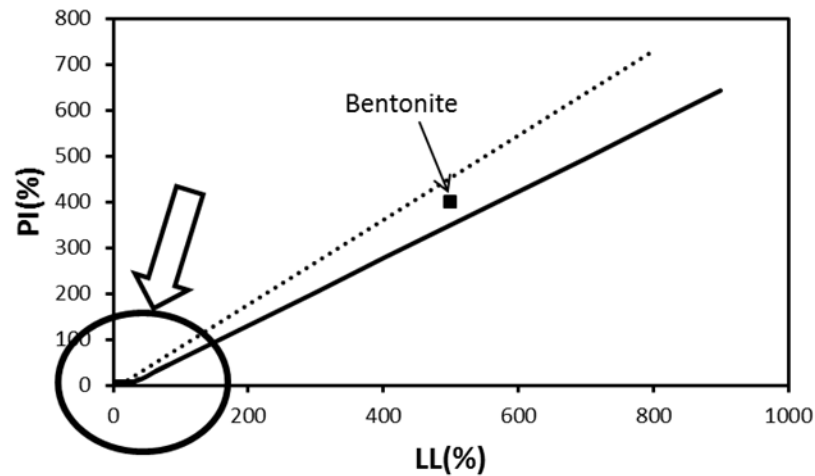


Figure 2.5 Extended plasticity chart.

Figure 2.6 has clearly showed the first part of plasticity chart including the location of several different materials on the chart (Das, 2012). More obvious soil plasticity and compressibility locations can be seen in Figure 2.7 (Das, 2012).

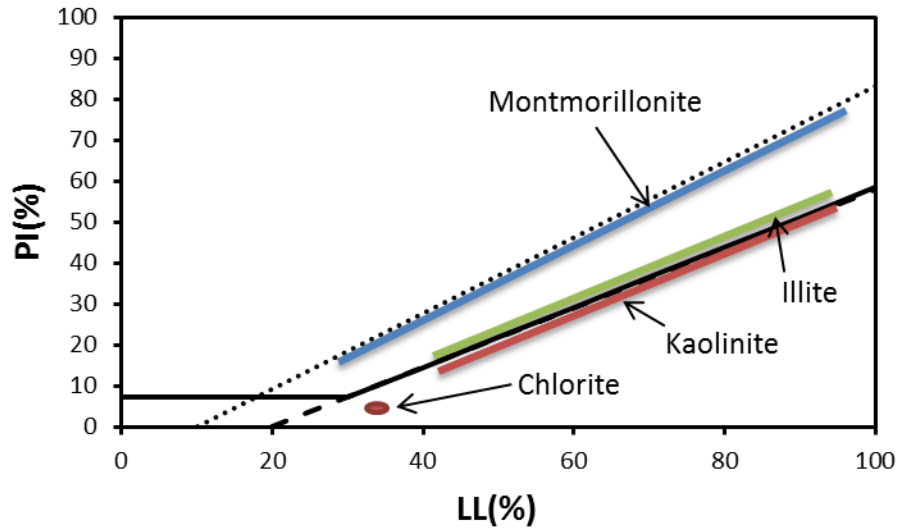


Figure 2.6 Plasticity chart (Das, 2012).

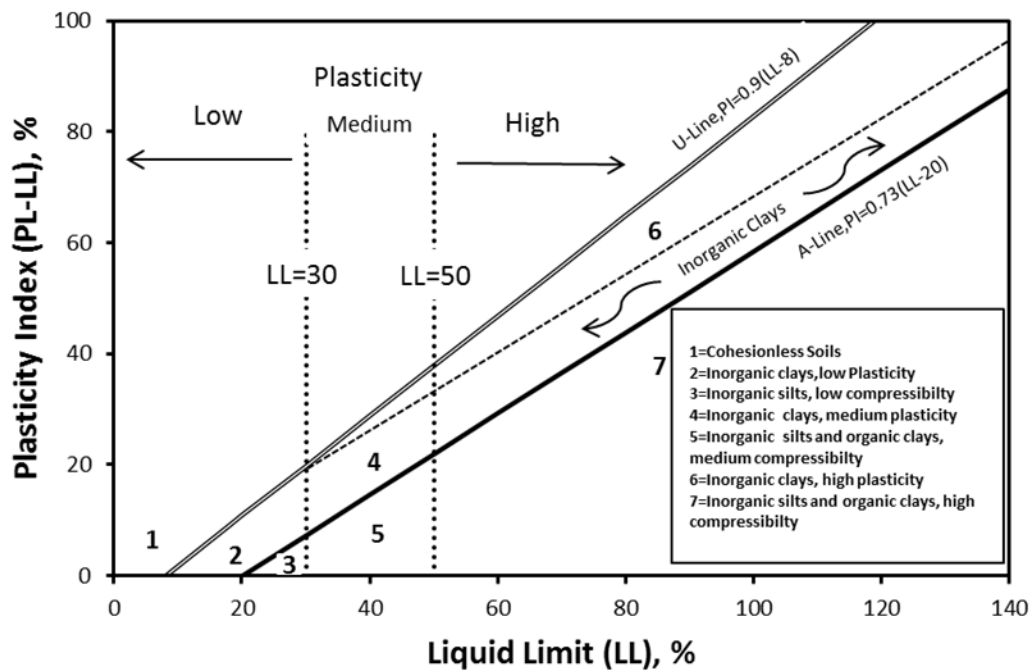


Figure 2.7 Different soil plasticity and compressibility locations on plasticity chart (Das, 2012).

Table 2.8 Identification and characteristics of special materials (NAVFAC DM 7.1, 1986)

Material	Geographic/ Geomorphic Features	Engineering Conditions
Quick Clay	Marine or brackish water clay composed of glacial rock flour that is elevated above sea level. Generally confined to far north areas, Eastern Canada, Alaska, and Scandinavia.	Severe loss of strength when disturbed by construction activities or seismic ground shaking. Replacement of formation water containing dissolved salt with fresh water results in strength loss. Produces landslide prone areas (Anchorage, Alaska).
Hydraulic Fills	Coastal facilities, levees, dikes, tailing dams.	High void ratio. Uniform gradation but variable grain sizes within same fill. High liquefaction potential. Lateral spreading. Easily eroded.
Collapsing Soil	Desert arid and semi-arid environment. Alluvial valleys, playas, loess.	Loss of strength when wetted. Differential settlement. Low density. Moisture sensitive. Gypsum/ Anhydrite often present.

Table 2.9 Cohesion and adhesion of interface materials (NAVFAC DM 7.1, 1986)

Interface Materials (Cohesion)	Adhesion Ca (kPa)
Very soft cohesive soil (0-12 kPa)	0-12
Soft Cohesive soil (12-24 kPa)	12-24
Medium stiff cohesive soil (24-48 kPa)	24-36
Stiff cohesive soil (48-96 kPa)	36-45
Very stiff cohesive soil (96-192 kPa)	45-62

**Table 2.10 Identification of fine-grained soil fractions from manual tests
(NAVFAC DM 7.1, 1986)**

Soil Classification	Dry Strength (Crush in Hand)	Dilatancy Reaction (Wet Shake)	Time to Settle in Dispersion Test (Hydrometer)
Sandy Silt	None to very low	Rapid	30 sec to 60 min
Silt	Very low to low	Rapid	15 to 60 min
Clayey silt	Low to medium	Rapid to slow	15 min to several hours
Sandy clay	Low to high	Slow to none	30 sec to several hours
Silty clay	Medium to high	Slow to none	Several hours to days
Clay	High to very high	None	15 min to several hours
Organic silt	Low to medium	Slow	15 min to several hours
Organic clay	Medium to very high	None	Several hours to days

Table 2.11 Criteria for describing dry strength (Das, 2012)

Description	Criteria
None	The dry specimen ball crumbles into powder with the slightest handling pressure.
Low	The dry specimen crumbles into powder with some pressure from fingers.
Medium	The dry specimen breaks into pieces or crumbles with moderate finger pressure.
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface.
Very High	The dry specimen cannot be broken between the thumb and a hard surface.

Note: ML = Silt; CL = Low plasticity clay; MH = Medium plasticity Soil; CH = High plasticity clay.

Table 2. 12 Criteria for describing dilatancy of a soil sample (Das, 2012)

Description	Criteria
None	There is no visible change in the soil samples.
Slow	Water slowly appears and remains on the surface during shaking or water slowly disappears upon squeezing.
Rapid	Water quickly appears on the surface during shaking and quickly disappears upon squeezing.

Table 2.13 Criteria for describing soil toughness (Das, 2012).

Description	Criteria
Low	Only slight pressure is needed to roll the thread to the plastic limit. The
Medium	Moderate pressure is needed to roll the thread to near the plastic limit.
High	Substantial pressure is needed to roll the thread to near the plastic limit.

Table 2.14 Identification of inorganic fine-grained soils (Das, 2012)

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None or Low	Slow to Rapid	Low or thread cannot be formed
CL	Medium to High	None to Slow	Medium
MH	Low to Medium	None to Slow	Low to Medium
CH	High to Very High	None	High

2.1.4. Soft soil shear strength correlations

In 1911, Atterberg suggested the boundaries of consistency for agricultural basins to obtain an obvious view for the range of water contents of a soil in the plastic state (Casagrande, 1932). The bounds of soil consistency specifically liquid limit (w_L) and plastic limit (w_P), well known as Atterberg limits (Casagrande, 1932, 1958), were systematized by Casagrande (1932, 1958) and expanded for classification of fine-grained soils.

These limits are regulated using simple tests, which are mainly strength based. Attempts have been done from 1911 onwards to understand Atterberg limits and create improved methods of determining the equivalent limits. Research contribution continues on Atterberg limits (Sridharan and Prakash, 1998; Sridharan and Nagaraj, 1999; Sridharan et al., 2000; Prakash and Sridharan, 2006; and Nagaraj and Sridharan, 2010). Due to different restrictions of the rolling thread method of quantifying plastic limit, especially human beings errors, efforts have been made to get the same from cone method (Hansbo, 1957; Towner, 1973; Campbell, 1976, 1983; Wood and Wroth, 1978; Belviso et al., 1985; Sampson and Netterberg, 1985; Wasti and Bezirci, 1986; Rao, 1987; Harison, 1988; Feng, 2004; Al-Dahlaki and Al-Sharify, 2008; Rashid et al., 2008; Lee and Freeman, 2009; and Sivakumar et al., 2009).

Sridharan et al. (1999) have come up with a method to quantify plastic limit through the correlation developed between plasticity index and flow index. In most of the tries to develop the testing techniques to quantify liquid limit and plastic limit, researchers have attempted to express liquid limit and plastic limit as strength based water content, and hence, the testing methods to quantify them. Nevertheless, the way of defining Atterberg

limits differs from the fundamental physical meaning, which is the water holding capacity of the soil at those conditions of consistency. Lambe and Whitman (1979) correlated Atterberg limits for a soil to the amount of water attracted to the surface of the soil particles. It is well carried out by Sridharan and Venkatappa Rao (1979), Sridharan et al. (1986, 1988), Sridharan and Prakash (1999) that the mechanisms governing undrained shear strength and liquid limit for kaolinitic soils is unlike montmorillonitic soils. In fact, it cannot be presumed that the strength at the liquid limit content to be the same for all soils. This characteristic has been approved by the results stated by Kenney (1963). Undrained shear strength of remolded clays have a robust correlation with the liquid limit and the plastic limit provided the shear strength at these limits can be correlated with each other. Based on the fact that soil adopts a unique state at the liquid limit yielding unique shear strength and that this shear strength stands a definite relationship with that at the plastic limit (Sharma and Bora, 2003).

As early as 1939, Casagrande proposed an average shear strength of soil at the liquid limit as 2.65 kN/m^2 taking in consider a large spread of values depending on the apparatus used for determining the liquid limit. Norman (1958) stated that the shear strength at the liquid limit controlled by using an apparatus compliant to the British standard ranged from 0.8 to 1.6 kN/m^2 whereas using an apparatus of ASTM standards, the strength varied from 1.1 to 2.3 kN/m^2 . Skempton and Northey (1953) described the value of shear strength at the liquid limit of four soils with very different values of plasticity index as 0.7 kN/m^2 to 1.75 kN/m^2 . Youssef et al. (1965) found that the values of shear strength of clay at the liquid limit of a large number of soils (liquid limit varying from 32 to 190%) ranged from 2.4 to 1.3 kN/m^2 with a mean value of 1.7 kN/m^2 .

Based on Federico's results (1983), the shear strength at the liquid limit of soils, falls within limits of 1.7 and 2.8 kN/m². Other studies (Russell and Mickel, 1970; Wroth and Wood, 1978; Whyte, 1982; and Nagaraj et al., 2012) have indicated that shearing strength of all fine grained soils at the liquid limit falls within a limited range of about 1.7–2.0 kPa. According to Wroth and Wood (1978), a considerable part of the strength variation at the liquid limit obtained by using the Casagrande apparatus can be appointed to the fact that soil deformation is self-weight-induced. In the cone test the soil deformation is affected by the cone weight and is essentially independent of the soil weight and hence of its water content. Wroth and Wood (1978) have attempted to redefine plastic limit in terms of strength as that water content that provides a 100-fold increase in shear strength over that at the liquid limit. Based on this principal, efforts have been made to develop an instrumented cone penetrometer to quantify the plastic limit with a mean value of 1.7 kN/m² as the best estimate of undrained shear strength of a remolded soil at its liquid limit (Stone and Phan, 1995). Atterberg limits are very crucial method for examining the behavior of fine-grained soils, but correlations of the same with the undrained shear strength are in need to be inspected.

Extensive study of the variation of undrained shear strength of soft soil through previous literature has been collected as shown in Table 2.15 where different correlations to predict the undrained shear strength (S_u) of soft soil have been reported. Most of the studies used vane shear device to measure shear strength at high moisture content. Most of tested soils were onshore soil with the 0.2 kPa as lowest measure undrained shear strength. Similarly, different proposed correlations of shear strength versus moisture content of soft soil have been collected as denoted in Table 2.16. The shear strength has

been correlated to soil properties such as plastic limit (W_p), liquid limit (W_L), and moisture content. Most of tested soils were different types of onshore soils with moisture content lower than liquid limit having different ranges of shear strength. From Table 2.15 and Table 2.16, it is required to study the relationship between the undrained shear strength and moisture content for soft soil with high moisture content preciously.

2.1.5. Non-destructive methods

Geophysical methods (geoelectrical, ground penetrating radar and seismic refraction) have become increasingly practiced in engineering site characterization as being non-invasive, non-destructive, rapid and cost-effective method. Among these methods, geoelectrical survey is a very attractive tool for delineating subsurface properties without soil disturbance (Samouelian et al., 2005).

Electrical resistivity investigation methods test soil properties by measuring the current and voltage between electrodes. Electrical resistivity methods, which were developed in the 1900s, have been used for the investigation of geological structures, under-ground spaces such as cavities, underground water contamination, and salinity distribution of aquifer water (Lee et al., 2003; and Kaya and Fang, 1997). Electrical resistivity survey was first applied to oil/gas exploration and prospecting of conductive or bodies, later it found applications in various engineering fields such as mining, agriculture, environment, archeology, hydrogeology and geotechnics (Siddiqui and Osman, 2012).

Limited attempts have been made by researchers to explore the phenomenon of electrical resistivity in soils and its relationship with other soil properties; such as

thermal resistivity, salinity, ground water distributions using four probe methods (Abu-Hassanein et al., 1996; and Yoon and Park, 2001). Almost no research work has been carried out so far, which actually correlates electrical resistivity with soft soil strength soil.

2.2 Soft soil improvement

The world population growth of human is in increase day by day and the appropriate soil to sustain loading from buildings or structure are becoming limited. Due to the lack of land, the growth of the swampy areas, mountainsides, coastal areas and landfill areas become the substitute places for the people to live where soft clay deposits are extensively exist and exhibit poor strength and compressibility (Gallant et al., 2007). Consequently, soil stabilization has become one of the powerful solutions to treat the soil in such areas to maintain the required engineering properties and specification so that structures can be located safely without undergoing large settlements. Soil stabilization is described as a technique to enhance the engineering characteristics in order to improve the parameters such as shear strength, compressibility, density, hydraulic conductivity. The techniques of soil stabilization can be categorized into a number of groups such as vibration, surcharge load, structural reinforcement improvement by structural fill, admixtures, and grouting and other methods. There are many techniques that can be used for different reasons by enhancing some features of soil behavior and improve the strength and properties of soil (Edil, 2003).

In the Table 2.15, the variation of undrained shear strength at liquid limit water content as reported in the literature have been shown.

Table 2.15 Variation of undrained strength at liquid water content as reported in the literature

Reference	Range of Undrained Strength at Liquid Limit Water Content (kPa)	Range of Liquid Limit	Test	Remarks
Skempton and Northey (1952)	0.7-1.75	30-97	Vane shear Test	Shear strength is lower than 1 kPa evaluated. Experimental study. Onshore natural soil samples.
Norman (1958)	0.8-1.6 (B.S. Standards), 1.1-2.3 (ASTM Standards)	41-72	Miniature Vane Shear Apparatus	Shear strength of the clay soils were 25 to 50% higher in ASTM standards. Experimental study. Onshore natural soil samples.
Youseff et al.(1965)	1.3-2.7	32-190	Vane Shear Test	Shear strength more than 1 kPa evaluated. Experimental study. Onshore natural soil samples.
Skopek and Ter-Stepanian (1975)	1-3	17-382	Vane Shear Test	Shear strength started from 1 kPa. Experimental study. Onshore natural and artificial soil samples.
Wroth and Wood (1978)	Mean Value of 1.7	26-190	Vane Shear Test	Average shear strength was 1.7 kPa. Experimental study. Offshore field data was used.
Houlsby (1982, 1983)	2.75-5.24	-	-	Shear strength is higher than 1 kPa. Theoretical analysis. No soil data used.
Federico (1983)	1.7-2.8	36-159	Vane Shear Test	Shear strength is higher than 1 kPa. Experimental study. On shore natural soil.
Wasti and Bezirci (1986)	0.5-5.6 0.8-4.8	27-526 30-328	Vane shear Test	Shear strength is lower than 1 kPa. Experimental study. Onshore natural and artificial soil samples.
Locat and Demers (1988)	0.2-2.04	27.4-62.8	Viscometer	Shear strength is lower than 1 kPa. Experimental study. Onshore artificial soil samples
Sridharan and Prakash (1998)	0.66-1.35	29.8-100.8	Viscometer	Shear strength is lower than 1 kPa. Experimental study. Onshore natural and artificial soil samples.
Kayabali and Tufenkci (2010)	1.2-12	26.4-83.6	Vane Shear Test	Shear strength is higher than 1 kPa. Experimental study. Onshore natural soil samples.
Remarks	Varied from 0.2 to 5.6 kPa	Varied from 17% to 526 %	Mainly Vane shear Test	Mainly shear strength less 1 kPa. Mostly experimental studies. Rarely offshore soils were tested.

In the Table 2.16, different shear strength correlation has been summarized.

Table 2.16 Correlation between undrained strength with physical properties of soil

Reference	Type of Soil	Equation	Description	Remarks
Schofield and Wroth (1968)	CL natural soil	$S_u = 170e^{(-4.6I_L)}$	Laboratory testing for onshore soil.	Su relating I_L .
Whyte (1982)	CL natural soil	$C_u = 1.6e^{4.23(1-I_L)}$	Laboratory testing for onshore soil.	Cu relating I_L . Saturated remoulded clay. Shear strength are quantified based on 1.6 and 110 kPa at liquid and plastic limits, respectively.
Federico (1983)	CL and CH natural soils	$C_u = e^{5.25(1.161-w/w_L)}$	Laboratory testing for onshore soil.	Cu relating w/w_L . High range of moisture content. Using cone penetration to determine shear strength.
Leroueil et al. (1983)	CL and CH natural soils	$S_u = \frac{1}{(I_L - 0.21)^2}$	Laboratory testing for onshore soil.	Su relating I_L . Il between 0.5 to 2.5. Predict infinite strength at $I_L = 0.21$ and it cannot be extended beyond this value.
Locat and Demers (1988)	CH artificial soil	$C_u = (19.8/I_L)^{2.64}$	Laboratory onshore soil.	Cu relating I_L . $IL \leq 6$. For IL from 2 to 5, the shear strength was 90 to 5 Pa.
Bell (2002)	Mostly CL natural soils	$C_u = 3718(w_p)^{-1.18}$	Laboratory onshore soil.	Cu relating w_p . Low plasticity clay. Low unconfined shear strength.
Lee (2004)	CL and CH natural soils.	$C_u = 8.779e^{-2.3714(w/w_L)}$	Laboratory onshore soil.	Cu relating w/w_L . Remolded dredged material. Highly compressive soft soil.
Berilgen et al. (2007)	CL natural soils	$\ln(c_u) = 11.5 - 2.2 \ln(w)$	Laboratory onshore soil	Cu relating w
Edil and Benson (2009)	CL natural soils	$S_u = 144.9e^{(-1.72I_L)}$	Laboratory onshore soil	Su relating I_L . Different types of soils. Undrained shear strength was 35 kPa.
Edil and Benson (2009)	CL natural soils	$S_u = 191.4e^{(-0.03w_L)}$	Laboratory onshore soil	Su relating w_L . Different types of soils. Undrained shear strength was 35 kPa.
Remarks	Mostly CL soils	Undrained shear strength with moisture content	Laboratory onshore soil	Different types of soils. Mostly the moisture content is lower than liquid limit. High range of shear strength.

Note: C_u or S_u are reported in the equations are in kPa, w_L is liquid limit, w_p is plastic limit, I_L is liquidity index, I_p is plasticity index, I_C consistency index.

The important properties of ground treatment includes: improving the bearing capacity of the ground, reducing the potential for total and differential settlement, reducing the time during which the settlement take place, reducing potential for liquefaction in saturated fine sand or hydraulic fills, reducing the hydraulic conductivity of the ground, removing or excluding water from the ground. The traditional method of soil improvement is to substitute the soft soil by suitable imported fill materials. Yet, this practice is naturally very expensive due to the cost of excavation, dumping and the filling material and impossible to implement in ultra-soft soil.

Utilization of various improvement methods for soft soil particularly soft clay is used in a wide range. These methods were based on using lime, cement and flay ash stabilization as presented by previous studies carried out by Ali et al. (1992), Balasubramaniam et al. (1999), Muntohar and Hantoro (2000), Muntohar and Hashim (2002), and Muntohar (2004). Further information has been reported in order to improve the soft ground by using soil cement column method as reported by Hebib and Farrell (2003) and Porbaha and Bouassida (2004). In addition, there are many investigations which were carried out to enhance the load bearing capacity of soft soil by adopting the soil reinforcement technique as presented by Hirao et al. (1992) and El Sawwaf (2008). Chemical stabilization by cement or lime is a proven practice for improving the implementation (strength and stabilization) of soil (Ismail et al., 2002; Aiban, 1994; Huang and Airey, 1998; Basha et al., 2005; Kolas et al., 2005; Sherwood, 1993; Al-Rawas, 2002; Tremblay et al., 2002; Lima et al., 1996; and Thome, 1999). Nevertheless, these chemical additives usually lead to a high stiffness and brittle behavior (Wang et al., 2003; and Basha et al., 2005). Incorporating reinforcement within the soil is also an

efficient and consistent technique in order to improve the engineering properties of soil. In contrast with conventional geosynthetics (strips, geotextile, and geogrid), there are several advantages in using randomly distributed fiber as reinforcement. First, the discrete fibers are basically added and mixed randomly with soil, in much the same way as cement, lime, or other additives. Second, randomly distributed fibers control potential planes of softness that can develop parallel to oriented reinforcement. Therefore, it has developed a focus of interest in recent years. A number of triaxial tests, unconfined compression tests, CBR tests, direct shear tests on the subject have been conducted by several investigators in the last few decades (Yetimoglu and Salbas, 2003; Yetimoglu et al., 2005; Gray and Al-Refeai, 1986; Ranjan et al., 1996; Prabakar and Sridhar, 2002; Kaniraj and Gayathri, 2003, Li et al., 1995, Al-Refeai, 1991, Krishnaswamy and Isaac, 1994; and Ranjan et al., 1994). Park and Tan (2005) studied the influences of short fiber (60 mm) reinforcement on the performance of soil wall. Miller and Rifai (2004) denoted that fiber inclusion increased the crack reduction and hydraulic conductivity of compacted clay soil. The previous studies have shown that the addition of fiber-reinforcement produced significant improvement in the strength and decreased the stiffness of the soil. More importantly, fiber reinforced soil presents greater toughness and ductility and smaller loss of post-peak strength, as compared to soil alone. Thus, the discrete fiber can be used as a good earth reinforcement material, which causes notable modifications and improvement in the engineering properties of soil. Nevertheless, more work is necessary to comprehend the impact of fiber inclusion on the mechanical behavior of cemented and uncemented soils, mainly in the interfacial interactions between fiber surface and reinforced soil matrix.

Soil stabilization by admixture was advanced in Japan during 1970 and 1980. It uses rotating mixer shafts, paddles, or jets that penetrate into the ground while injecting and mixing Portland cement with some other stabilizing agent. These techniques contain deep cement mixing, soil mix walls, and deep mixed method. The treated soil has more strength, less compressibility and lower hydraulic conductivity than the original soil (Raison, 2004). The use of admixture such as lime, cement, oils and bitumen is one of oldest and most widespread method for improving soil. When mixed with soil, it forms a material called soil-cement. The objective of admixture is to provide artificial cementation, thus increasing the strength and reducing both compressibility and hydraulic conductivity.

The deep mixing method can be used in most soft soils. The mechanized development of mixing is by using a rotating mixing tool, drilling the tool into the soil. Next, the drilling rotation is overturned, extracting it and at the same time as the dry binder is injected and mixed into the soil. Within the rotating movement, the soil is mixed with the binder and an immediate reaction begins. The improved soil obtains the share of a column (Kazemian, 2009). The column so formed can have diameters varying from 0.5 to 1 m and the lengths up to 25 m. The columns can also be interconnected to provide cellular structure of in-situ wall or the entire mass can be stabilized. Dry mixing is extremely operative ground treatment system used to enhance the load performance of soft soils. By changing the proportion of lime, cement and admixtures, a range of strength improvements can be achieved. The highest improvements can be achieved in inorganic soils with low moisture content (Hashim and Islam, 2008).

The original technique known universally as the deep mixing method (DMM) was developed simultaneously in Sweden and Japan in the mid-1970 s. DMM is a ground treatment, improvement, and support method of global application and increasing popularity and value (Mitchell and Jardine, 2002). In comparison with other similar ground improvement methods, the Deep mixing Method (DMM) is the method particularly designed to treat the soft soils. DMM are divided into three systems specifically, SSM (Shallow Soil Mixing), DSM (Deep Soil Mixing) and JGS (Jet Grouting Systems) (Bruce, 2000).

Deep mixing method is an in-situ soil treatment technology where the soil is mixed with cementitious and/or other materials. The deep mixing method is often categorized into two methods: dry and wet method, based on the type of binder, the mechanism of bleeding in rotary or jet used, and the vertical extent over which blending is accomplished. The former operates the dry powdered binder whereas the latter utilizes the water-binder slurry. Certainly, there are some differences in the execution machines between dry and wet methods. Nevertheless, there is no considerable difference in the features of treated soils between them. The apparent difference in the design procedure and application comes from the purpose of improvement, which gives rise to the difference in the installation outlines and in the order of strength required (Bromes et al., 1999). Deep mixing method highlights on column type techniques using lime/cement. It is a soil improvement method, which is executed to improve the strength, deformation properties and hydraulic conductivity of the soil. It is depended on mixing binders, such as cement, lime, fly ash and other additives, with the soil by the use of rotating mixing tools in order to form columns of a hardening material since pozzolanic reactions

between the binder and the soil grains are created. The main benefit of these methods is the long-term increase in strength, especially for some of the binders used (Anagnostopoulos and Chatziangelou, 2008). Pozzolanic reaction can continue for months or even years after mixing, resulting in the increase in strength of cement stabilized soil with the increase in curing time (Hashim and Islam, 2008).

Normally, grouts that are frequently moving will turn into a gel less quickly, and the penetration from continuous injection will be greater than that from the same volume of grout used in batch injection. When gelling happens before pumping is stopped, the last injected grout typically moves to the outside of the grouted mass and both large and small openings are filled. Jet Grouting is appropriate to be used as the injection method for the deep mixing method (DMM). It uses a fluid jet (air, water and/or grout) to erode and mix the in-situ soft or loose soils with grout. It applies high velocity, 28 to 42 MPa backpressure and jet to hydraulically shear the soil and adding proper binder to form a column. The result is considerably increased shear strength and stiffness of the soil (Mitchell and Jardine, 2002). The first jet grouting was applied in England in the 1950 s; however, the actual development of jet grouting was in Japan during 1960 s and 1970 s. Jet grouting is the latest method compared with other methods. In the mid 1970 s, jet grouting was spread to Europe and has become popular worldwide. This technology was initially pointed at improving the effectiveness of water tightness, in chemical grouting, by eroding the untreated or partially treated soil, which was then removed to the surface for disposal being replaced with cement-based slurry for imperviousness (Moseley, 2000). Jet grouting is the production of hard, impervious column in the ground by the enlargement of a drill hole using rotating fluid jets to soften and mix grout with, or to

excavate and replace, soil (Raison, 2004). Jetting and grouting are transported out during controlled withdrawal and rotation of the drill string and the jetting head from the hole. There are several variations dependent on the nature and pressure of the jetting and grouting the in-situ soil may be mixed with the grout, partly mixed and partly removed or wholly replaced. In general, there are four jet grouting systems which are widely used and classified as Single phase (grout injection only), Dual phase (grout + air injection), Triple phase (water + air injection and followed by grout injection), Super Jet Grouting (air injection + drilling fluid by grout injection) (Mitchell and Jardine, 2002).

The grouting method is one of the ground improvement procedures suitable for the soft soil. Modern grouting started in the mining industries, concerned with the seepage and strength control in mines, tunnel and shaft, then was taken up by civil engineering. Many functions of grouting available depend on the intention and the condition of the site. It contains permeation grouting, compaction grouting, hydro fracture grouting, jet grouting, rock grouting, compensation grouting, cement grouting and fracture grouting. Due to the various functions of grouting, the differences between grout characteristic and differences between the soil type to be grouted need to be focused. Thus, the generalization about the grouting equipment and method are difficult to achieve (Shroff and Shah, 1999). A grout is also simply defined as a material used for grouting (Karol and M. Dekker, 1983).

Choosing the right method for deep soil stabilizing however, depends on a number of conditions like the type and alternative layers of soil, load size, the situation and type of project, among others (Mitchell and Jardine, 2002). Grouting generally is used to seal voids in the ground (fissures and porous structures) with the goal to increase resistance

against deformation, to provide cohesion, shear-strength, compressive strength and finally to decrease hydraulic conductivity or interconnected porosity in an aquifer (Moseley and Kirsch, 2004).

Chemical stabilization is the effective method to improve the soil properties by mixing additives to soils. Usually the additives are cement, lime, fly ash and bituminous material. These additives enhance the properties of soil. Generally, two major reactions for the chemical stabilization are cation exchange reaction and cementation (Mitchell, 1983). The common chemical agent for cementation process is Portland cement, lime, fly ash, sodium silicate polyacrylamides and bituminous emulsion.

Many of chemical grouts are of sodium silicate and a reagent process used in coarse granular soils uses calcium chloride as a reagent. Other reagents are organic ester, sodium aluminates and bicarbonates. The reagent and the proportion can be chosen to control the gel time, the initial viscosity and the order soil.

Chemical grouts are injected into voids as a solution, in contrast, to cementitious grout, which are suspension of particle in a fluid medium. The difference between chemical grout and cementitious grout is the chemical grout can be used to fill the finer voids of soil particles up to 10 to 15 μ m in diameter. In other word, it has better penetration ability than the cementitious grout.

Chemical grout can be categorized in single step and two step processes. In one-step process, all the components are premixed prior to injection, the system are designed that the reaction takes place in-situ. In the two-step process, the initial chemical is injected into soil mass then followed by the second chemical material to react with the first in-situ

and to stabilize the mass. There are several types of chemical grouts, each type of grout have different features and different functions. The most common, are sodium silicate, acrylate, lignin, urethane, and resin grouts (Shroff and Shah, 1999).

In order to select a grout type, several properties of grout should be concerned such as rheology, setting time, toxicity, strength of grout and grouted soil, stability or permanence of the grout and grouted soil and the penetrability and water tightness of the grouted soil (Rawlings et al., 2000).

Moreover, the spreading of grouts plays a significant role in the development of grouting technology. In the actual filed, the grouting method requires an extensive consideration on the grout hole equipment, distance between boreholes, length of injection passes, number of grouting phases, grouting pressure and pumping rate (Shroff and Shah, 1999).

2.3 Contamination

Drilling fluid has a vital role during the drilling operations. It supplies as medium of carrying the cuttings from the bottom hole to the surface. It provides pressure on the well walls and avoids the walls from collapsing and the formation fluid from entering the wells. Furthermore, it operates as a lubricant for the bit and the drilling string. Water based drilling muds are used in on shore and offshore drilling operations. For drilling operations under HPHT (high-pressure high-temperature) conditions, the industry prefers to use OBM (oil based mud), which has benefit over WBM (water based mud) since it can maintain its rheological properties at high range of temperatures. However, this is not always feasible due to logistic, environmental restrictions control, cost, cuttings and used

mud disposal difficulties, and safety that make the WBM preferable (Elward-Berry and Darby, 1997).

Bentonite clays in fresh water are effective for controlling the amount of fluid loss to the formation, by forming a “mud cake” along the walls of the wellbore. However, under saline conditions in the wellbore, the filtration control is lost due to flocculation of the clay particles. Furthermore, shale swelling within the formation may have negative consequences during the drilling operation (Sherwood, 1994). This problem is produced by the stress concentration around the wellbore and the interchange of water and ion into or out of shale formation as interaction with drilling fluid takes place. In addition, fluid infiltration in fractured shale and weak bedding planes can quicken these problems, as hydraulic support of the wellbore is lost and large blocks of fractured shale collapse into the hole (Mody and Hale, 1993). Moreover, squeezing salts is a major drilling problems in many areas of the world for over half a century. It is responsible for problems such as stuck pipe and casing failure during drilling and casing failure during production. Since 1960, approximately \$170 million (at 1992 drilling costs) has been spent redrilling wells with failed casing strings (Muecke and Kiji, 1993).

Generally, diffusion osmosis is unrecognized driving force, which is determined by the difference in concentrations of the solutes in the drilling fluid and shale pore fluid. Diffusion osmosis results in transfer of solutes and associated water from higher to lower concentration for each species. Interactions of water-based drilling and completion fluids with shale formations have been considered as a major factor in the cost of finding and producing oil and gas (Simpson and Dearing, 2000). Much progress has been made in

understanding the mechanisms responsible for the destabilization of shale and subsequent problems such as high torque, lost circulation and cementing failures (Chenevert, 1970).

Increasing demand for oil and gas with limited availability of near surface reservoirs around the world is resulting in deeper oil wells with productive reservoirs. One of the challenges related to the drilling deep and ultra-deep wells is to preserve desirable rheological properties of the drilling fluids during the drilling operations (Hassiba and Amani, 2013). Drilling fluids represent 15% to 18% of the total cost of petroleum well drilling (Khodja et al., 2010). The WBM contains mainly water and clay but their performance is directly evaluated by the stability of the mud system with the rheological and filtration properties (Beihoffer et al., 1988).

Salts can contaminate the drilling fluids from several sources. The drilling fluids can be contaminated when drilling through salt beds and the probability encountering such layer during drilling process is higher for deep wells. Soluble salts can contaminate the water-based mud by affecting the thickening time, gel strength, swelling and increasing the water loss caused primarily by the interaction between the dissolved salts and the clays in the mud (Jilani et al., 2002; and Njobuenwu and Wobo, 2007). Salts of mono and divalent cations have the ability to increase the gelation time of the drilling mud and this effect is a function of the charge/size ratio of the cation (Al-Muntasheri et al., 2007). The Gulf of Mexico is the most productive deepwater region in the world, currently providing some of the extreme challenges in scope and opportunity for the industry. Undiscovered, recoverable resources are appraised to be at least ~ 13 billion barrels of oil equivalent (Willson et al., 2003).

The U.S. gulf coast basin contains the largest amount of sediments of salt in the world (Barker et al., 1994). The ability of salt to collapse under temperature and pressure and its very low permeability and porosity makes it a very successful hydrocarbon trap generator. In the Gulf of Mexico, salt sheets could be concealing oil and gas bearing strata under as much as 60% of the Gulf of Mexico less than 5,000 ft of water (Barker et al., 1992). The complex salt tectonics and tremendous water and reservoir depths require very high growth costs, in addition of the needful of innovative technology to bring these fields on stream (Unger and Howard, 1986). A well lifetime of 10 to 25 years is essential to successful economic growth (where the cost of a single well can be from U.S. \$20 to 60 million). In addition, a considerable majority of the wells will potentially penetrate significant salt thickness, with 1000 to 6000 ft of salt (Ballew, 2001). The original bedded salt thickness has been estimated to have been 4,000 to 5,000 ft thick (Barker et al., 1994). An critical property of salt is that it does not increase in density with burial depth. When salt underlies higher-density deposits, an unstable situation exists. The combination of this instability with movable salt is thought to be the major reason for vertical salt movement (Halbouty, 1979). Since salt does not increase in density with depth, a limited depth exists where overburden density of condensing sediments will equal salt density. When this happens, salt can start lateral flow, which creates salt overhangs and sheets. Salt found along the U. S. gulf coast is usually very pure, often as high as 97% pure mineral halite. The pure salt mineral has a density of 2.17 g/cm^3 ; however, in-situ salt density usually averages $= 2.10 \text{ g/cm}^3$ (Hackney, 1985). Typically in sand/shale formations, the in-situ stress in the vertical direction is equal to the overburden stress and the horizontal stress is somewhat less than the vertical stress. For a salt

formation, the in-situ stress is generally assumed to be equal in all directions and is equal to the weight of the overburden stress. If the hydrostatic pressure of the mud used to drill a well is less than the salt stress, salt will creep into the wellbore. The closure rate of salt increases with increasing temperature and increasing differential pressure between the salt stress and the mud-weight hydrostatic pressure. At shallow depths, temperature and differential pressures are minor and the creep rate will be slight. At greater depths, higher temperatures and higher differential pressures will cause a much increased creep rate. For most cases, even a slight amount of salt creep can cause a reduced wellbore radius or an undergauge wellbore, which leads to stuck pipe and casing collapse loading (Barker et al., 1992).

The stress distribution (Yew et al., 1990), petro-physical (permeability, saturation, porosity), chemical (membrane efficiency, water activity), and mechanical (strength, stiffness, Poisson's ratio) (Chenevert, 1970, and Hale et al., 1992) properties near the wellbore formation may be changed by this movement. It is well-recognized that swelling and pore pressure increase, while strength and stiffness of the near wellbore formation decrease with the influx of water, which may cause wellbore instability problems (Chenevert, 1970, Hale et al., 1992, Van et al., 1995, and Mody and Hale, 1993). On the other hand, it is commonly believed that the dehydration of shale is valuable in terms of wellbore stability because this back flow lowers the near wellbore pore pressure and thereby "strengthens" the shale (Mondshine and Kercheville, 1966; Chenevert 1970; and Salisbury and Deem, 1990). Even though the dehydration is beneficial to recover wellbore stability when drilling soft, high water content (Mondshine and Kercheville, 1966; Chenevert and Strassner; 1975), it is believed to be harmful to wellbore stability in

naturally fractured formation. The over-dehydration may cause the shale to shrink and grow tensile failure, especially in hard, naturally fractured shale formations (Horsrud et al., 1998). This effect has been documented in many areas of the world and wellbore destabilization has been observed in several wells at Gulf of Mexico (Rojas, 2006). The drilling fluid design with regard to salinity is a crucial part of a “Stressed Shale Drilling Strategy.”

Hassiba and Amani (2013) have studied the impact of up to 7% of NaCl and KCl on the water based drilling mud mix. Their study showed that the NaCl contaminated samples had higher shear stress-shear rate curves than water based mud while KCl contaminated samples had lower shear stress-shear rate curves than water based mud. It was found that presence of salt (sodium chloride, NaCl) or contamination increased the filtration of the water based drilling fluid by 30%. Presence of sodium chloride can be detected by measuring resistivity due to influence of salt (sodium chloride, NaCl) on the increasing the conductivity. It has been shown that with the contaminating 0.5% sodium chloride the resistivity decreased by 86% (Basirat et al., 2013). Salt contamination reduced the plastic viscosity and electrical resistivity of 6% (w/w) bentonite mud. Addition of 0.1% salt decreased the plastic viscosity and electrical resistivity by about 10% and 18% respectively (Ali et al., 2013).

2.4 Filter cake

Drilling fluids are a mixture of solids, liquids, and chemicals, and the liquid being the continuous phase; they are important for the oil, gas and geothermal drilling industry because they perform many functions like transporting rock cuttings to surface, lubricating the drill bit, applying hydrostatic pressure in the well bore to ensure well

safety and minimizing fluid loss across permeable formations by forming a filter cake on the walls of the well bore (Skalli et al., 2006). To stabilize the wellbore, the drilling fluid forms a filter cake, which stabilizes the formation face. Filter cake builds up over the face of the porous medium during the entire drilling operation and the filtrate is lost into the formation (Civan, 1993, 1998). When the slurry contains particles of different sizes, the larger particles of the slurry form the skeleton of the filter cake and the smallest particles can migrate and deposit within the porous cake formed by the larger particles. During drilling operations, the liquid phase of the drilling fluid may flow into the formation because of pressure and temperature differential and formation permeability resulting in fluid loss. The fluid loss rate is also governed by formation of the mud cake and its properties. In the case of permeable formations, the filtrate loss is governed by the mud-cake permeability. The cake filtration analysis began with the classical work by Ruth et al. (1933) more than 80 years ago. Based upon a systematic study of fluid loss and mud cake permeability, Williams and Cannon (1938) concluded that the filtration rate of drilling fluid was determined primarily by the amount and the nature of solids present and the addition of weighting materials increased the fluid loss. Other factors that influenced the filtration rate included the degree of dispersion of solids, distribution of particle sizes, degree of hydration of clays, and presence of dispersing or agglomerating agents (Byck, 1940; and Rogers, 1963). When the drilling mud contains particles of different sizes, the larger particles of the drilling mud form the skeleton of the filter cake and the smallest particles can migrate and deposit within the porous cake formed by the larger particles. In addition to the deposition of drilling mud particles on the formation around the wellbore, the small particles in the mud invade the formation causing internal formation damage,

ranging from less than an inch to a maximum of about 1 ft (Economides et al., 1994). Simultaneously, the cake may undergo a consolidation process under high pressure and temperature as the fluid flows through the cake (Tien et al., 1997). In oil or gas drilling with a drilling fluid, mud cake is formed on the borehole wall when pressure within the wellbore is higher than the pore pressure in the rock. The filtrate enters the formation and leaves behind the deposit of solids that form the filter cake (Jiao and Sharma, 1992; Parry, 2006). The drill string can get stuck if it becomes embedded in the cake as rather high net forces acting on the drill string will push it against the wall. A further problem for mud-cake analysis in the wellbore is the sealing of different sections of the well by packers, the interaction of these packers with the cake, and the resulting sealing efficiency. At early stages of filtration, both large and small particles deposit on the cake surface; because the drag force driving the particles to the cake surface is high, then only smaller and smaller particles are deposited (Jiao and Sharma, 1994). The cake growth rate gradually decreases until an equilibrium filtration rate is attained at which no particles small enough to be deposited are available in the suspension. This mechanism of cake growth gives rise to a heterogeneous cake with both large and small particles at the internal and only small particles at the external portion of the cake.

The filtration properties are an important characteristic of all drilling fluid. The filtrate invasion into the formation can substantially reduce the permeability of the near wellbore region by a number of different mechanisms: particle plugging, clay swelling, and water blocking. In addition, the nature and the thickness of the filter cake deposited on the formation face will increase the potential for differential pressure sticking.

The reduction in local permeability of the filter cake results from two factors: cake consolidation and cake clogging (Tien et al., 1997). Cake consolidation arises from the compressive stress within the cake while cake clogging is caused by the retention of fine particles. The amount of the fines involved may be small but its effect on permeability can be substantial. A relatively simple approximate model has been developed by Tiller (2002) to describe the behavior of compactible cakes deposited under constant applied pressure. Zinati et al. (2009) developed simple model that can predict the cake thickness and velocity profiles in radial geometry for a suspension containing mono-sized particles. They showed that simplifying assumptions may lead to errors in predicting the cake profiles. The velocity and permeability of the filter cake can be evaluated as

$$u(r) = -\frac{1}{4\mu(T)} \frac{dP_{inj}}{dz} (R-h)^2 \left[1 - \left(\frac{r}{R-h} \right)^2 \right] \text{ and} \quad (2.1)$$

$$\bar{u}_{cf} = -\frac{(R-h)^2}{8k_c} \frac{\partial \psi(z)}{\partial z}, \quad (2.2)$$

where $u_{cf}(r)$ = velocity, $\bar{u}_{cf}$ =average cross flow velocity, $\mu(T)$ = viscosity, dP_{inj} =change in the pressure, dz = change in the depth, R = borehole radius plus cake thickness, h = cake thickness, r = borehole radius, k_c = cake permeability, and

$$k_c = \text{cake permeability}, \psi(z) = \frac{k_c}{\mu} (P_{inj}(z) - P_R), \quad (2.3)$$

$P_{inj}(z)$ = pressure inside the bore hole, P_R = pressure at affected zone (R).

One approach based on fundamental filtration theory (Tiller, 1990, 2002) assumed that there is no effect of sedimentation during cake formation. Li et al. (2005) showed simplified filter cake permeability test method based on cake filtration followed by flow through already formed cake. Martinez et al. (2000) developed an alternative method for

determining permeability of the filter cake at the well site where accuracy is not important. Osisanya and Griffith (1997) developed an equation to determine filter cake permeability based on filtrate volume, shear stress, plastic viscosity, and yield point of the fluid as shown in the flowing equation

$$Q = \frac{kh(P_c - P_w)}{\mu \ln(r_c/r_w)}, \quad (2.4)$$

where Q = flow rate $\left(\frac{cc}{sec}\right)$, k = permeability (darcy),

h = height of the core (cm), P_c = pressure at r_c (atm),

P_w = pressure at r_w (atm), filtrate viscosity (cp),

r_c = radius of the core plus the thickness of the filter cake (cm), and

r_w = radius of the core (cm).

It is generally known in the petroleum industry that drilling muds have a more complex flow behavior than Newtonian fluids, yet it is still common practice to express the flow properties of muds in terms of a single viscosity. The drilling muds characterized as plastic materials, which obey the laws of plastic flow, or viscous materials where the viscosity change with the shear rate. When there is loss of water from the drilling mud filter cake are formed. Based on the rate of fluid loss the formation of filter cake will be affected and the permeability will vary with time.

The filter cake in the borehole will be subjected to varying shear and normal stresses based on the location of cake formation. Filter cake builds up over the face of the porous medium during the entire drilling operation that varies from several hours to days and the filtrate is lost into the formation. Currently used fluid loss model (API model) assumes

that there are no changes in the permeability and the solid fraction in the filter cake during the formation of the cake and hence the fluid loss is directly proportional to the square-root of time with no limit for the maximum fluid loss (Charles and Xie, 1997; Sherwood and Meeten, 1997; and Vipulanandan et al., 2014). In reality, there should be a limit on the fluid loss based on the total volume and type of drilling mud used for testing and one such time dependent relationship will be hyperbolic (Vipulanandan and Mohammed, 2014).

Formation damage, a common problem related with field operations, is often a main factor in reducing the productivity of a well in a petroleum reservoir (Liu and Civan, 1995). Numerous laboratory and field studies indicate that formation damage occurs during many phases of reservoir development, drilling, completion, work-over, production, stimulation, water-flooding or improved oil recovery (Almon and Davies, 1981). It has been recognized that the major cause of damage is the transport of fine particles in porous media.

The permeability reduction with time of the filter cake results from cake consolidation and cake clogging (Tien et al., 1997; and Moghadasi et al., 2004). Cake consolidation arises from the compressive stress and expulsion of fluid within the cake while cake clogging is caused by the deposition of finer particles. Filter cake porosity (n) is considered as one of the critical cake parameters to be quantified (Zinati et al., 2009). The porosity of the filter cake can be represented as

$$n = \frac{w}{w + \frac{1}{G_s}}, \quad (2.5)$$

where w is the moisture content of the filter cake and G_s is the specific gravity of the solid particles. Khatib (1994) studied the effect of applied pressure, solids type, and oil presence on the porosity and permeability of thin cakes using compression-permeability cell. The solids investigated were iron sulfide, iron hydroxide, calcium sulfate, calcium carbonate and produced silt and clay. Based on the study, a correlation between permeability (K_c) and porosity (n) of silt/clay filter cake was developed and the relationship is as

$$K_c = 0.491 * (1 - n)^{-1.97}, \quad (2.6)$$

where K_c is the permeability (darcy) of the mud cake.

Past studies on the formation of filter-cake were focused on the model tests in the laboratory. Cheng (2001) conducted laboratory model tests for pure bentonite suspensions in medium coarse sand, and the results showed that the density and viscosity of suspensions had a great impact on the filter cake formation. Fritz (2007) developed an apparatus for measuring the support pressure of slurry and conducted the support pressure tests to evaluate the best combination of additives such as polymer, sand and vermiculite of size 0.7 mm to 4 mm. Han and Zhu (2008) evaluated the factors that control the filter cake formation based on some tests conducted using a modified permeameter.

In addition, solid–liquid separation is commonly used in various industrial fields for the treatment of wastewater and the removal of suspending particles or indissolvable toxicants from liquid phase. Filtration separation has also found applications in pharmaceuticals, biotechniques, smelters and bioreactors. The migration and enrichment of fine particles form arched structure with dense fabrics in the vicinity of filter medium. The permeability of the filter medium will be reduced dramatically due to the formed

filter cake, demonstrating negative effect on the separation efficiencies and therefore requiring careful consideration in the industrial designs and operations (Li et al., 2009).

Moreover, slurry dewatering is one of the filtration processes at constant pressure. The conventional chemical engineering description of this process distinguishes two stages. During the initial filtration stage, unfiltered slurry remains above the cake. This is followed by expression, during which the filtercake is compacted. These are convenient practical terms, but it should nevertheless be possible to describe the entire dewatering process by a single model if sufficient information is available concerning the properties of the filter cake (Sherwood, 1997).

On the other hand, slurry type shield tunneling is widely used in tunnel construction in saturated, granular and highly permeable ground. One of the key issues for its successful operation is how to form a filter-cake to protect the working face (Anagnostou and Kovari, 1994; Koyama et al., 2009; and Li et al., 2009). However, there are too many influencing factors of it, such as the permeability of the ground, the setting of mud pressure, and the properties of the slurry (density, viscosity, stability and grain size distribution) (Zhang et al., 2004).

In addition, soil-bentonite (SB) vertical cutoff walls are commonly used in geotechnical engineering applications to control subsurface groundwater flow beneath and seepage through hydraulic containment structures (e.g., embankments and dams) and in geoenvironmental engineering applications to minimize the extent of subsurface pollution resulting from the migration of contaminated groundwater (Xanthakos, 1979; D'Appolonia, 1980; U.S. EPA, 1984, 1992; Ressi and Cavalli, 1985; Evans, 1993; and

LaGrega et al., 2001). These engineered structures are constructed using the slurry trench method, whereby a trench is excavated typically in highly permeable soils (e.g., sands and gravels) and backfilled with bentonite slurry consisting of a mixture of water and from 4–6% by dry weight of conventional (untreated) sodium bentonite (CSB) to maintain trench stability. The hydraulic conductivity, compressibility, and strength of the backfill are important considerations for the proper performance of SB cutoff walls (D'Appolonia, 1980). The hydraulic conductivity of the backfill depends primarily on the soil gradation and the amount of bentonite added to the backfill.

Maintaining a stable wellbore is one of the major challenges while drilling a well. Studies indicate that unscheduled events relating to wellbore instability is approximately more than 10% of well costs, with estimate over \$1 billion in annual cost to the industry (Jahanbakhshi and Keshavarzi, 2012).

Wellbore instability is caused by a radical change in both the mechanical stress and the chemical and physical environments when a hole is drilling and exposing the formation to drilling mud. Wellbore instability is observed most often as sloughing and caving shale, resulting in hole enlargement, bridges and fill, tight hole in the formations which they have time depend behaviors. The most common consequences are stuck pipe, sidetracks, logging and interpretation difficulties, and sidewall core recovery difficulties, difficulty running casing, poor cement jobs, and lost circulation. All contribute to increased costs, the possibility of losing part of the hole or the entire well, or reduced production (Al-Ajmi and Zimmerman, 2006).

Information in the literature on fluid loss studies with filter cake properties are summarized in Table 2.17. The Bentonite content in the drilling muds varied from 2% to 5% (W/W). The testing time for the fluid loss varied from 20 to 600 minutes while current API fluid loss method recommends a testing time of 30 minutes although drilling operations can vary in time from hours to days and weeks based on the project (Khalaf, 1987; and Hamida et al., 2010). As summarized in Table 2.17, fluid loss studies have been performed with varying pressures and temperatures.

One of the HPHT test results reported in the literature was conducted using three different types of drilling muds at a pressure of 300 psi and temperature of 225°F (Elkatatny et al., 2012a). All the drilling muds composition were similar except the amount of calcium carbonates was 28 gm in fluid A and 40 gm in B while fluid C was similar to A but it also had 50 gm of manganese tetra oxide in addition. The material properties were as follows: drilling mud density varied from 9.2 to 10.3 ppg, plastic viscosity varied from 12 to 13 cP, yield point varied from 7 to 11 lb/100 ft². In this study, CT scan was used to measure the filter cake formation with time. The filter loss after 30 min ranged from 8 to 8.4 cm³ while the spurt volume varied from 2.8 to 4.3 cm³ (Table 2.17). Filter cake thickness and permeability were characterized based on several equations in the literature and they varied from 0.02 to 170 µd. The CT scan showed two layers of filter cake one close to drilling fluid with a thickness varying from 0.08 to 0.1 in, and the other close to the surface of the disk with a thickness varying from 0.05 to 0.07 in. Another study was done to characterize the filter cake properties at different time period using CT scan during the HPHT test (300 psi, 250°F) (Elkatatny et al., 2012b). Test time interval was 5 min up to 30 min and the porosity of the layer close to the

drilling fluid ranged from 50 to 0% volume while the porosity of the layer close to rock surface varied from 55 to 10% volume. Total cake thickness increased from 0.06 to 0.13 in and the permeability of the layers closer to drilling mud and closer to rock surface decreased from 1384 to 0 μd and 2148 to 41 μd respectively.

Filtration process may occur under static and/or dynamic conditions. In static filtration process, the particles are continuously deposited to form thicker filter cakes until the space available is full of the filter cake. Schematic filter cake formation process is shown in Figure 2.7. During the filtration process, number changes can occur within the filter cake and affect the fluid loss. The modeling of this phenomenon can be idealized as shown in Figure 2.9. Cake filtration is widely used in chemical and processing industries for solid/liquid separation. In the cake filtration, the suspension to be treated is forced, under pressure, through a septum which allows the passage of the suspending liquid but retains the suspended particles, leading to the formation of a filter cake at the upstream side of the septum. According to the conventional cake filtration theory, the resistance to liquid flow. According to the conventional cake filtration theory, the resistance to liquid flow offered by a cake can be characterized by its specific cake resistance, which is inversely proportional to the product of the permeability (k), and porosity (n_s) of the cake (Zhao et al., 2003). The cake permeability is directly proportional to the void ratio which is considered one of the most critical cake properties to be quantified (Sherwood, 1997).

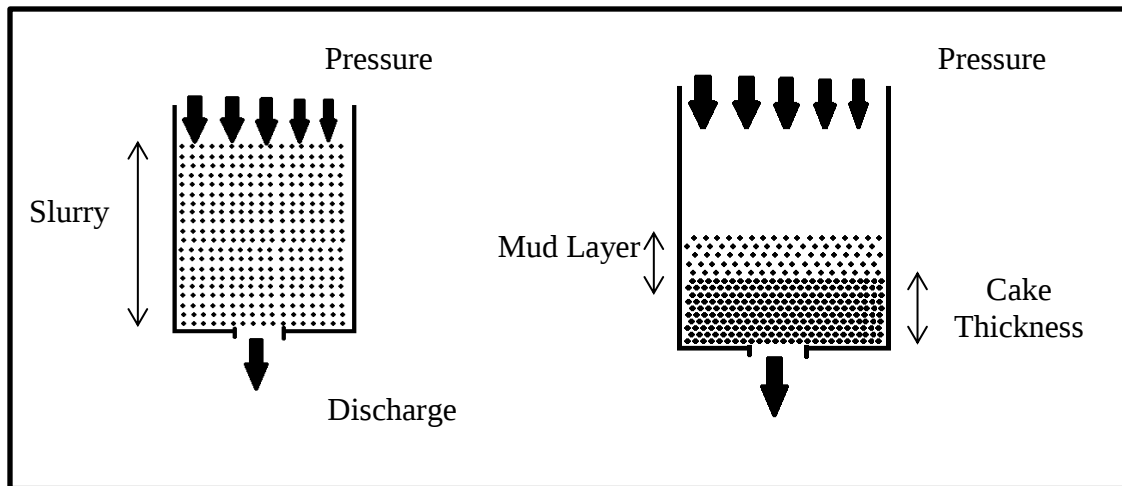


Figure 2.8 Schematic filter cake formation.

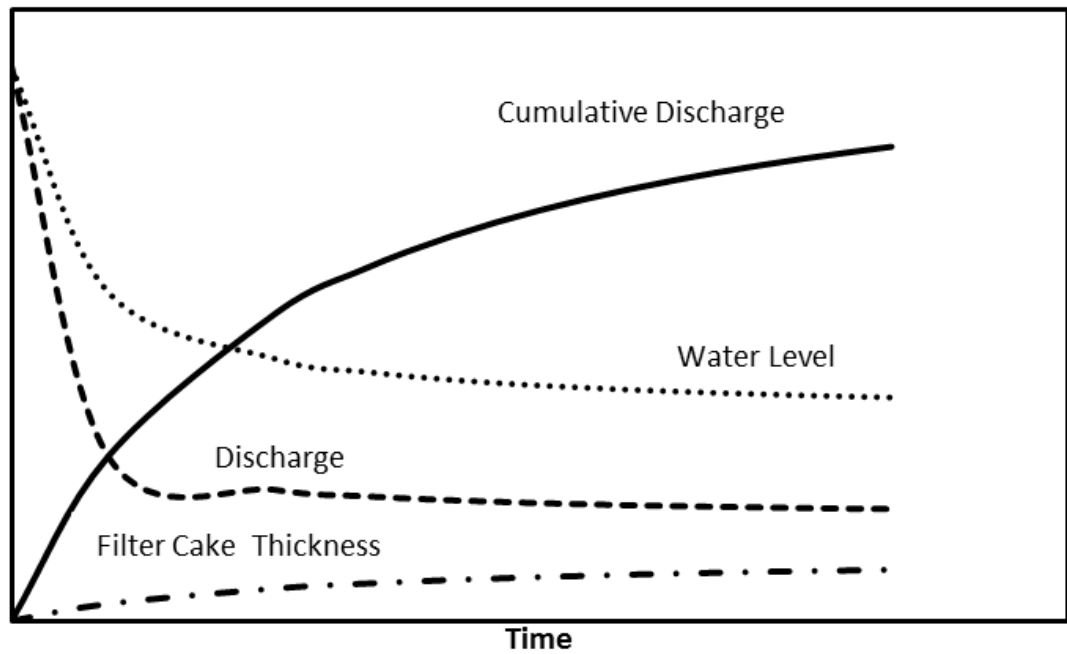


Figure 2.9 Idealization of Filter Cake Formation.

Table 2.17 Summary of filter cake properties reported in the literature.

Reference	Drilling Mud	Test and Time	Total Fluid Loss (cm ³)	Spurt Volume (cm ³)	Cake Thickness	Cake Permeability	Remarks
Jiao and Sharma (1994)	4% Bentonite	Static Filtration (P=1000 psi, T=25 C); 600 min	15-21	Zero	0.75- 2.3 mm (0.029-0.09 inch)	63.1-581.5 d	Testing time more than 30 min Cake thickness less than 5 mm High permeability
Osisanya and Griffith (1997)	2% Bentonite	Dynamic Filtration; 120 min	Not reported	Not reported	1-4 mm (0.039-0.157inch)	(0.55-0.1)*10 ⁻⁵ md	Testing time more than 30 min Cake thickness less than 5 mm Very low permeability
Bai and Tien (2005)	Kaolin	Static Filtration (P=15 psi, T=25 C); 20 min	Not reported	Zero	0.009-0.012mm (3.54-4.724)*10 ⁻⁴ inch	3.261 md	Testing time less than 30 min Cake thickness less than 5 mm Low permeability
Li et al. (2005)	Rice Hull Ash	Static Filtration (P=100 psi, T=25 C); 20 min	Not reported	Zero	Not reported	0.3-0.32 d	Testing time less than 30 min Cake thickness not reported High permeability
Fan et al. (2010)	Clay	Static Filtration (P=300 kPa, T=25 C) 15 min	50-300	Zero	0.2-43 mm (0.008-1.69) inch	0.3-100 md	Testing time less than 30 min Cake thickness has wide range High Permeability
Fritz (2006)	5% Bentonite	Static Filtration (P=40 psi, T=25 C); 72 min	250	Zero	Not Reported	Not Reported	Testing time more than 30 min Cake thickness not reported Permeability not reported
Soo et al. (2006)	2% CaCO ₃ 5% Kaolin	Static Filtration (P=15-115 kPa, T=25 C); 281 min	2	Zero	10-50 mm 0.393-1.96 inch	2.01-49.5 md	Testing time more than 30 min Cake thickness more than 5 mm High permeability
Elkatatny et al. (2012)	5% Bentonite	HPHT, CT scan (225F, 300 psi); 30 min	8-8.4	2.8-4.3	0.322-1.143mm 0.012-0.45 inch	0.023-170μd	Testing time is 30 min Cake thickness less than 5 mm Very low permeability
Elkatatny et al. (2013)	5% Bentonite	HPHT, CT scan (225F, 300 psi); 30 min	8.5	3	3.302 mm 1.3 inch	122-413μd	Testing time is 30 min Cake thickness less than 5 mm Very low permeability
Remarks:	Solid content varied from 2% to 5 %	Testing time varied from 20 to 600 minutes	Varied from 2 to 250 cm ³	Only in HPHT. Varied from 2.8-4.3 cm ³	Varied from 0.009 to 50 mm	Varied from 0.55*10 ⁻⁵ to 581.5*10 ⁻³ md	Mainly Bentonite drilling mud has been used. Filtration can last more than 30 minutes Cake thickness generally is less than 5 mm. Cake porosity is difficult to be quantified

P = Pressure; T = Temperature, 1psi = 6.894 kPa, F=32+C*9/5, 1d = 8.58 *10⁻⁴ cm/s

2.5 In-situ testing

Over the past two decades, offshore oil and gas facilities have been gradually extended from shallow-water fixed production systems to deep-water floating production systems (Randolph and White, 2008). Hydrocarbons demands need to be transported carefully within individual developments from wells to production facilities, between adjacent fields and also to shore for treatment and processing. Unlike in shallow water, deep-water pipelines are generally rested directly on the seabed with no additional stabilization measures. Precious assessment of pipeline embedment or sea bed soft soil properties is a critical aspect of the design of deep-water pipelines in respect of on-bottom stability, lateral resistance to thermally induced buckles and axial resistance (Bruton et al., 2006). Additionally, proper design and successful construction of any structure require an accurate determination of the engineering properties of the soils at the site especially near surface seafloor soils such as designing and installation of mat foundations, seafloor cable system and or even in coastal areas where stability of most natural and engineered structures such as building, roads, tunnels, slopes and bridges depend on vital aspect of geotechnical engineering (Cosenza et al., 2006; Nauroy et al., 1998; and Puech et al., 2000). For instance, dredged nearshore materials exhibit properties such as high water contents, low dry densities, and low shear strengths (Bartos, 1977) where the shear strength of most clayey soils is less than 2 psf (10 Pa) with in situ bulk densities of 72 to 84 pcf (1.15 to 1.35 kg/L) (DeMeyer and Mahlerbe, 1987). Another reason for determining the properties of the offshore soft soil is to quantify the strength during nearshore or contaminated dredged material placement operations. For example, when placing a sand cap over contaminated dredged materials, the engineering

behavior of the dredged material must be predicted or known in order to achieve successful cap placement. If the soft dredged material has insufficient shear strength to resist the imposed cap stress, failure will occur and the purpose of the cap will be defeated (Myre et al., 2000). Furthermore, soft soil existence is widespread and playing a crucial role in landslide, erosion, creep, soil suction and earth flow that even affects the stability of slope equilibrium conditions (de Riso and Nota d'Elogio, 1973; Guadagno, 1991; and Gao and Mamoru, 2014).

Due to a lack of land in coastal cities, renovation works are being carried out to increase the land where areas with favorable foundation for reclamation are becoming scarce. As such, reclamation works have to be carried out on unfavorable foundation such as waste pond, slurry pond, and recently formed estuary deposits (Bo et al., 2010). Because of the difficulty of obtaining undisturbed samples in coastal areas or deep water, there has been a movement to rely more on situ testing. However, special geotechnical problems related to geohazard evaluations have also motivated developments within the field of in situ testing (Lunne, 2001).

The conventional method of obtaining soil engineering parameters is laboratory investigations performed on soil samples acquired from site/field through borehole sampling. However, bore hole sampling in soft soil is almost impossible due to the low strength of the soil and even if it is possible then it would be very costly and time-consuming option (Pozdnyakova and Pozdnyakova, 2002). Whereas, field tests such as cone penetration test is the most widely used tool to characterize the near surface strength of ocean bottom soils. However, cone penetration test have limited capability when

accurate determination of the shear strength of soft sediments is desired (Randolph, 2004; Randolph et. al., 2007).

In the seabed testing and within the development of lighter rigs, there are a number of companies around the world that can perform seabed testing mainly for pipeline and cable route surveys where penetration of 3 m to 5 m is required (Hawkins and Marcus, 1998). Several methods such as Vane Shear Test (VST), Cone Penetration Test (CPT) and T-Bar method are used to the downhole testing. The vane shear test has been used widely in offshore site investigations especially in Gulf of Mexico (Geise et al., 1988; Johnson et al., 1988; and Young et al., 1988) while T-bar can be considered as a modified cone penetration device and both were used for soft soil characterization (Teh and Houlsby, 1991; and Lu et al., 2004). These current methods are not able to quantify the seabed ultra-soft soil properties precisely due to the size and weight compared to the ultra-low strength of the upper layer of soft soil.

2.5.1 Cone penetration test (CPT)

The cone penetration test (CPT) was created by a Dutch engineer in the early 1930 under the name of “Dutch Sounding Test”. Formerly, because it was operated manually, this device was only used to discover the thickness and the consistency of very soft layers at shallow depth. Later on, the cone penetration test has become one of the highly common and prevalent methods for in-situ subsurface soil investigation because it is dependable, repeatable, fast and cost effective. The standard electronic cone penetrometer contains two main components, namely, a cone tip part and a friction sleeve part. The cone tip has 60 degree apex angle with the projected base area (A_c) of 10 cm². The

friction sleeve has the length of 13 cm and 150 cm^2 of surface area (A_s). Figure 2.10 shows the main components and dimensions of the CPT (Shin, 2005).

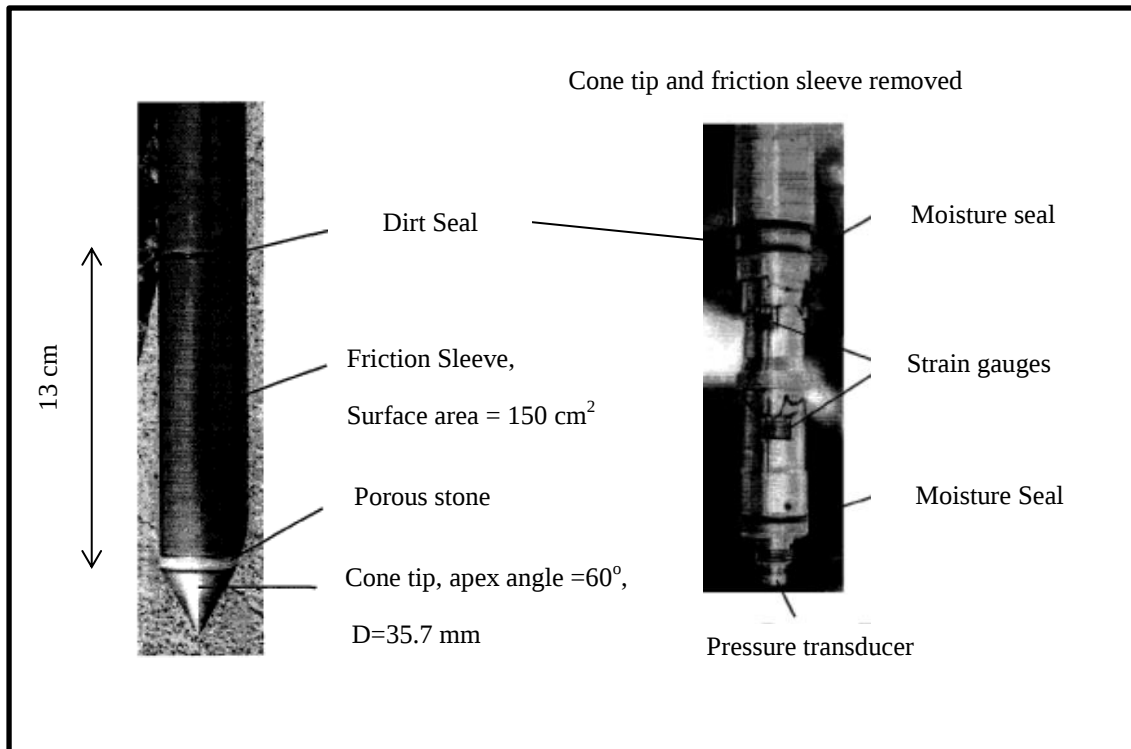


Figure 2.10 The cone penetrometer (Shin 2005).

A hydraulic actuator and hollow steel pushing rods are used to push the cone penetrometer. A multi-channel electric cable runs through the pushing rods to provide signal conditioning for the cone penetrometer and to transfer the CPT measurements to a data acquisition system at the ground surface.

While the cone penetrometer is being driven into the ground at a speed of 2 cm per second, the resistance forces on the cone tip (Q_c) and the friction sleeve (F_s) are recorded using strain gauges. Readings are taken at a constant interval, typically every 2 or 5 cm of penetration. The strain gauge at the bottom used to measure Q_c while the top strain gauge

measured the total force, $Q_c + F_s$. As a result, the sleeve resistance (F_s) can be calculated by subtracting Q_c from the total force.

The excess pore water pressure due to cone penetration can be recorded using a pressure transducer and a porous ring placed immediately above the cone tip (Wissa et al., 1975). Generally, coarse grained soils such as sands do not produce excess pore water pressure during penetration due to their high permeability. However, low permeability soil such as clay induces high excess pore water pressure because the soil is undrained. Also, the rate of the excess pore water pressure dissipation can be measured. The rate of dissipation presents important information about the coefficient of consolidation which is a function of the compressibility and hydraulic conductivity of the soil. Measurements of excess pore water pressure help to improve and correct the analysis of CPT tip resistance and friction sleeve resistance and specify general soil properties such as the coefficient of hydraulic conductivity (Campanella et al., 1982; and Robertson et al., 1986).

Tip and sleeve resistance in CPT can be calculated as

$$\text{Tip Resistance: } q_c = \frac{Q_c}{A_c} \text{ and} \quad (2.7)$$

$$\text{Sleeve Resistance: } f_s = \frac{F_s}{A_s}, \quad (2.8)$$

where $A_c = 10 \text{ cm}^2$ and $A_s = 150 \text{ cm}^2$ for a conventional electronic cone penetrometer.

Pore water pressure acts on opposite surfaces of the cone tip and thus, q_c has to be corrected for unequal end areas as shown in Figure 2.11. The corrected tip resistance is calculated by (Campanella et al., 1982; and Jamiolkowski et al., 1985)

$$\text{Corrected Tip Resistance: } q_t = q_c + (1 - a) \cdot u_2, \quad (2.9)$$

where a is the unequal end area correction factor. Many cones have values of net area ratio ranging from 0.90 to 0.60. u_2 is the pore water pressure measured using “Type 2”

piezo cones in which the porous ring is located between the friction sleeve and the cone tip as shown in Figure 2.11.

The sleeve friction may be divided by the corrected tip yielding a dimensionless Friction Ratio (FR)

$$FR (\%) = \frac{f_s}{q_t} * 100. \quad (2.10)$$

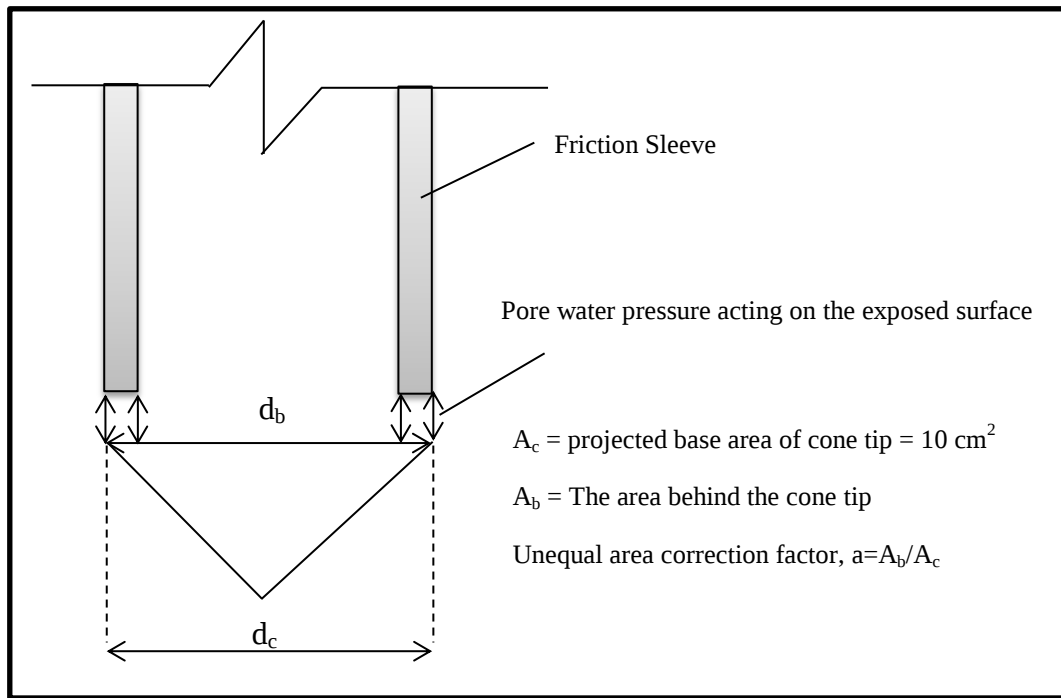


Figure 2.11 Dimensions for unequal area correction.

The CPT has its limitations. The most significant limitations of the conventional CPT are its inability to obtain a soil specimen for visual assessment and to detect relatively thin layers of soil. As shown in Figure 2.12, the zone of influence on tip resistance could extend far below a cone tip. In stiff sand, the influence zone could reach to a distance of 20 cone diameters ($\approx 70 \text{ cm}$) (Lunne et al., 1997). Therefore, the tip resistance is more

illustrative of the average strength and deformation characteristics of soils over some vertical interval than at the actual elevation of the cone tip. The size of the affected zone depends on the strength and stiffness of the soil.

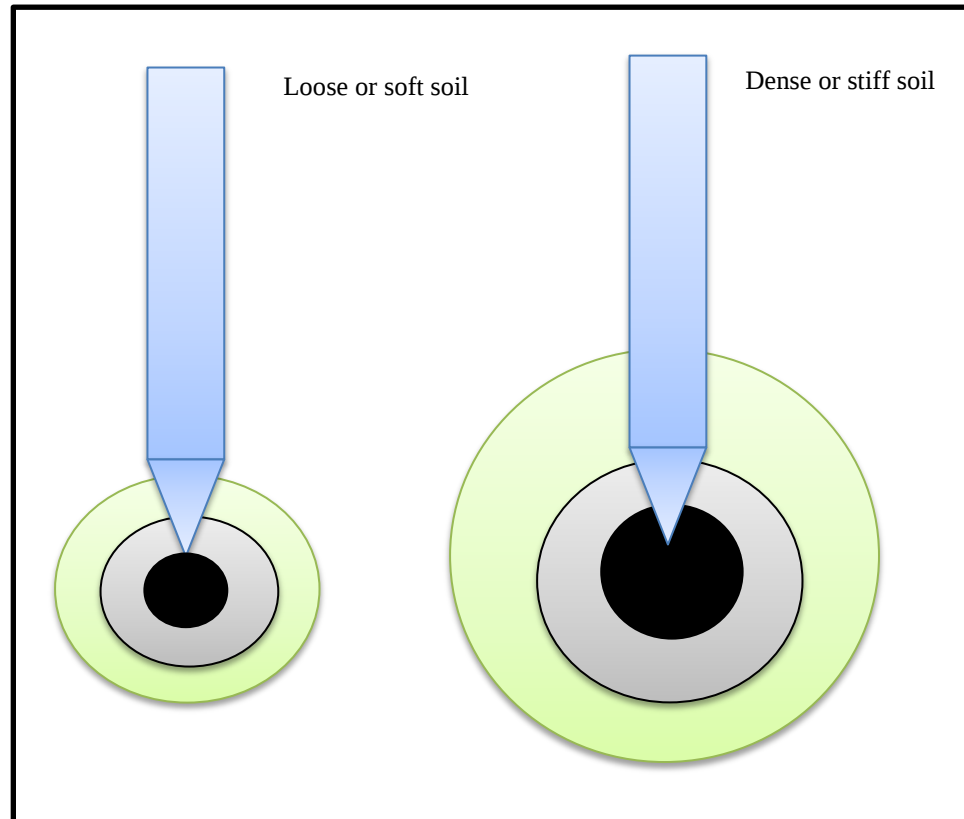


Figure 2.12 Zones of influence on cone tip resistance.

2.5.2 Standard penetration test (SPT)

The Standard Penetration Test (ASTM D 1586) has been established for the geotechnical field testing for many decades. This test consists of dropping a 63.5 kg hammer from a height of 76 cm to push a metal, split-spoon soil sampler 45 cm into the bottom of a soil boring in three 15-cm increments and recording the number of blows required to drive the sampler each of the three 15 cm increments. The number of blows for the second and third 15 cm increments is added, and the resulting number is

commonly stated as the “blow count or NSPT value”. Over time, adjustments have been done to reduce or eliminate the source of many errors found with the original SPT practice. Recently, additional data has been achieved when SPTs are performed by applying a clockwise torque to the sampler after the sampler has been pushed 45 cm into the ground (Ranzini, 1988).

There is great uncertainty in SPT measurements. There are some sources of errors related to the SPT procedure and equipment that can be used before interpretation of test results. The blow count numbers calculated in the field are corrected as follows

$$N_{60} = C_h C_r C_s C_d N_{SPT}, \quad (2.11)$$

where N_{60} = blow count for 60% hammer energy efficiency (Blows), C_h = hammer correction (unitless), C_r = rod length correction (unitless), C_s = sampler correction (unitless), and C_d = borehole diameter correction (unitless), N_{SPT} = measured in situ blow count (Blows). C_h corrects for the type of hammer because different hammers have different energy efficiencies. C_r corrects for the rod length because measured N values performed with a rod length less than 10 meters has artificially high N values and must be corrected. C_s corrects for a sampler that accommodates a sampler liner and is used without the liner. Without the liner, the sampler allows for internal relief and the measured N value is artificially low. C_d corrects for the diameter of the borehole. As the diameter increases, the measured N value becomes artificially low due to a decrease in confinement at the base of the borehole and must be corrected.

Based on an adjustment of work proposed by Terzaghi and Peck (1967), Kulhawy and Mayne (1990) suggested the following N_{60} vs. undrained shear strength S_u correlation

$$\frac{S_u}{P_a} = 0.06N_{60}, \quad (2.12)$$

where S_u = undrained shear strength, P_a = reference stress (100 kPa), N_{60} = blow count for 60% hammer energy efficiency. Based on Hara et al. (1974), N_{60} is related to S_u as

$$\frac{S_u}{P_a} = 0.29(N_{60})^{0.72}. \quad (2.13)$$

Commonly, measured N values for soft clayey soils are low. For example, the percent error for ten blows when the N value is measured incorrectly by 1 blow is 10%. As the blow count decreases, the percent error increases. When the N value is calculated in either Equation 2.7 or 2.8 or a similar relationship, the error is being compounded and this may lead to extremely conservative or unconservative undrained shear strength estimates that are then used for engineering design.

2.5.3 Vane shear testing (VST)

Vane shear test (VST) has been advanced by John Olsson in 1920s. The test is used for in-situ characterization of the undrained shear strength of intact, fully saturated clays. This test is performed by pushing a vane into soil (Figure 2.13), and rotating it to failure. The torque applied to fail the soil under shear is calculated. Then the undrained shear strength is measured from the torque at failure, the vane dimensions, and other factors such as rate effects. As the vane is thin to minimize soil disturbance, it can only be expanded in soft to medium cohesive soils. This test is a quick test in low permeability soils; hence, the measured strength is always undrained shear strength. Shear failure occurs near the surface and ends of a cylinder having a diameter to overall width of the vane. The shear strength is calculated as

$$T_f = \pi \cdot S_u \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right), \quad (2.14)$$

where T_f is the torque at failure, d is the overall vane width, and h is the vane length. Nevertheless, the shear strength over the cylindrical surface may be different than the horizontal end surfaces, subjected to the anisotropy of the soil. If necessary, after the initial test, the vane may be rotated rapidly through several rotations to achieve remolded shear strength.

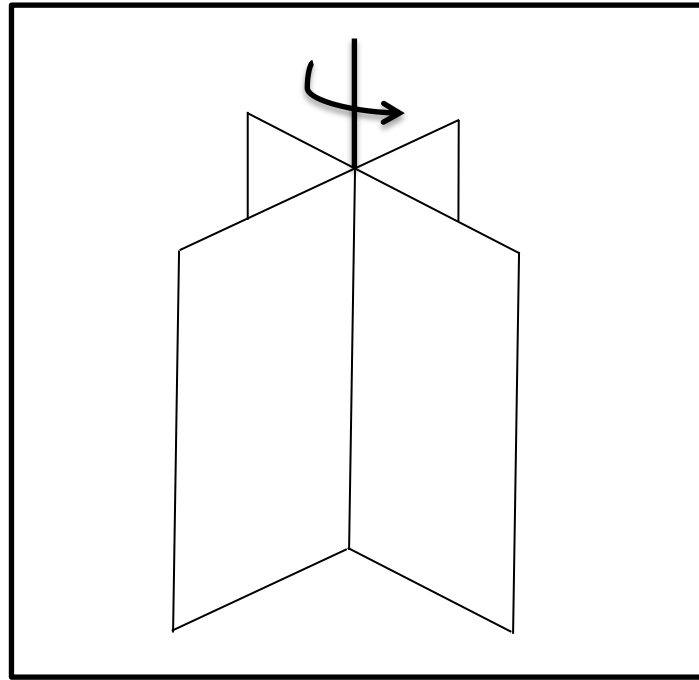


Figure 2.13 The vane shear test.

An important matter with the field vane (FV) shear test is whether undrained shear strength found from this test for overconsolidated soils can be used for foundation design. A number of researchers (Dascal et al., 1972; La Rochelle et al., 1974; Graham, 1979; and Lefebvre et al., 1987) have noticed significantly higher values of S_u from FV tests than those calculated from conventional laboratory tests and back analyses of field failures. Dascal et al. (1972) performed a full-scale embankment failure test, and the S_u found from the back analysis of this artificially failed embankment was compared with S_u

obtained from FV shear test using conventional analysis. $S_u(\text{FV})$ was discovered to be much greater than the back-analyzed S_u , even after reducing the $S_u(\text{FV})$ by using the correction factor proposed by Bjerrum (1972). Also, Lefebvre et al. (1987) performed various tests to compare with the results of the FV test. The FV test denoted much higher strength when compared with other field and laboratory tests. Therefore, it is recommended that the FV test should not be used to determine the S_u of a clay crust, but only its thickness. Garga and Khan (1992) related this behavior to the fact that the major portion of the torque mobilized is developed on the cylindrical face of the field vane, which is normal to the in-situ horizontal stress (σ'_{ho}), where $\sigma'_{ho} > \sigma'_{vo}$. They have suggested that the undrained shear strength on top and bottom horizontal planes of a FV (S_{uh}) compares well with S_u obtained from K_o consolidated laboratory triaxial tests. Therefore, Garga and Khan (1992) are proposing $S_{uh}(\text{FV})$ should be used for design in the overconsolidated crust instead of $S_u(\text{FV})$ from conventional analysis.

2.5.4 T-bar and ball penetrometers

Full flow penetrometers should ensure a projected area of about 10 times that of the connecting shaft, and, the length of the T-bar should remain 4 to 6 times the diameter for the shaft. When the penetrometer is attached directly to standard size cone rods (35.6 mm diameter), the recommended dimensions for the T-bar are a length and diameter of 250 mm and 40 mm, and for the Ball a diameter of 113 mm (Figure 2.14). These dimensions give a subsequent projected area of 100 cm^2 compared with the standard cone area of 10 cm^2 (Randolph, 2004; Randolph and Andersen, 2006; Boylan and Long, 2007; Yafraate et al., 2007; and DeJong et al., 2008). In offshore practice, it has been ordinary to adopt a

smaller Ball diameter of 60 to 80 mm, with a correspondingly reduced diameter of connecting shaft (Kelleher and Randolph, 2005; and Peuchen et al., 2005).

Penetrometer shape and ratio are the main factors affecting soil flow during penetration. Area ratio is defined as

$$A_R = \frac{A_s}{A_p}. \quad (2.15)$$

Compared to the ball, the T-bar is more critical to bending if variable circumstances are encountered and vertical alignment may not be supported due to probe drift. It is also possible for the T-bar to twist, causing extraction or cycling along a different orientation, although local softening of the soil will help prevent this. The axisymmetric geometry of the ball is self-centering and thus less susceptible to drift during penetration and extraction. The geometry of the ball also enables placement within smaller diameter drill pipe (Peuchen et al., 2005).

To guarantee full flow and appropriately assess soil strengths, a penetrometer area ratio of 10:1 is recommended. As the area ratio decreases towards 1:1, the influence of the displacement produced by the push rod on the flow mechanism increases and eventually turns to be similar to the full displacement that happens around the cone penetrometer.

In situ investigation with T-bars of 2, 5, 10 and 15:1 area ratios indicated penetration resistance decreases with increasing area ratio and generally stabilizes between an area ratio of 5:1 and 10:1. Less than a 10% increase in penetration resistance was noticed when the area ratio was reduced to 5:1.

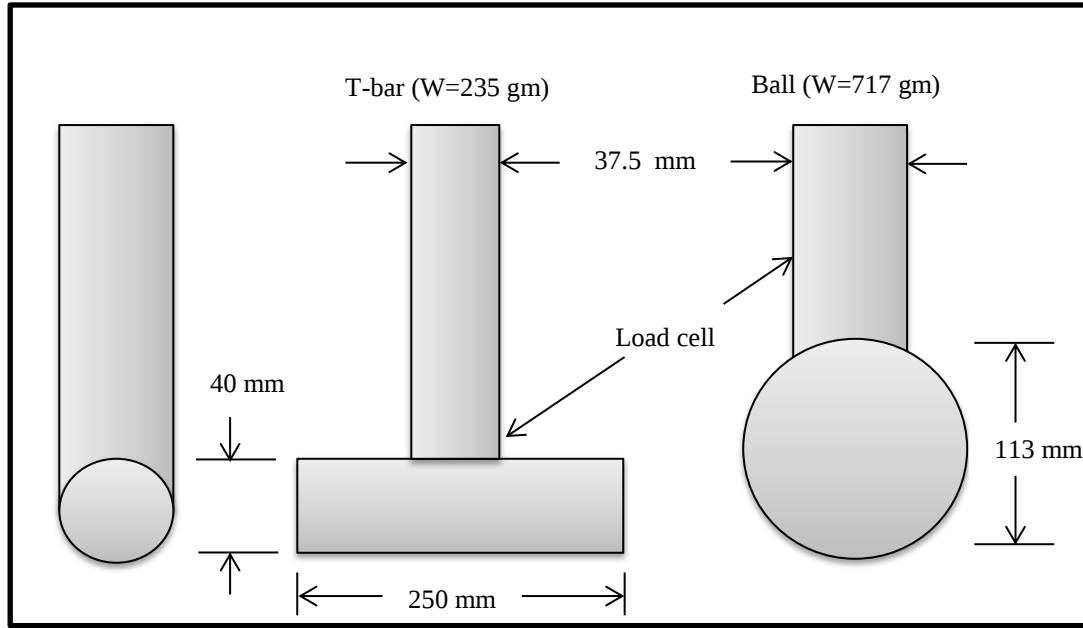


Figure 2.14 Schematic T-bar and ball recommended dimensions.

The undrained shear strength is predicted as the ratio of initial net penetration resistance to a strength factor N .

$$S_u = \frac{q_{net(T-bar)}}{N_{(T-bar)}} = \frac{q_{net(Ball)}}{N_{(Ball)}}, \quad (2.16)$$

where S_u is undrained shear strength obtained from a reference test (FVT), $q_{net(T-bar)}$ and $q_{net(Ball)}$ are the initial net penetration resistance for the T-bar and ball, $N_{(T-bar)}$ and $N_{(Ball)}$ are undrained shear strength factors for the T-bar and ball. Both sensitivity and rate of strain softening were shown by Yafrate et al. (2007) and DeJong et al. (2008) to have a primary influence on empirically estimated N values.

However, the minimum measured undrained shear strength were 8 kPa and 11 kPa as T-bar and ball penetrometers used respectively (DeJong et al., 2011) while in the ultra-soft soil the undrained shear strength may have much lower value and it could reach to

0.01 kPa (Vipulanandan and Raheem, 2015). Hence, new penetrometer with more accuracy is required to quantify the undrained shear strength of ultra-soft soil.

2.6 Anchor

Recently, attention is focused on the offshore construction as the demands in offshore and deepwater development gradually increase; and it is expected that a number of offshore structures will be constructed in the near future. Because of the severe construction condition, a few foundation systems are economically feasible for the offshore foundations, one of which is the anchor (Lee et al., 2012). For the analysis of the local variations of the shear strength, the forces can be characterized as either bearing forces acting on the leading edges of the anchor or shearing forces acting on the planes of the anchor in the direction of penetration. The soil will be characterized by the undrained shear strength, the submerged unit weight and the sensitivity of the clay (Ruinen, 2004).

One of the critical challenges in offshore deep water application is the proper understanding of the anchor-soil interaction which is mainly affected by the soft soil properties. Very soft soils are encountered in the top layer of the deep-water sea bed affecting the stability of the infrastructure placed on it (Nazir and Azzam, 2010). Anchor failures can cause drifting and collusion with production structures resulting in damages to oil / gas pipelines of subsea production systems (Monaco, 2013). Drag embedment anchors are the most used anchor for floating systems in the Gulf of Mexico (Yang et al., 2012). There have been several anchor failures during hurricanes (Ivan, Katrina, Rita, and Ike) in the past few years (Aubeny et al., 2011). For instance, hurricane Ike had caused at least four failures in floating systems. While during hurricanes Ivan, Katrina, and Rita, 24 floating systems experienced failures. Anchors were dragged during some of hurricane

failures and caused numerous instances of pipeline loss that in turn led to interruption in restoring oil and gas production after the hurricanes. Hence, better characterization of soft soil is required for quantification of any differential movements beyond acceptable limits could result in high strain to exceed the design limit, or in extreme cases, buckling or tensile fracture (Castille et al., 2009).

In estimating the ultimate bearing capacity of deep foundations, several methods have been developed and used in geotechnical engineering practice. The application of Cone Penetration Test (CPT) in predicting the ultimate bearing capacity of anchor has increased over the last two decades due to the similarity between cone penetrometer and anchor foundation. However, traditional static analysis based on soil characteristics obtained from soil boring and laboratory tests have been mainly used in practice. Compared with the average cost of the traditional boring and the associated laboratory test, the CPT is less costly by 5.3 to 5.9 times (Titi and Abu-Farsakh, 1999). In addition, the CPT is simple, fast and provides continuous soil soundings with depth. Due to the large degree of uncertainties in deep foundation analysis associated with the empirical formulas and design engineer's judgments, anchor load tests at field are usually conducted under 2 to 3 times of design load or until its failure. Although anchor load test cannot be a substitute of anchor design analysis, it usually helps in verifying its anchor design.

Several methods were developed to estimate the axial capacity of anchors utilizing the CPT (Schmertmann, 1978; Bustamante and GIANESELLI, 1982; De Ruiter and BERINGEN, 1979). These methods can be classified into two groups: direct methods from CPT data, the cone tip resistance (q_c) and sleeve friction (f_s); or

indirectly through evaluating strength soil parameters such as shear strength (S_u) and angle of internal friction (ϕ). In CPT direct method, the unit end bearing capacity (q_t) is predicted from the measured cone tip resistance (q_c) through the influence depth, and the unit skin friction (f) is evaluated either from the measured cone tip resistance (q_c) or from the measured sleeve friction (f_s).

2.6.1 Static analysis

The predicted anchor capacities (Q_p) were calculated using the static method (α -Tomlinson method for clay and Nordlund method for sand) depending upon the soil conditions.

(a) α -Tomlinson method

α -method (Tomlinson, 1980) is based on total stress analysis. For a soil with $\phi = 0$ or in total stress analysis, the ultimate skin resistance per unit area of pile can be calculated as

$$f_s = \alpha S_u, \quad (2.17)$$

where α is an empirical adhesion coefficient and S_u is the undrained shear strength.

The skin friction resistance (Q_s) is as

$$Q_s = \int_0^L f_s C_d dz, \quad (2.18)$$

where L is length of pile in contact with soil and C_d is effective perimeter of pile. The pile tip resistance (Q_p) is calculated as

$$Q_p = A_p S_u N_c, \quad (2.19)$$

where A_p is cross sectional area of the pile and N_c of 9 is used in this study.

(b) Nordlund method

In sand, the anchor tip resistance (Q_p) can be calculated as

$$Q_p = A_p q \propto N_q, \quad (2.20)$$

where q is the effective vertical stress at tip level, α is a dimensionless correction factor, and N_q is a bearing capacity factor varying with ϕ .

The skin friction resistance (Q_s) was evaluated using the equation proposed by Nordlund (1963, 1979) in this research as

$$Q_s = \int K_\delta C_f P_D \sin(\delta) C_d dz, \quad (2.21)$$

where K_δ is a coefficient of lateral stress, P_D is effective overburden pressure, δ is anchor-soil friction angle, C_d is effective anchor perimeter, and C_f is a correction factor.

2.6.2 CPT methods

There are two main approaches in estimation of anchor capacity using CPT data, which are the indirect method and the direct method. In the indirect method, CPT data is used to estimate soil strength parameters to predict anchor capacity. It is believed that the indirect method is less suitable in engineering practice (Eslami and Fellenius, 1997). In direct CPT methods, the anchor capacity is predicted from the sum of anchor tip resistance (Q_t) and the skin friction resistance (Q_s), which can be expressed as the following equations:

$$Q_t = q_t A_b = (c_b q_{c,avg}) A_b, \quad (2.22)$$

$$Q_s = f A_s = \alpha_c f_s A_s \text{ and} \quad (2.23)$$

$$Q_s = f A_s = (C_s q_c) A_s, \quad (2.24)$$

where q_t is the unit tip resistance, q_c is the cone tip resistance, $q_{c,avg}$ is the average cone tip resistance in the zone above and below the anchor tip, f is the unit skin friction, f_s is the cone sleeve friction, c_b is the correlation coefficient of tip resistance, α_c is the reduction factor, c_s is the correlation coefficient of friction resistance, A_b is the anchor tip area, and A_s is the anchor surface area.

2.6.3 Schmertmann method

Schmertmann (1978) proposed a direct CPT method based on the model and full-scale anchor load tests. To estimate the anchor tip resistance (Q_t), the average cone tip resistance ($q_{c,avg}$) is obtained in the zone ranging from 8D above the anchor tip to 0.7D-4D below the anchor tip(where D is the anchor diameter). Schmertmann suggested c_b of 1.0 for sand and 0.6 for clay. The unit skin friction is calculated from the sleeve friction (f_s) and α_c of 0.2 to 1.25 for clayey soil was proposed.

2.6.4 Bustamante and Gianeselli method (LCPC/LCP method)

Bustamante and Gianeselli method is known as the French method or LCPC/LCP method (Bustamante and Gianeselli, 1982). In this method both unit tip resistance (q_t) and unit skin friction (f) are calculated from the cone tip resistance (q_c). The average cone tip resistance ($q_{c,avg}$) is obtained in the zone ranging 1.5 D above and below the anchor tip. The correlation coefficient of tip resistance (c_b) from 0.15 to 0.6 was proposed for

different soil types and installation procedure based on the empirical correlation. The unit skin friction (f) is obtained from cone tip resistance (q_c) and the correlation coefficient of friction resistance (c_s) varies from 0.5 to 3.0 % depending on soil type, anchor type and installation process.

2.6.5 De Ruiter and Beringen method

De Ruiter and Beringen method is known as the European method and based on the experience from the anchor tested in offshore in the North Sea (De Ruiter and Beringen, 1979). In sand, the unit tip resistance (q_t) is obtained from same way as Schmertmann method. The unit skin friction (f) for the compression anchors is the minimum among (f_s , $q_{c(side)}/300$, and 20 kPa). In clay, the unit tip resistance (q_t) is determined from the conventional bearing capacity theory as

$$q_t = N_c S_u \text{ and} \quad (2.25)$$

$$S_u = \frac{q_c}{N_k}, \quad (2.26)$$

where N_c is the bearing capacity factor and N_k is the cone factor ranging from 15 to 20 depending on soil type and pile type. The unit skin friction (f) can be obtained as

$$f = \beta S_u, \quad (2.27)$$

where β is the adhesion factor: $\beta = 1$ for normally consolidated (NC) clay and 0.5 for overconsolidated (OC) clay.

2.7 Summary

Based on the literature review, the following can be concluded:

- 1) The most characteristic mineral component of bentonite is crystalline and definitely montmorillonite.
- 2) It has been suggested that kaolinite, which has the composition $\text{H}_2\text{Al}_2\text{Si}_2\text{O}_8 \cdot \text{H}_2\text{O}$, may, without its water of crystallization, be regarded as the first of a series of acids of which cimolite and pyrophyllite are respectively the second and third members.
- 3) Soil can be classified based on several parameters such as water content, void ratio, porosity, degree of saturation, density, and activity.
- 4) Clayey soil can be further classified based on the plasticity index where soft clay can have plasticity index higher than 40.
- 5) Particle size diameters of soft soil is less than 0.002 mm.
- 6) Soft soil mainly is any soil with shear strength lower than 25 kPa.
- 7) Soft soil sensitivity could be greater than 16.
- 8) Soft soil can be considered very sticky material.
- 9) Adhesion for soft soil could be lower than 12 kPa.
- 10) Soft soil strength may reach lower than 0.2 kPa in onshore or offshore conditions.
- 11) Very limited attempts have been done to explore soft soil properties using electrical resistivity method. However, almost no research work has been carried out so far to correlate electrical resistivity with soft soil strength soil.
- 12) Traditionally, lime and cement have been used to increase the strength of the soft soil. Nevertheless, the treated soil strength usually was higher than 10 kPa where no ultra-soft soil treatment has been investigated.

- 13) Never polymer effectiveness was investigated to treat ultra-soft soil.
- 14) Most of offshore soft treatment needs extraordinary huge in-situ equipment. However, the offshore treatment usually ignores the surface soft soil treatment and used for high-depth treatment.
- 15) Serious problems can be caused by salt contamination to the oil wellbores such as losing filtration control, shale swelling, stress concentration around the wellbore, stuck pipe, casing failure, lost circulation, and cementing failures. No clear study is done to incorporate the impact of both temperature and salt contamination on the rheological properties of drilling mud or the shear strength of ultra- soft soil.
- 16) No precious measurement is used to monitor the behavior of drilling mud or ultra-soft soil under the impact of combined effect of temperature and salt contamination.
- 17) Fluid loss and filter cake permeability are the most important factors to be studied in oil well conditions. However, no unique precious model is available to quantify the fluid loss, cake permeability, cake porosity, and the relative solid content during cake formation.
- 18) None of the available current methods that used for in-situ soft soil characterization is able to quantify the ultra-soft soil shear strength. The minimum measured undrained shear strength were 8 kPa and 11 kPa as T-bar and ball penetrometers used respectively while the ultra-soft soil strength is about 0.01 kPa.
- 19) No studies have been investigated the anchor-ultra-soft soil interaction. However, several severe damages have been reported during the Hurricane events where the ultra-soft soil is exist in the offshore surface.

Chapter 3 - Materials and Methods

3.1.Introduction

In this chapter, the details of different materials and methods that were used in this study are discussed. Different materials such as bentonite, kaolinite, lime, polymer, sand, and cement have been used. Various devices such as conductivity meter, impedance spectroscopy analyzer, electrical viscometer, CIGMAT penetrometer, high pressure and high temperature device, modified vane shear, ultrasonic pulse velocity test, and low load-displacement controlled device have been used.

3.2. Materials

3.2.1. Bentonite

In this study, an ordinary (unaltered), light gray-colored commercial sodium bentonite was used to prepare the 2% and 10% bentonite (w/w) soft soil. The bentonite that used in these tests was had a liquid limit of 500 and plastic limit of 100 with a density between 2.65 kN/m^3 to 2.75 kN/m^3 . When 30%-deionized water was added to the bentonite (on a dry weight basis), 0.203 m (8 in) slump would occur. Such kind of bentonite soft soil exhibits sufficient viscosity, and shear strength, with only minimal filtrate loss. A bentonite concentration of 5% (dry weight basis) is typical of that used in the field, which have low shear strength and high moisture content.

3.2.2. Kaolinite

Kaolinite is white aluminosilicate clay, with fine and porous, non-abrasive particles disperses easily in water. The kaolinite clay used in this study had a liquid limit of 62,

plasticity index of 30, and specific gravity of 2.65. Kaolinite was used to prepare the 2% and 10% kaolinite (w/w) soft soil.

3.3.Improvement materials

3.3.1.Lime

Lime for soft soil treatment purposes is typically used in the form of quicklime (CaO) or hydrated lime ($\text{Ca}(\text{OH})_2$). Quicklime (CaO) is manufactured by chemical activities that convert calcium carbonate (limestone – CaCO_3) into calcium oxide (CaO). When quicklime reacts with water, it transforms into hydrated lime. The hydrated lime ($\text{Ca}(\text{OH})_2$) reacts with soft clay particles and modifies the the soft clay based on its mineralogy.

3.3.2.Polymer

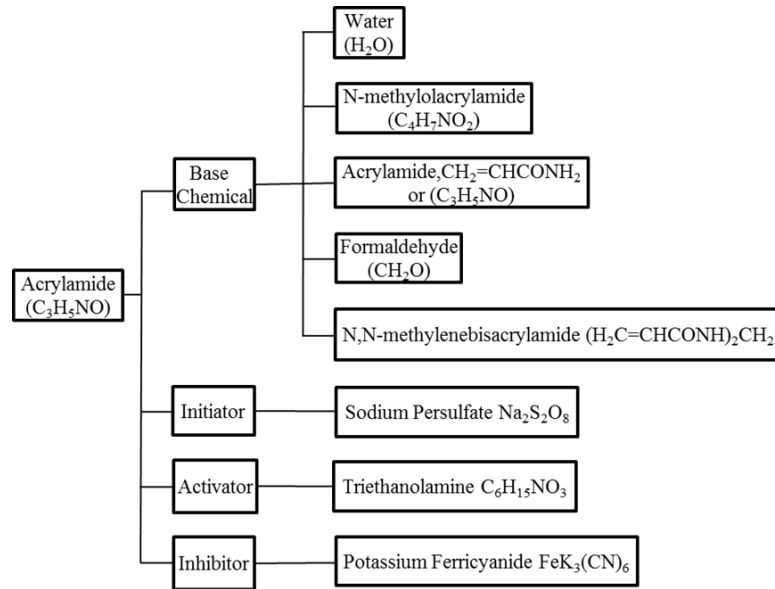
A polymer solution was prepared by mixing 15% of water-soluble acrylamide polymer with 0.5% of catalyst, 0.5% of activator, and 84% of water. Thus, the polymer solution had 15% polymer dissolved in it. The pH of the polymer solution was 10. Hence, if 10% of the polymer solution content is used to treat the soft soil (based on dry weight of soil), then the actual amount of polymer used is 1.5%. Three different types of polymer mixes 1%, 5%, and 10% were used in this research to check the effectiveness of the polymer in treating the soft soil (Table 3.1). The used acrylamide polymer chemical composition in summarized in Figure 3.1.

Table 3.1 Polymer solution mixes design

Mix	Polymer (gm)	Catalyst (gm)	Activator (gm)	Water (gm)	Total (gm)
1	6.67	0.5	0.5	92.33	100
2	33.34	0.5	0.5	65.66	100
3	66.67	0.5	0.5	32.33	100

3.3.3.Sand

Uniformly-graded sand with sub-rounded typical shape of the grains was used in this study. It had a coefficient of uniformity (C_u) of 2.53, a coefficient of gradation (C_c) of 0.90, and 50% of the particles passing a 0.46 mm sieve ($d_{50} = 0.46$ mm). Hence, the average surface area was $0.05 \text{ m}^2/\text{g}$. The specific gravity of sand was 2.65. Used sand was siliceous in mineralogy with no organics or soil finer than the No. 200 sieve size.

**Figure 3.1 Chemical notation of components of acrylamide polymer.**

3.3.4.Cement

A Portland cement was used as an improvement agent to treat ultra-soft soil. It is one of the cements that has the property of setting and hardening under water by virtue of chemical reactions. A Portland cement particle is a heterogeneous substance, containing minute tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and a solid solution described as tetracalcium alumino-ferrite (C_4A).

3.4.Conductivity meter

Conductivity was measured using a Thermo-Orion conductivity meter model 125, which has a range of 0 to 199.9 $\mu S/cm$ and accuracy of 0.01 $\mu S/cm$. Before measurement, conductivity meter was calibrated using the standard solution. Standard solutions differ based on diverse applications. In this study, since the samples conductivity is around 10 mS/cm, the standard solution with the standard conductivity of 12.9 mS/cm in 25°C was prepared. To prepare this solution, 7.23 gr of NaCl was stirred inside 1 liter of deionized water and then calibration of the conductivity meter was performed based on the equation:

$$\text{Calibration factor} = \frac{\text{Standard conductivity}}{\text{Measured conductivity}}. \quad (3.1)$$

With this equation, a calibrated coefficient is specified to the conductivity meter and with that, the device is then calibrated. Sample conductivity was then measured using the calibrated conductivity meter device and based on the following equation

$$\rho(\Omega. m) = \frac{1}{\text{Measured conductivity} \left(\frac{mS}{cm} \right)} * 10, \quad (3.2)$$

the electrical resistivity was calculated. The conductivity meter can be identified in Figure 3.2.

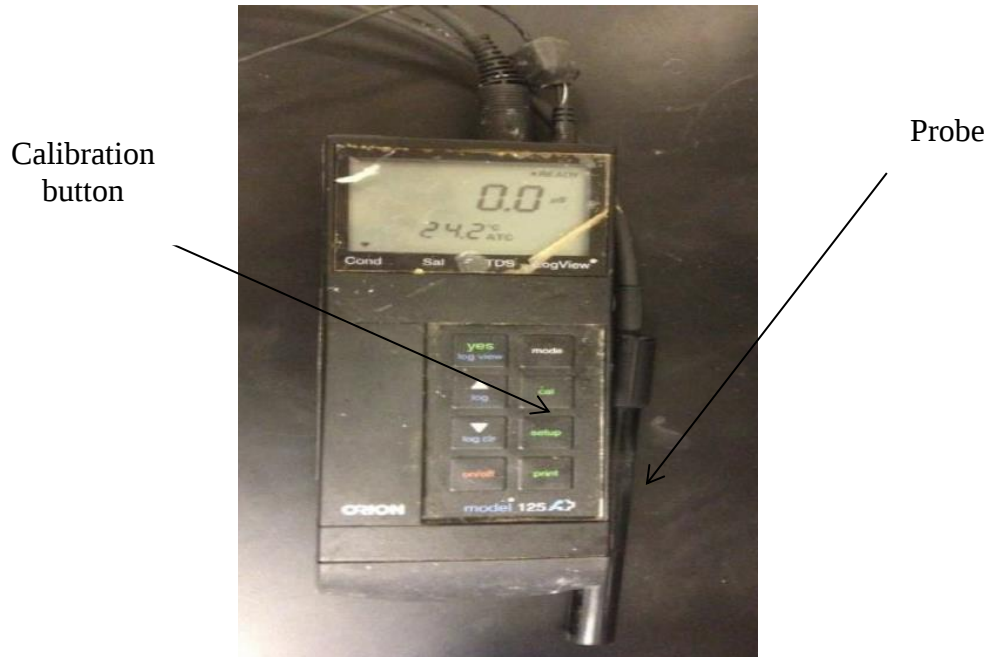


Figure 3.2 Conductivity meter device.

3.5.Electrical resistivity

Electrical resistivity of the sample can be estimated from the measurement of the electrical resistance, having a known uniform cross section and uniform length according to the following equation

$$R = \rho \frac{L}{A} = \rho * k, \quad (3.3)$$

where ρ is the resistivity of the sample, R is the measured resistance, A is the cross-sectional area, and L is the distance between two probes.

3.6. Mold calibration

To measure the resistivity of any sample after particular time, every mold was calibrated with the standard salt solution. Resistance and resistivity of the sample were measured using the LCR meter (AC-resistance meter) and conductivity meter device. A/L is defined as a K parameter of the specimen, which is defined as

$$K = \frac{R}{\rho} \text{ and } K = \frac{L}{A}. \quad (3.4)$$

The mold configuration can be seen in Figure 3.3.

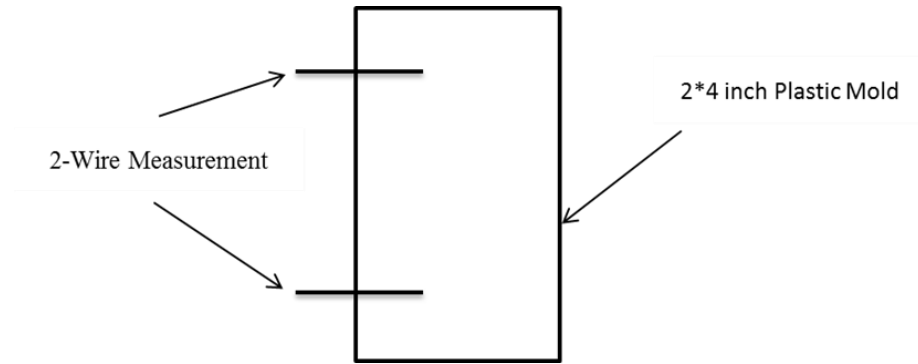


Figure 3.3 Plastic mold configuration.

3.7. Impedance spectroscopy analyzer

Impedance spectroscopy was measured by the two-probe method using AC measurement. Agilent E4980A LCR meter with 0.01 Ohm was used for AC measurement with a frequency range between 20 Hz to 300 kHz. Device description is shown in Figure 3.4.



Figure 3.4 Impedance spectroscopy analyzer.

3.8.Viscometer

Rheology of a drilling fluid considered as one of the fundamental physical properties. It is the analysis of the deformation of fluids where the main elements are viscosity, friction pressure loss, and the fluids velocity profile. Viscosity (μ) is the fluids internal resistance to its forced flow, or in simply how thick the fluid is. The viscosity of the fluid is affecting the fluid loss properties such as permeability and the thickness of the filter cake. A fluid does not necessarily have one fixed viscosity, it can change depending on the applied shear rate. Only Newtonian fluids have a determined viscosity, the most standard Newtonian fluid is water. Nevertheless, drilling fluids are not always Newtonian, most often they are non- Newtonian. The shear stress –shear strain rate of 2% and 8% bentonite drilling mud with and without salt contamination under different temperatures have been studied using digital viscometer. The viscometer is computerized and offers digital data collection provision. The instrument measures apparent viscosity at different strain rates ranging from 0.1 to 1200 s^{-1} as shown in Figure 3.5. The motor speeds range from 1 rpm to 600 rpm and the temperature can be controlled up to $85 \text{ }^{\circ}\text{C}$.

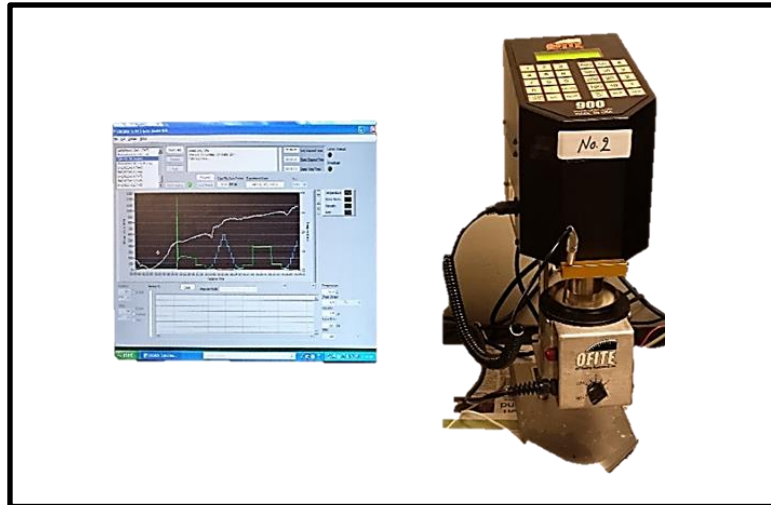


Figure 3.5 Digital viscometer.

3.9.CIGMAT penetrometer

Based on the size and weight of the miniature CIGMAT Penetrometer, it can be used in different soils and it is one of the best tools for measuring the shear strength for ultra-soft soil. A schematic figure of miniature CIGMAT penetrometer can be seen in Figure 3.6.

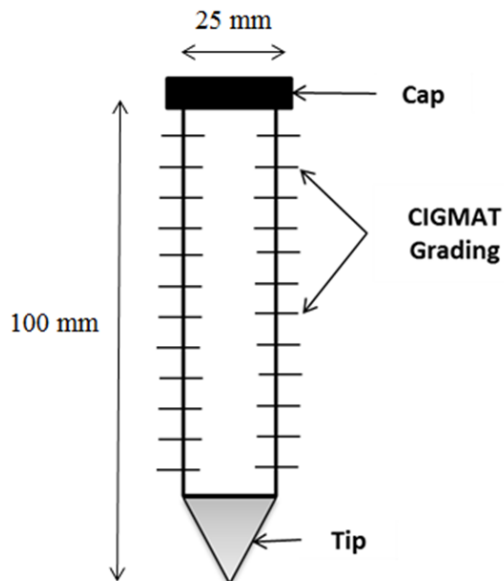


Figure 3.6 Schematic of miniature CIGMAT penetrometer

3.10. API-HPHT device

In the device setup, the standard filter press cylindrical mold (3" diameter* 3.4" height) was used and the pressure cell with filter paper was positioned in the bottom of the drilling mud. The standard API filter press test has the area of 45 cm^2 while the area of HPHT filter press test is 22.58 cm^2 . Various range of pressures and temperatures can be applied. Typical HPHT test is performed under a pressure of 100 psi and temperature of 100°C . HPHT device can be identified in Figure 3.7.



Figure 3.7 HPHT device.

During the filtering process, the air pressure is applied on the slurry and the filter cake is formed over the time on the bottom of the HPHT cell. The accumulative fluid loss is collected in a graded cylinder placed in underneath the HPHT cell. The schematic process

of filtering is clearly summarized in Figure 3.8. The characteristics of the HPHT filter press test that has been used for this study is summarized in Table 3.2. The viscosity ratio of the fluid in HPHT device is given in Table 3.3.

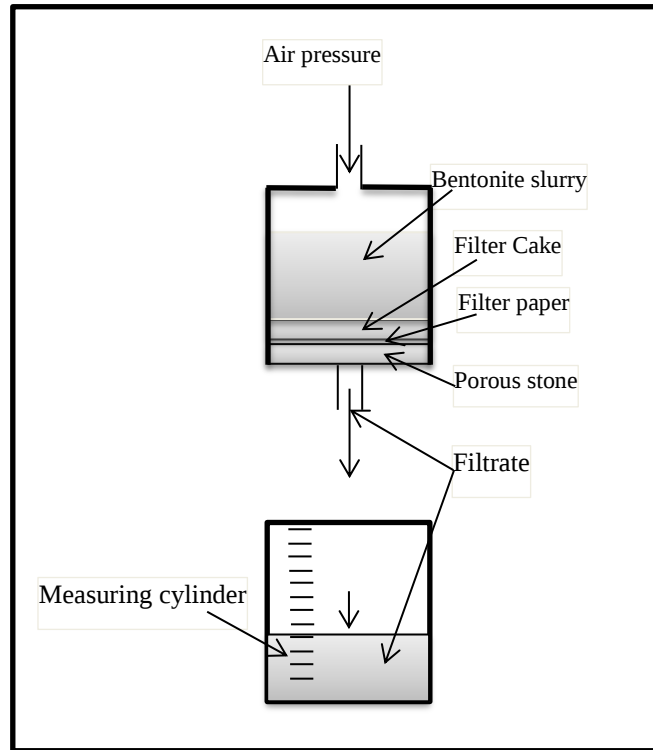


Figure 3.8 Schematic filtering process.

Table 3.2 HPHT device characteristics.

Category		Specification
Maximum Temperature		500° F (260° C)
Maximum Cell Pressure		1800 psi (12.4 MPa)
Backpressure		750 psi (5.17 MPa)
Sample cell volume		500 ml
Receiver volume		100 ml
Filtering area		3.5 in ² (22.58 cm ²)
Heating capacity		800 watts
Pressure connection		Nitrogen or Carbon Dioxide
Power requirement		115/230 VAC, Frequency 50/60 Hz

Table 3.3 Viscosity ratio of water at different temperatures.

Temperature ° C	Viscosity ratio $\mu(T)/\mu(20\text{ }^{\circ}\text{C})$ (Hardy and Cottingham 1949)
20	1
40	0.6518
60	0.4656
80	0.3577
90	0.3142
95	0.2970
100	0.2816
110	0.2544
120	0.2316
125	0.2216

3.11. Modified vane shear

Initially, vane shear test was developed and investigated extensively in Sweden form late 1940s. Undrained shear strength of cohesive or soft soil (S_u) is obtained using vane shear test by measuring torque ($T_{\max}$) and rotation (ASTM D 2573). The vane consists of four thin rectangular blades or wings welded to an extendable circular rod. Generally the height of the vane is about twice of its width. The vane is pushed into the soil for at least twice its height and then rotated at a constant rate of 0.1 to 0.2 degrees per second until the soil is failed. The maximum torque required to shear soil is then converted to the undrained shear resistance of the cylindrical surface. To measure the strength of ultra-soft soil, the height and diameter of the vane shear device have been modified to 4” and 2” respectively. Schematic details for the vane shear device is idealized in Figure 3.9. Different springs have been provided with the device and each spring has specific strength. Based on the degree of rotation to shear the soft soil for each specific spring, the mobilized torque is identified as shown in Table 3.4.

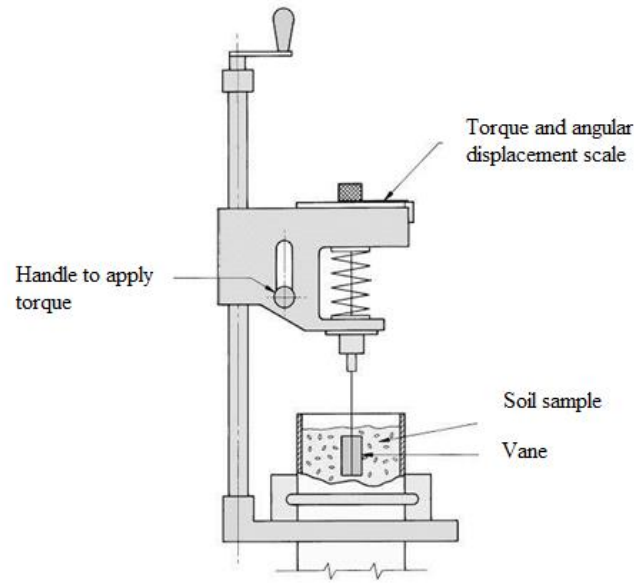


Figure 3.9 Schematic of vane shear device.

3.12. Ultrasonic pulse velocity test

The main use of ultrasonic testing are that it generates compression and shear wave velocities, in addition to ultrasonic values for the elastic constants of soft soil. The propagation velocities of the compression and shear waves, V_p and V_s , respectively, were as

$$V_p = L/T_p \text{ and} \quad (3.5)$$

$$V_s = L/T_s, \quad (3.6)$$

where V = pulse-propagation velocity, m/s,

L = pulse-travel distance, m, T = effective pulse-travel time (measured time minus zero time correction), sec, and subscripts P and S denote the compression wave and shear wave, respectively.

Table 3.4 Vane shear calibration chart.

Torque (kg.cm)	Degree of rotation			
	Spring no. 1	Spring no. 2	Spring no. 3	Spring no. 4
0.25	30	14	9	6
0.5	60	28	19	11
0.75	90	43	28	17
1	120	58	38	22
1.25	150	73	47	27
1.5	180	88	56	33
1.75		104	65	39
2		119	74	44
2.25		135	83	50
2.5		165	92	55
2.75		180	101	61
3			111	66
3.25			121	71
3.5			131	76
3.75			140	82
4			150	87
4.25			159	92
4.5			169	97
4.75			179	102
5				107
5.25				113
5.5				119
5.75				125
6				130
6.25				135
6.5				141
6.75				146
7				152
7.25				158
7.5				164
7.75				169
8				175

Young's modulus of elasticity (E_p) relating compression and shear waves is calculated as

$$E_p = \frac{\rho V_s^2 (3V_p^2 - 4V_s^2)}{(V_p^2 - V_s^2)}, \quad (3.7)$$

where E_p = Young's modulus of elasticity from pulse velocity method, MPa, and ρ = density of the material, kg/m³. The equation for Poisson's ratio (μ) is as

$$\mu = \frac{(V_p^2 - 2V_s^2)}{2(V_p^2 - V_s^2)} \quad (3.8)$$

Pulse velocities for the soft soil were measured using a commercially-available portable V-meter. Lead ceramic transducers with a natural frequency of 150 kHz were used to pass compression or shear waves throughout the specimens. Commercially-available grease was used to present good coupling between the specimens and transducers. The required travel time of the ultrasonic pulse through the specimens under direct transmission (with the transducers on opposite faces along the length) was measured up to an accuracy of 0.1 μ s.

3.13. Low load-displacement controlled device

In this study, a displacement-controlled device for anchor testing in compression and tension loading under different rates have been shown in Figure 3.10. Different anchor models made of steel, concrete, and plastic with diameter of 5 in and length of 1 ft were used. The soft soil was mainly prepared of 10% bentonite content. The test was performed in two different rates of 0.0333 and 0.3999 in / min.

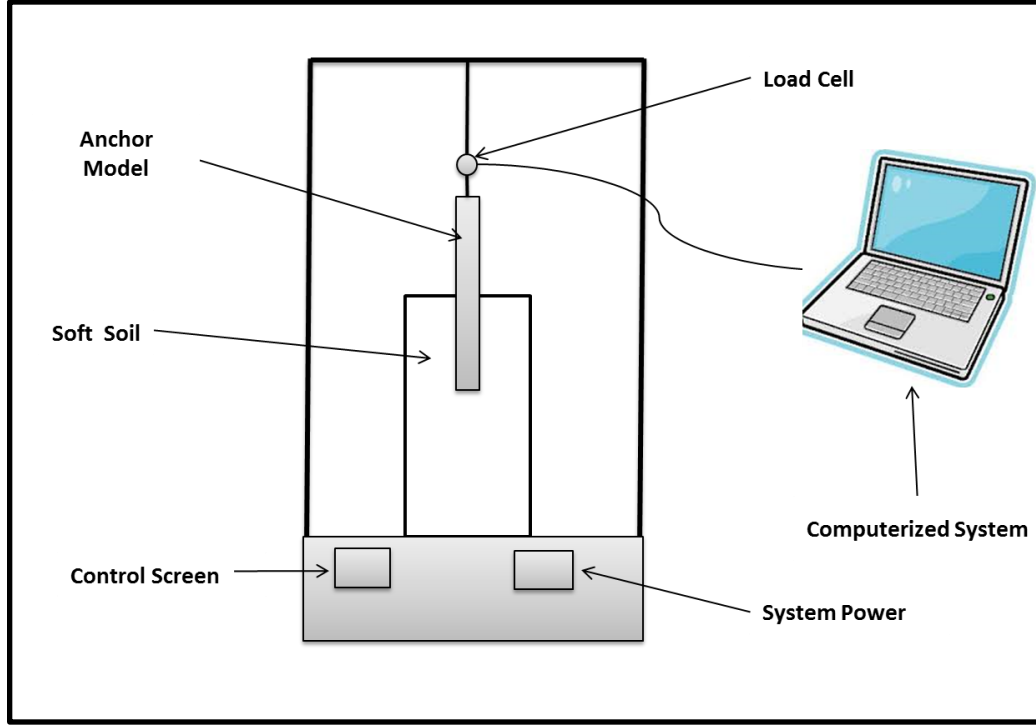


Figure 3.10 Laboratory set up of anchor testing.

3.14. Comparison of model predictions

In order to determine the accuracy of any model predictions in the study, both the root mean square error (RMSE) and coefficient of determination (R^2) in curve fitting as defined in Eqs. (3.9) and (3.10) were quantified using

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (y_i - x_i)^2}{N}} \text{ and} \quad (3.9)$$

$$R^2 = \left(\frac{\sum_i (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_i (x_i - \bar{x})^2} \sqrt{\sum_i (y_i - \bar{y})^2}} \right)^2, \quad (3.10)$$

where y_i is the actual value; x_i is the calculated value from the model; $\bar{y}$ is the mean of actual values; $\bar{x}$ is the mean of calculated values and N is the number of data points.

3.15. Summary

In this chapter, the following points can be summarized:

1. Different percentage of bentonite and kaolinite ranged from 2% to 10% were used to prepare the ultra-soft soil.
2. Different percentage of lime ranged from 2% to 10% was used to treat the ultra-soft soil for both short-term and long-term evaluation.
3. Different percentage of polymer ranged from 1% to 10% was used to treat the ultra-soft soil for short-term and long-term evaluation.
4. Different percentage of sand ranged from 1% to 3% was used to treat the ultra-soft soil.
5. Cement was used to treat ultra-soft soil with 10% up to 3 hours curing time.
6. Conductivity meter was used to measure the electrical conductivity (inverse of resistivity) of untreated and treated soft soil with different additives.
7. Impedance spectroscopy analysis is performed on untreated and treated ultra-soft soil using AC-LCR meter.
8. CIGMAT penetrometer is used as an in situ method to characterize untreated and treated ultra-soft soil with and without contamination.
9. API-HPHT is used to quantify the fluid loss of bentonite drilling mud for both short-term and long-term analysis.
10. Modified vane shear device is used to characterize the shear strength for untreated and treated ultra-soft soil with and without contamination.
11. Ultrasonic pulse velocity test is used to characterize the ultra-soft soil properties.
12. Low load-displacement controlled device is used to quantify the anchor-soft soil interaction using different applied displacement rates.

13. Viscometer is used to analyze the shear stress-shear strain rate relationship for bentonite drilling mud with and without contamination under different tested temperatures.

Chapter 4 - Soft Soil Characterization and Modification

4.1. Introduction

In this chapter, soft soil characterization of both bentonite and kaolinite were investigated. Soft soil characterization included low shear strength measurement, water content, electrical resistivity, and miniature penetrometer. Also, correlations such as shear strength versus solid content, water content versus solid content, electrical resistivity versus solid content, shear strength versus electrical resistivity, and water content versus electrical resistivity were studied. The effect of different additives such as lime, polymer, sand, cement, and kaolinite on the behavior of 10% bentonite soft soil was studied. The effect of different additives on the shear strength, electrical resistivity, water content, density, and electrical impedance of 10% bentonite soft soil were examined. Several small scale-tests have been performed to quantify the interface shear strength of soft soil using small concrete solid cylinders. Several soft soil compositions, including 10% bentonite soft soil, 10% bentonite soft soil treated with 5% to 20% lime, and 10% bentonite soft soil treated with 1% to 10% polymer were tested to quantify the maximum induced interface shear stress (shear strength). Soft soil profile characterization was studied using electrical resistivity measurements at different levels in parallel with hydrometer measurement. Large-scale laboratory test on particle settling was performed up to two days to monitor the changes in the electrical resistance, electrical resistivity, solid contents along the depth, and shear strength variation along the depth. The electrical resistivity technique has been tested through full-scale field test at the University of Houston to validate the efficiency of this method to detect the changes in the electrical resistivity with different materials and depth. Long-term effects of the additives such as

lime, polymer on the 10% bentonite soft soil properties up to 28 days have been considered. The penetrations of CIGMAT penetrometer with different weights on the shear strength of both 10% bentonite untreated and treated ultra-soft soil has been investigated. Finite element method (FEM) has been used to model the CIGMAT penetration versus shear strength of the soft soil. In addition, FEM has been extended to study the variation of the stress contours underneath the CIGMAT penetrometer for both untreated and treated soft soil. pH measurement for both untreated and treated bentonite ultra-soft soil have been explored. Electrical impedance for both untreated and treated soft soil has been modeled accurately. Experimental, analytical, statistical, and finite element method were used to model the stress-strain relationship for 2% and 10% bentonite soft soil. Non-destructive pulse velocity was used to study the variation of the compression wave velocity of soft soil with different bentonite contents. A new mathematical equation has been provided to correlate the shear strength and the water content for the soft soil. Uplift and compression behavior of different types of anchors such steel, plastic and concrete in bentonite soft soil has been studied under different loading rates. In addition, both analytical and numerical methods via hyperbolic model and finite element analysis have been used to model the load-displacement relationship for steel anchor in soft soil.

4.2.Soft soil preparation

4.2.1. Bentonite

Soft soil samples were prepared in laboratory by mixing different percentage of commercially Wyoming available bentonite from 2% to 10% with water at room temperature for about ten minutes until a homogenous mixture was obtained. The soil

slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. Low shear strength measurement is one of the major challenges in the laboratory or even in the field, however; the modified vane shear device was used to measure the extreme low strength of the prepared bentonite mud soft soil. In Figure 4.1, both shear strength and water content (percentage of water) of different soft soil vs. bentonite content is shown. It is clearly shown that the shear strength increased from 0.01 ± 0.0001 kPa to 0.17 ± 0.0017 kPa and the water content decreased from 98% to 90 % as the bentonite content increased from 2% to 10% respectively.

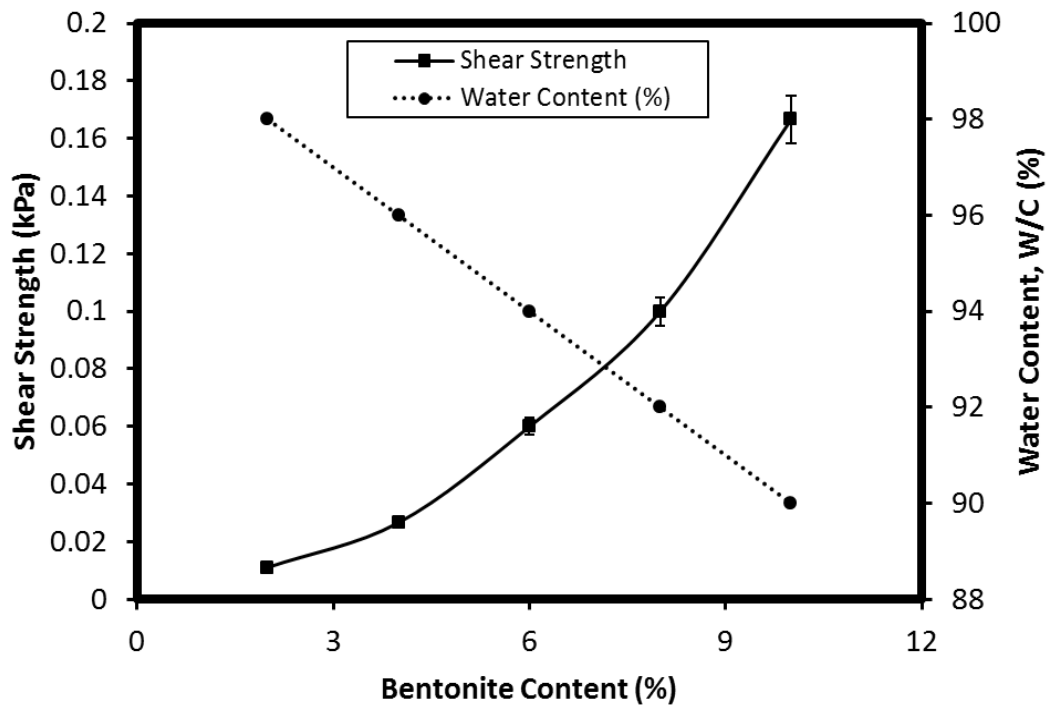


Figure 4.1 Variation of shear strength and water content with bentonite percent of soft soil.

Modified vane shear device and mode of failure of different samples of soft soil can be shown in Figure 4.2 and Figure 4.3 respectively. The zone of failure can be identified clearly at the ultimate applied shear stress (shear strength).

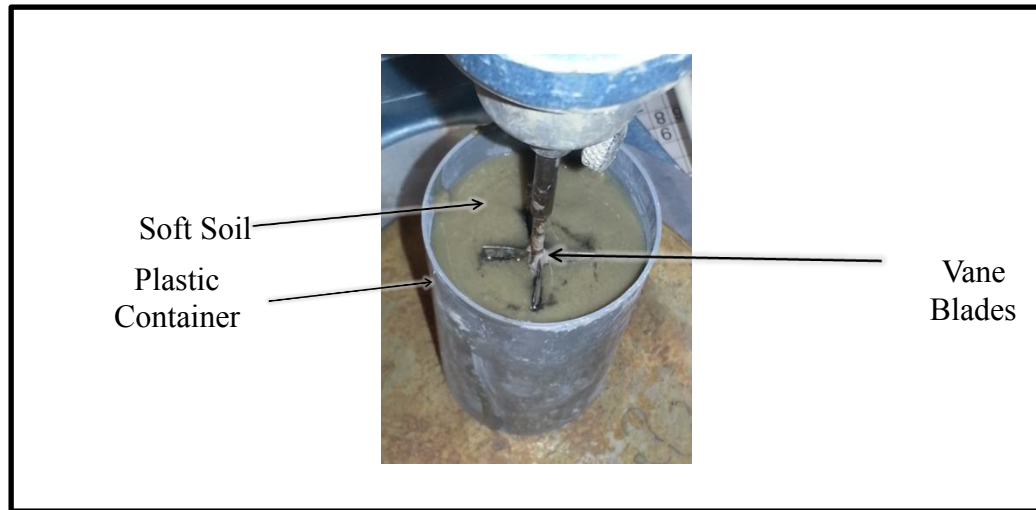


Figure 4.2 Modified vane shear test of soft soil.



Figure 4.3 Mode of failure of the soft soil.

The variations of both initial resistivity and water content of soft soil versus bentonite content has been studied as shown in Figure 4.4. As water content has decreased from 98% to 90%, the initial resistivity has decreased from 6.29 ± 0.1 Ohm.m to 2.56 ± 0.05 Ohm.m. Due to the increase in the soft soil shear strength with the increase of bentonite content, the initial resistivity has decreased because the medium became more conductive. The following model has been proposed to predict the initial resistivity vs. bentonite content

$$\rho_o = A * (\text{Bent. \%})^B, \quad (4.1)$$

where ρ_o is the initial resistivity, A and B are model parameters.

Eq. 4.1 showed a very good agreement with the experimental data having a high degree of correlation ($R^2=0.98$). Table 4.1 summarized the model parameters for Eq. 4.1.

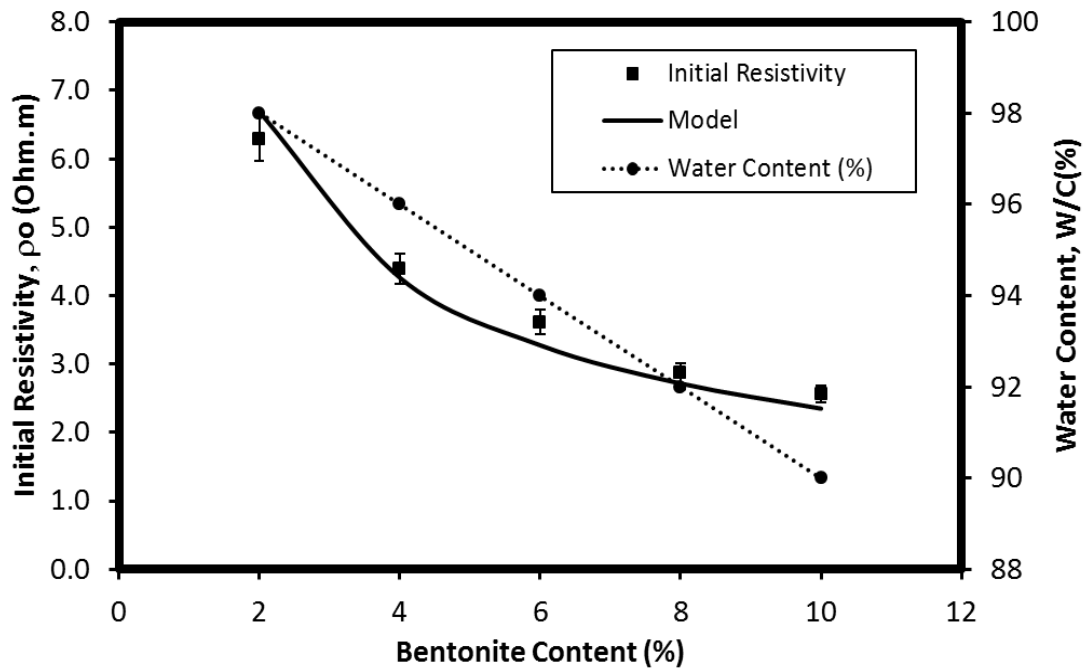


Figure 4.4 Variations of initial resistivity and water content of soft soil vs. bentonite content.

Table 4.1 Model parameters for Eq. 4.1

A (Ohm.m)	B	R ²
13	-0.8	0.98

The variations of shear strength of soft soil vs. initial resistivity can be shown in Figure 4.5. It has shown as the shear strength decreased, the initial resistivity increased. Shear strength vs. resistivity behavior has been modeled as

$$\text{Shear Strength} = C * (\rho_o)^D, \quad (4.2)$$

where ρ_o is the initial resistivity, C and D are model parameters. Eq. 4.2 showed a very good agreement with experimental data ($R^2=0.99$). Table 4.2 summarized the model parameters for Eq. 4.1.

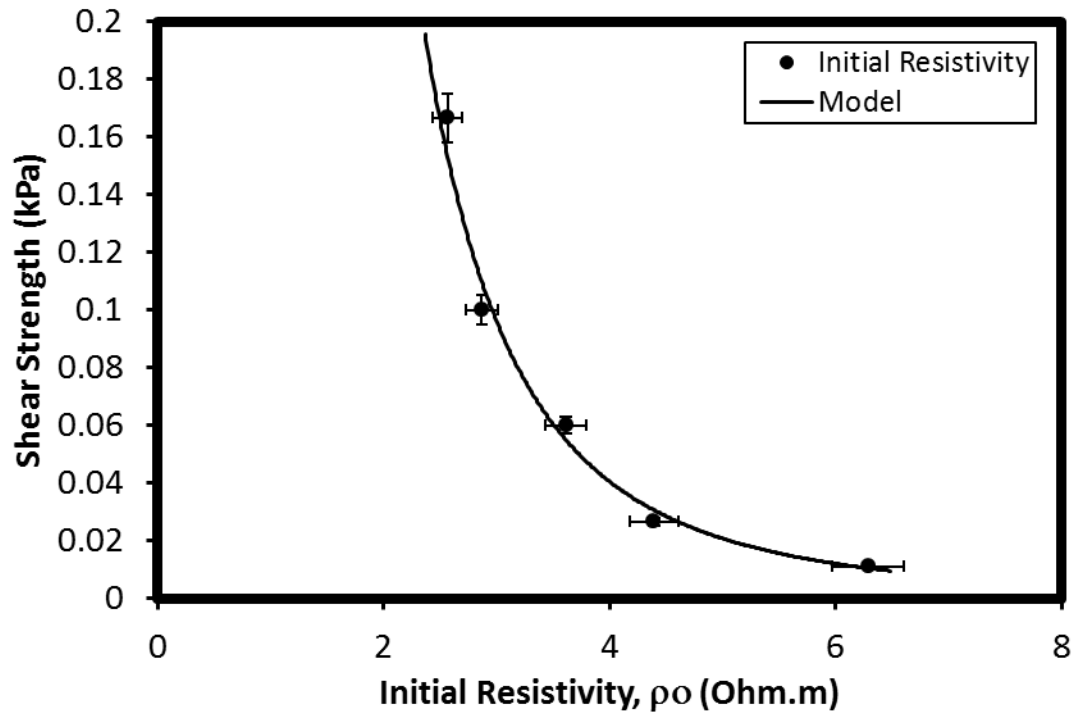


Figure 4.5 Variations of shear strength vs. resistivity of bentonite soft soil.

Table 4.2 Model parameters for Eq. 4.2

C (kPa/(Ohm.m))	D	R ²
3.2	-3.28	0.99

The variation of the soft soil density and water content versus bentonite content has drawn in Figure 4.6. As the bentonite content increased, the water content decreased due to the increasing of the amount of bentonite particles in the same occupied volume. The soft soil density increased from 1.012 gm/cm³ to 1.066 gm/cm³ while the water content decreased from 98% to 90% as the bentonite content increased from 2% to 10%.

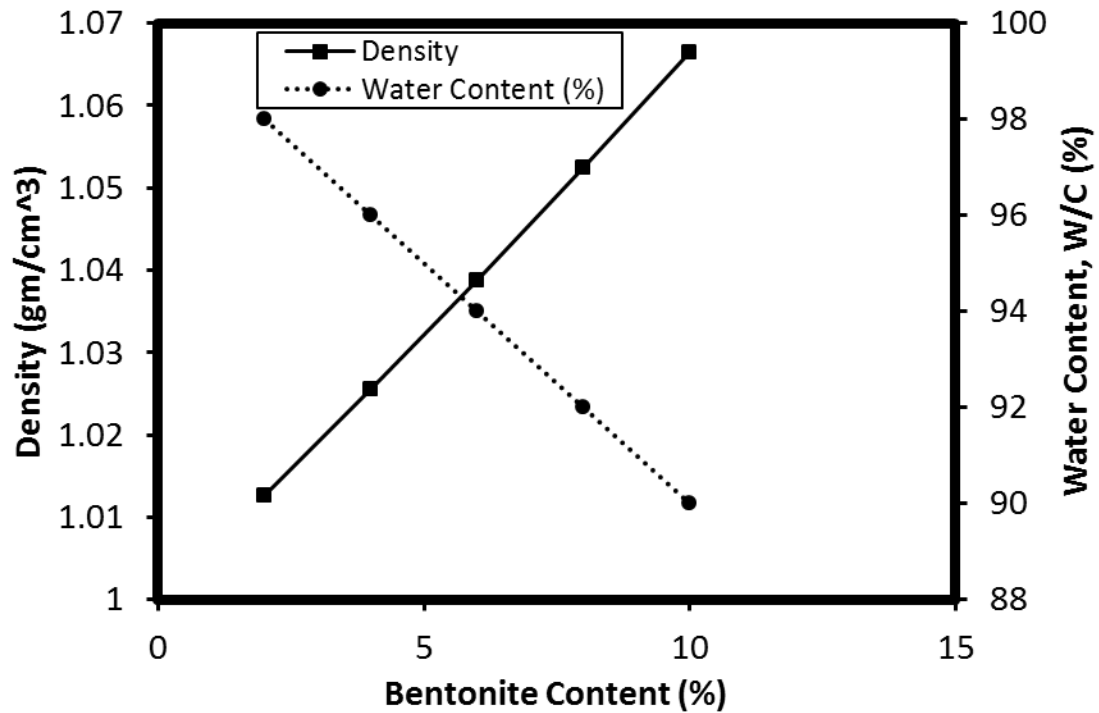


Figure 4.6 The variations of the soft soil density and water content vs. bentonite content.

The variations of water content of soft soil vs. initial resistivity has been studied as shown in Figure 4.7. As the water content increased from 90% to 98%, the initial resistivity increased from 2.56 ± 0.05 Ohm.m to 6.29 ± 0.1 Ohm.m. Water content vs. resistivity has shown a linear relationship and it can be modeled as

$$W/C (\%) = E * (\rho_o) + F, \quad (4.3)$$

where ρ_o is the initial resistivity, E and F are model parameters. Eq. 4.3 has shown a very good prediction with experimental data ($R^2 = 0.91$). Table 4.3 summarized the model parameters for Eq. 4.1. The electrical impedance versus frequency of different soft soil mixture has been measured using LCR device as shown in Figure 4.8. For all the soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the bentonite content increased due to the increase in the conductivity of the medium with higher bentonite content. The drop in the electrical impedance has shown a significant value from 2.67 ± 0.001 k. Ohm to 1.71 ± 0.005 k. Ohm and 1.6 ± 0.003 k. Ohm to 0.607 ± 0.001 k. Ohm at 0.02 kHz and 300 kHz respectively.

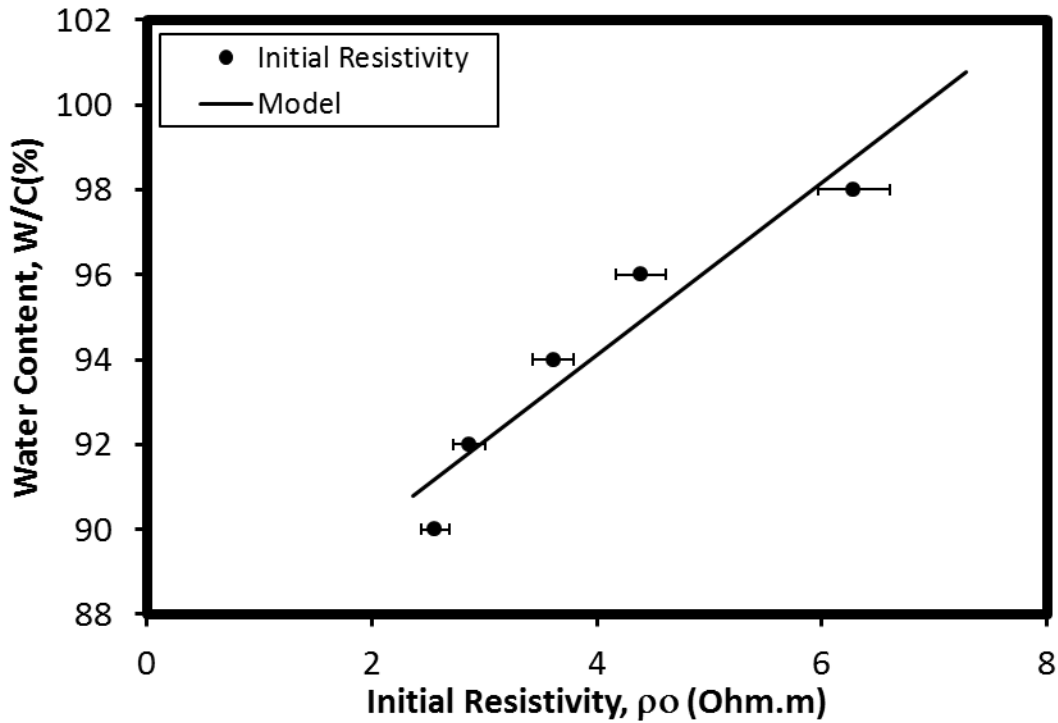


Figure 4.7 The variation of the soft soil density and initial resistivity vs. bentonite content.

Table 4.3 Model parameters for Eq. 4.3

E (1/(Ohm.m))	F	R ²
2.3	86.02	0.91

4.2.2.Kaolinite

Soft soil samples were prepared in laboratory by mixing different percentage of commercially available kaolinite from 2% to 10% with water at room temperature for about ten minutes until homogenous mixture was obtained. The soil slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. Low shear strength measurement is one of the main challenges in the laboratory or even in the field, however; the modified vane shear device has measured the extreme low strength of the prepared kaolinite mud soft soil. In Figure 4.9, both shear strength and water content (percentage of water) of different soft soil vs. kaolinite content is shown. It is clearly shown that the shear strength increased from 0.01 ± 0.0001 kPa to 0.14 ± 0.0002 kPa and the water content decreased from 98% to 90 % as the kaolinite content increased from 2% to 10% respectively. The variations of both initial resistivity and water content of soft soil versus kaolinite content has been studied as shown in Figure 4.10. As water content has decreased from 98% to 90 %, the initial resistivity has decreased from 23.9 Ohm.m to 13 Ohm.m. Due to the increase in the soft soil shear strength with the increase of kaolinite content, the initial resistivity has decreased because the medium became more conductive. Similar model as Eq. 4.1 is used to predict the relationship between the initial resistivity and kaolinite content. The proposed model showed a very good agreement with the experimental data ($R^2=0.93$). Table 4.4 summarized the model parameters similar to Eq. 4.1.

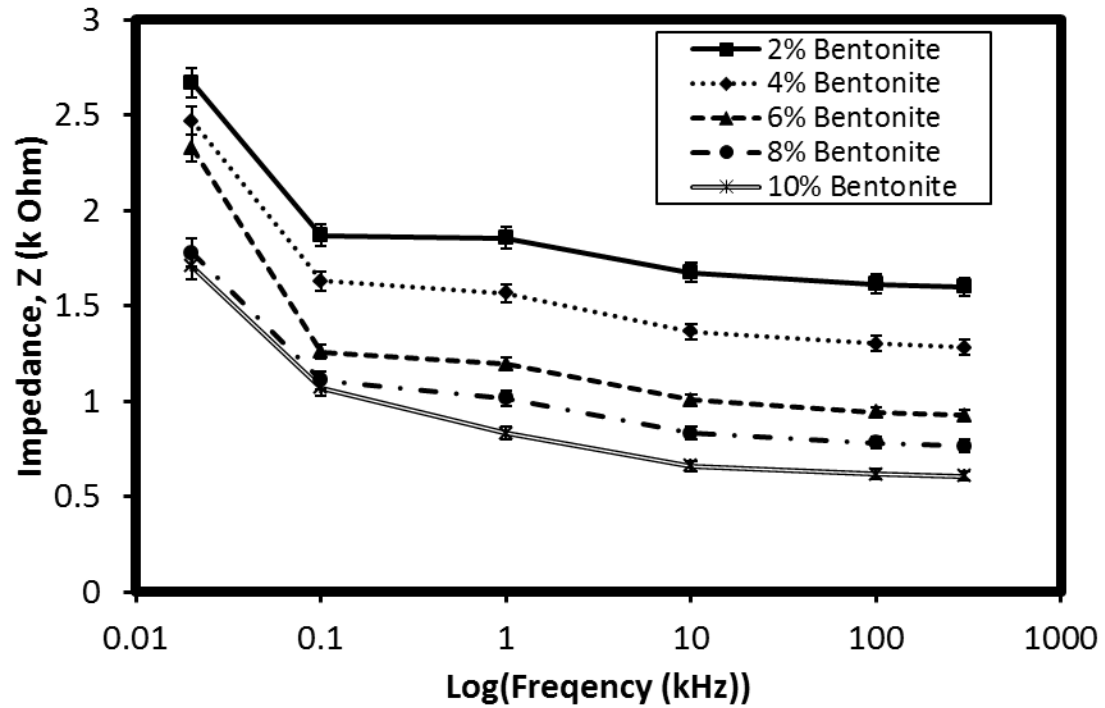


Figure 4.8 Electrical impedance vs. frequency change of different soft soil contents.

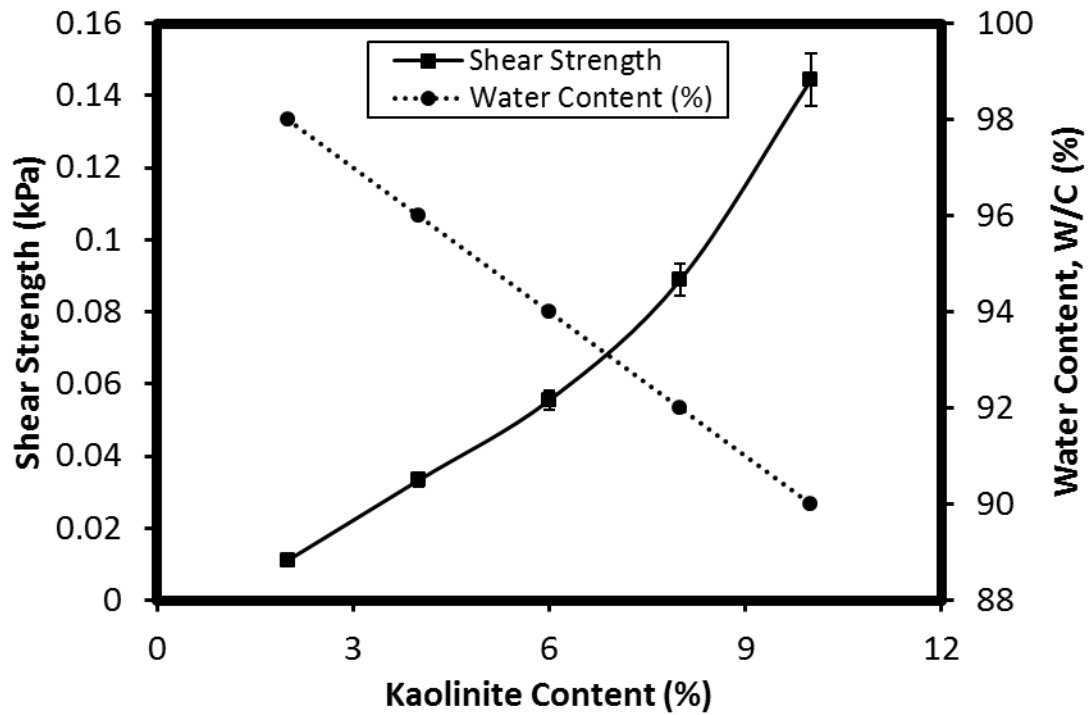


Figure 4.9 Variation of shear strength and water content with kaolinite percent of soft soil.

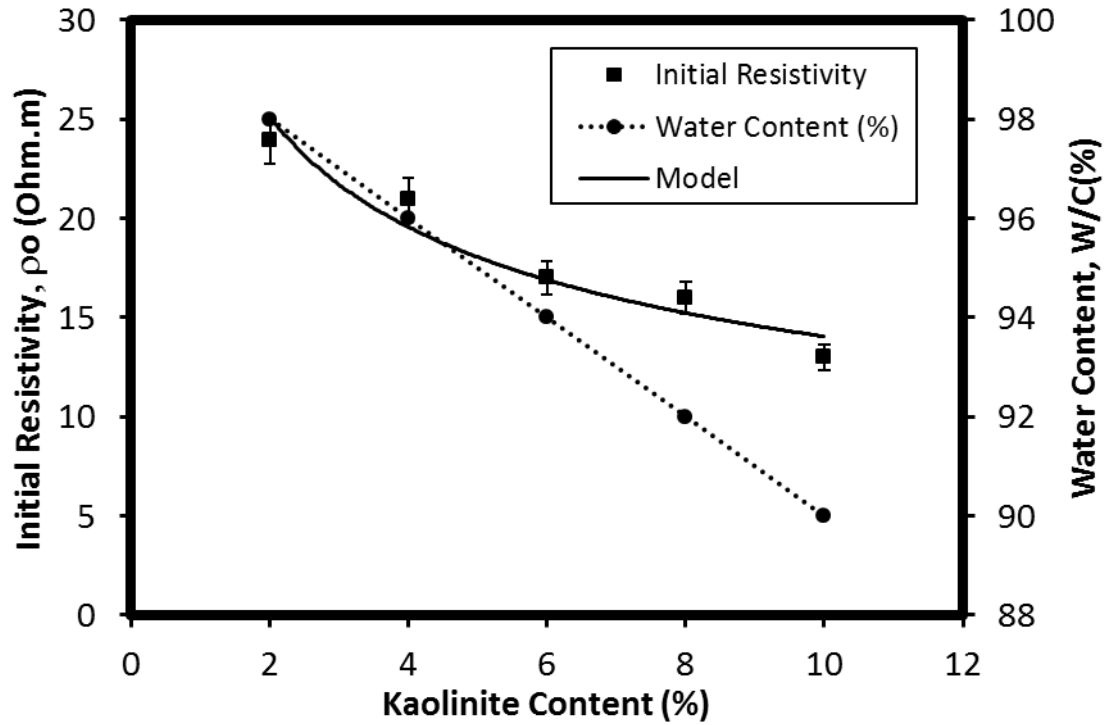


Figure 4.10 Variations of initial resistivity and water content of soft soil vs. kaolinite content.

Table 4.4 Model parameters similar to Eq. 4.1

A (Ohm.m)	B	R ²
32.3	-0.36	0.93

The variations of shear strength of soft soil vs. initial resistivity can be shown in Figure 4.11. It has shown as the shear strength decreased, the initial resistivity increased. Similar model as Eq. 4.2 is used to model the relationship between shear strength and initial resistivity. The model predicted the experimental data with a very good degree of agreement ($R^2=0.94$). Table 4.5 summarized the model parameters similar to Eq. 4.2.

The variation of the soft soil density and water content versus kaolinite percentage has drawn in Figure 4.12. As the kaolinite content increased, the water content decreased due to the increasing of the amount of kaolinite particles in the same occupied volume.

The soft soil density increased from 1.011 gm/cm³ to 1.065 gm/cm³ while the water content decreased from 98% to 90% as the kaolinite content increased from 2% to 10%.

The variations of water content of soft soil vs. initial resistivity have been studied as shown in Figure 4.13. As the water content increased from 98% to 90%, the initial resistivity increased from 13 Ohm.m to 23.9 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.96$). Table 4.6 summarized the model parameters similar to Eq. 4.3.

Table 4.5 Model parameters similar to Eq. 4.2

C (kPa/(Ohm.m))	D	R^2
5054	-4.02	0.94

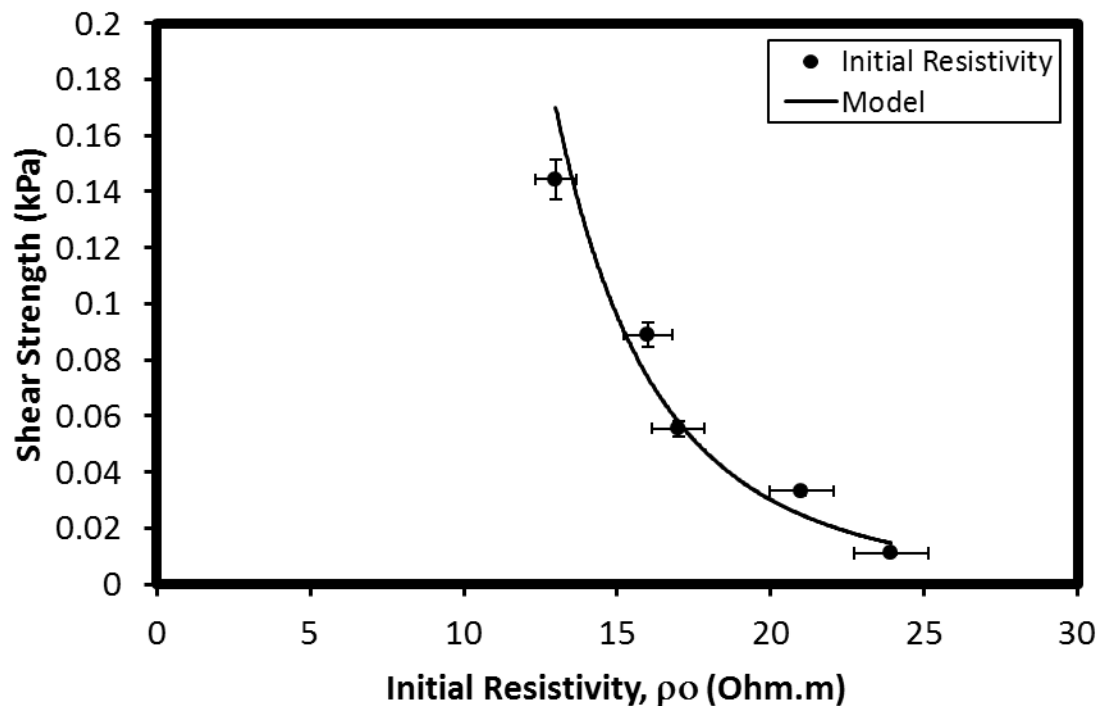


Figure 4.11 Variations of shear strength vs. resistivity of kaolinite soft soil.

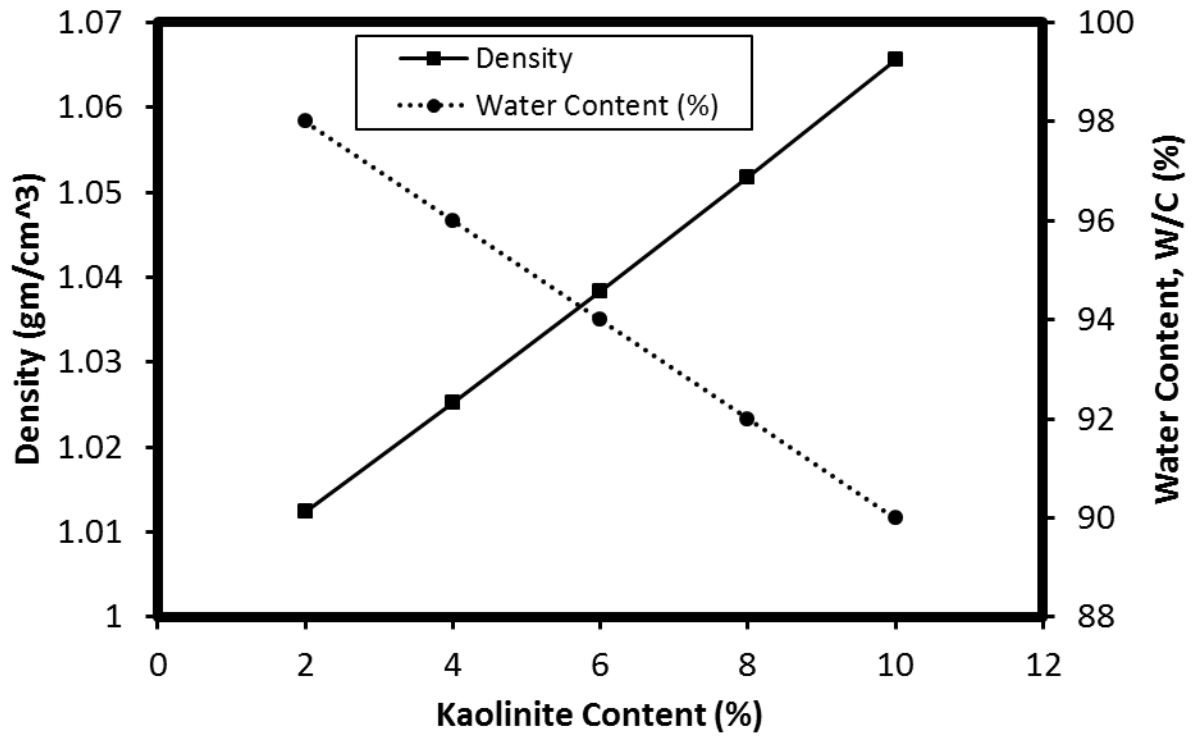


Figure 4.12 The variations of the soft soil density and water content vs. kaolinite content.

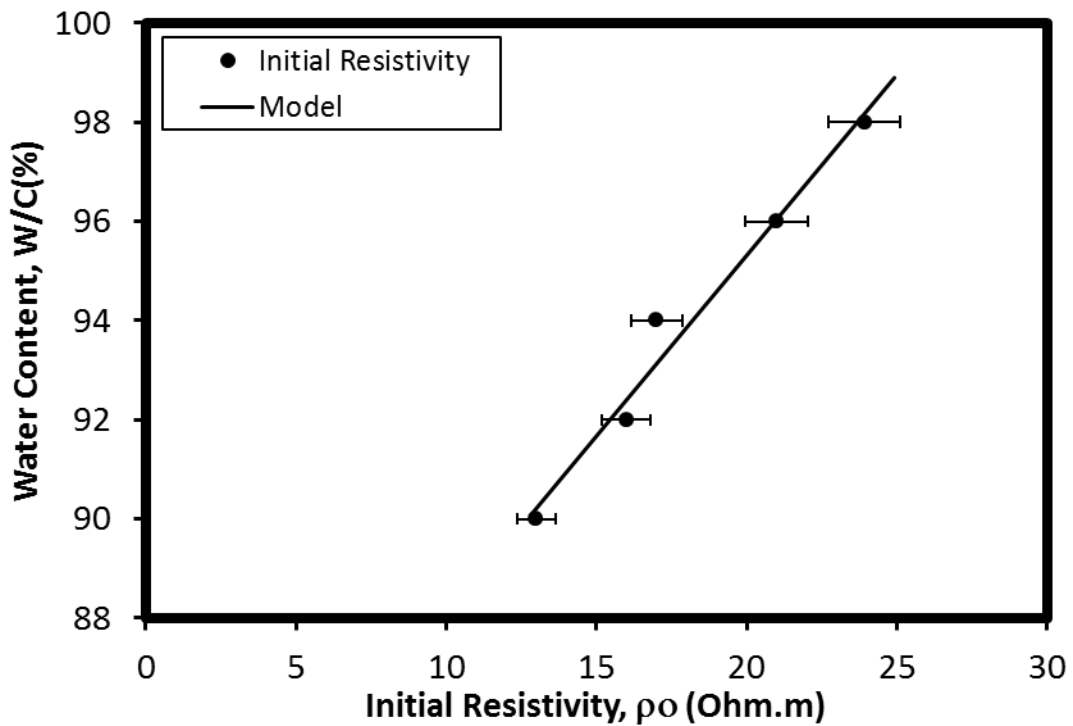
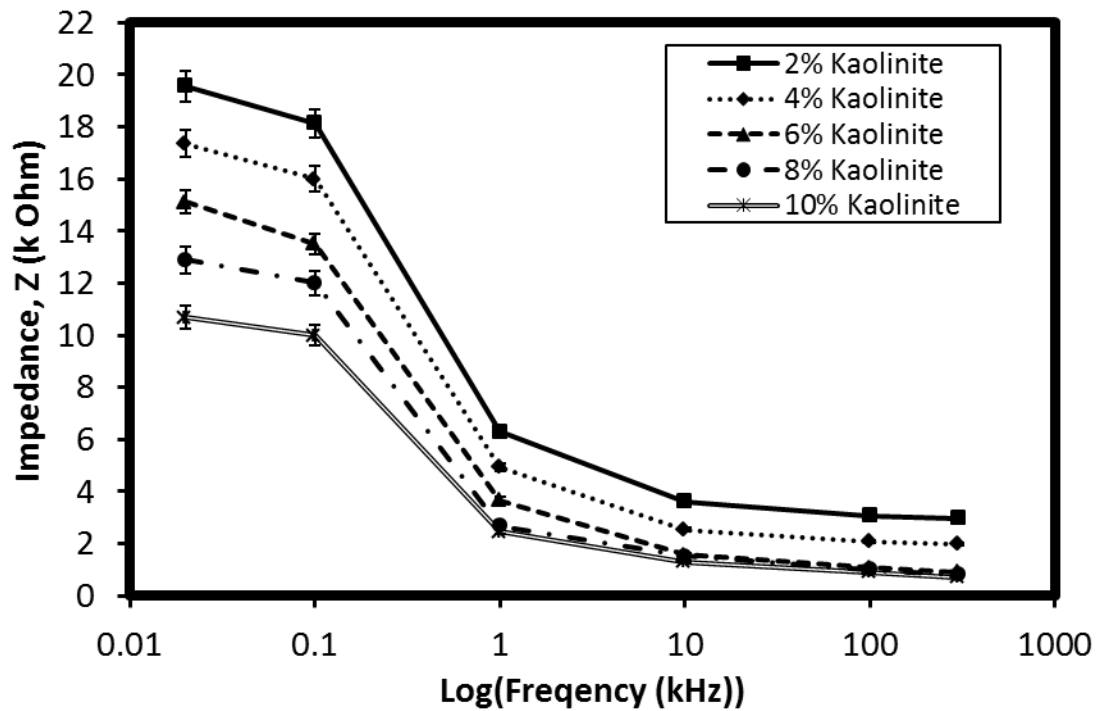


Figure 4.13 The variation of the soft soil density and initial resistivity vs. bentonite content.

Table 4.6 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R ²
0.73	80.8	0.98

The electrical impedance versus frequency of different soft soil mixture has been measured using LCR device as shown in Figure 4.14. For all the soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the kaolinite content increased due to the increase in the conductivity of the medium with higher kaolinite content. The drop in the electrical impedance has shown a significant value from 19.56 ± 0.001 k.Ohm to 10.68 ± 0.0005 k.Ohm and 2.97 ± 0.0003 k.Ohm to 0.71 ± 0.0002 k.Ohm at 0.02 kHz and 300 kHz respectively.

**Figure 4.14 Electrical impedance vs. frequency change of different soft soil contents.**

4.3. Soft soil improvement

4.3.1. Lime

Soft soil improvement samples were prepared in the laboratory by adding different percentage of lime from 2% to 10% with 10% bentonite soil then mix the soft soil for about ten minutes at room temperature until homogenous mixture was obtained. The soft soil slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. The modified vane shear device was used to measure the extreme low strength of the prepared mud soft soil. In Figure 4.15, both shear strength and water content of 10% bentonite treated soft soil with lime vs. lime content is shown. It is clearly shown that both shear strength and water content decreased from 0.17 ± 0.001 kPa to 0.15 ± 0.001 kPa and from 90% to 80 % as the lime content increased from 0% to 10% respectively with an optimum shear strength of 0.27 kPa at 2% lime treatment. The variations of both initial resistivity and water content of 10% bentonite treated soft soil with lime vs. lime content has been studied as shown in Figure 4.16. As water content has decreased from 90% to 80 %, the initial resistivity has decreased from 0.65 ± 0.0001 Ohm.m to 0.38 ± 0.00005 Ohm.m. Due to the decrease in the soft soil shear strength with the increase of lime content, the initial resistivity has decreased because the medium became more conductive. Similar model as Eq. 4.1 is used to predict the relationship between the initial resistivity and lime content. The proposed model showed a very good agreement with the experimental data ($R^2=0.97$). Table 4.7 summarized the model parameters similar to Eq. 4.1.

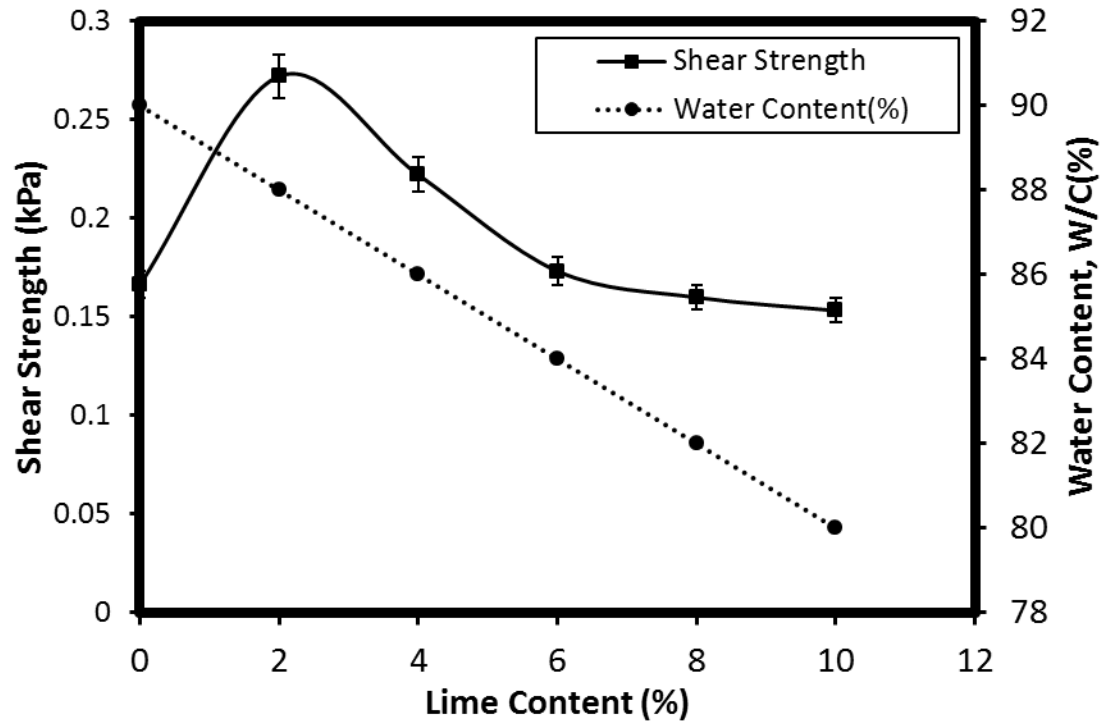


Figure 4.15 Variation of shear strength and water content of 10% bentonite treated soft soil with lime.

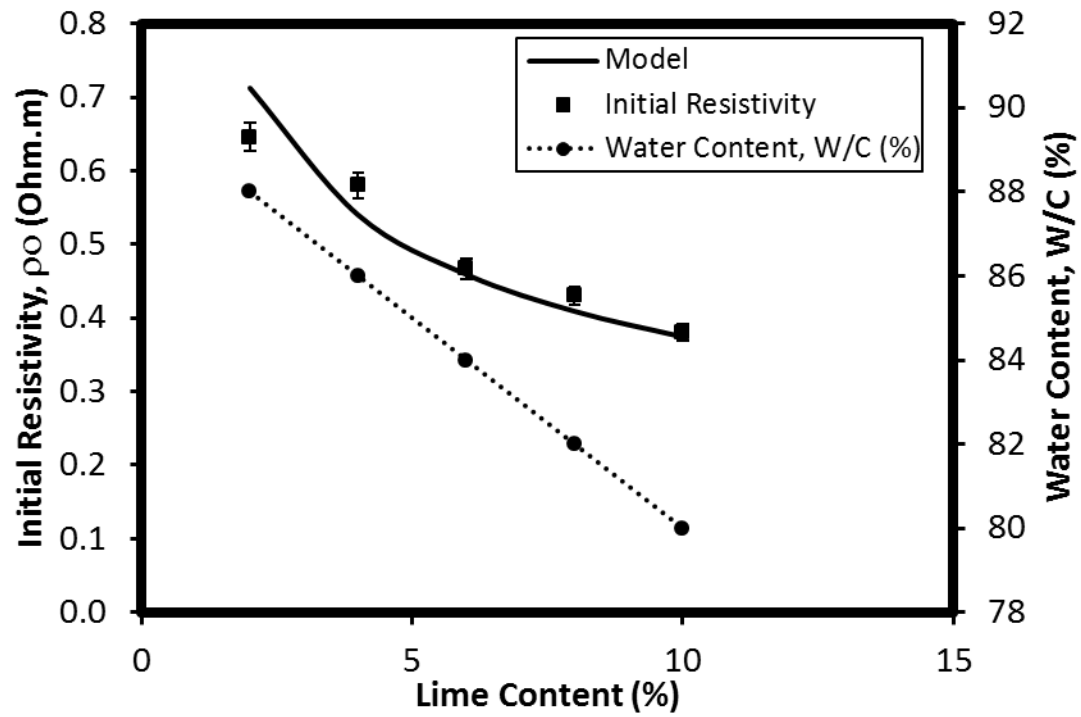


Figure 4.16 Variations of initial resistivity and water content of 10% bentonite treated soft soil with lime.

Table 4.7 Model parameters similar to Eq. 4.1

A (Ohm.m)	B	R ²
0.94	-0.4	0.97

The variations of shear strength of 10% bentonite treated soft soil with lime vs. initial resistivity can be shown in Figure 4.17. It has shown as the shear strength increased, the initial resistivity increased too. A linear model can be used to predict the relationship between the shear strength and electrical resistivity as

$$\text{Shear Strength} = G + H * \rho_o, \quad (4.4)$$

where ρ_o is the initial electrical resistivity, G and H are model parameters.

The proposed model predicted the experimental data with a very good degree of agreement ($R^2=0.96$). Table 4.8 summarized the model parameters for Eq. 4.4.

The variation of the soft soil density and water content of 10% bentonite soft soil treated with lime versus lime content has drawn in Figure 4.18. As the lime content increased, the water content decreased due to the increasing of the amount of lime particles in the same occupied volume. The soft soil density increased from 1.024 gm/cm³ to 1.13 gm/cm³ while the water content decreased from 90% to 80% as the lime content increased from 2% to 10%.

Table 4.8 Model parameters for Eq. 4.4

G (kPa)	H (kPa/(Ohm.m))	R ²
0.45	-0.029	0.96

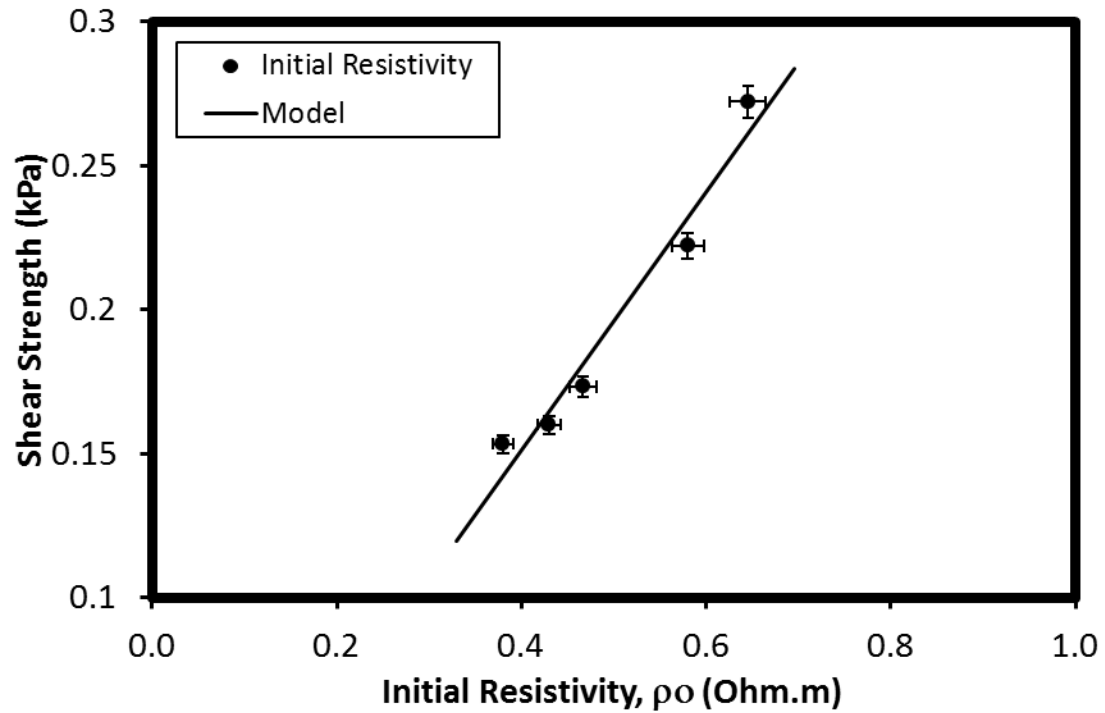


Figure 4.17 Variations of shear strength vs. resistivity of 10% bentonite treated soft soil with lime.

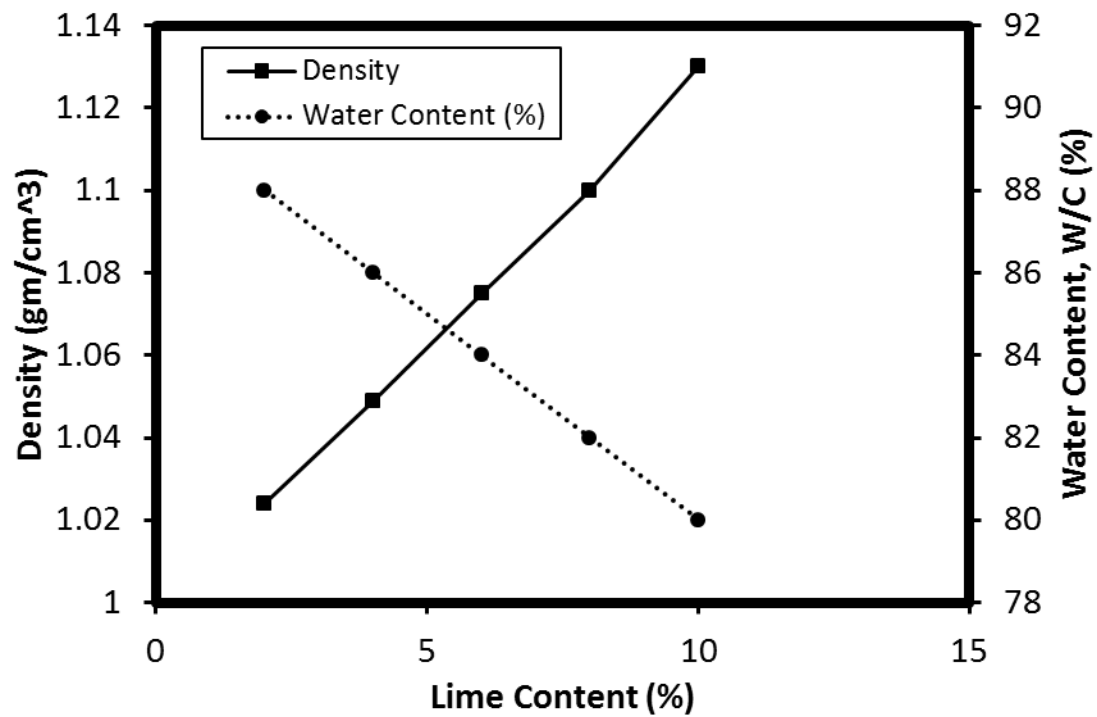


Figure 4.18 The variations of the soft soil density and water content of 10% bentonite treated soft soil with lime vs. lime content.

The variations of water content vs. initial resistivity of 10% bentonite treated soft soil with lime have been studied as shown in Figure 4.19. As the water content increased from 80% to 90%, the initial resistivity increased from 0.38 ± 0.00005 Ohm.m to 0.65 ± 0.0001 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.97$). Table 4.9 summarized the model parameters for Eq. 4.4.

The electrical impedance versus frequency of 10% bentonite soft soil treated with different lime contents has been measured using LCR device as shown in Figure 4.20. For all the different soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the lime content increased due to the increase in the conductivity of the medium with higher lime content. The drop in the electrical impedance has shown a significant value from 0.707 k.Ohm to 0.414 k.Ohm and 0.28 k.Ohm to 0.16 k.Ohm at 0.02 kHz and 300 kHz respectively.

Table 4.9 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R^2
28.4	69.8	0.97

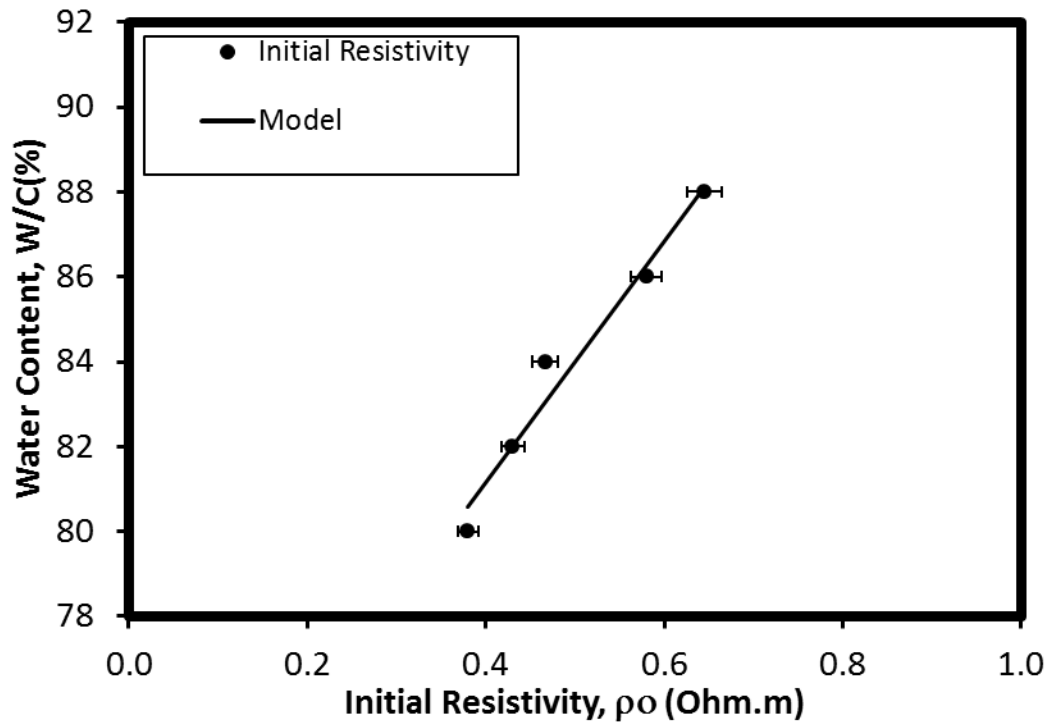


Figure 4.19 The variations of the soft water content of 10% bentonite treated soft soil with lime vs. initial resistivity.

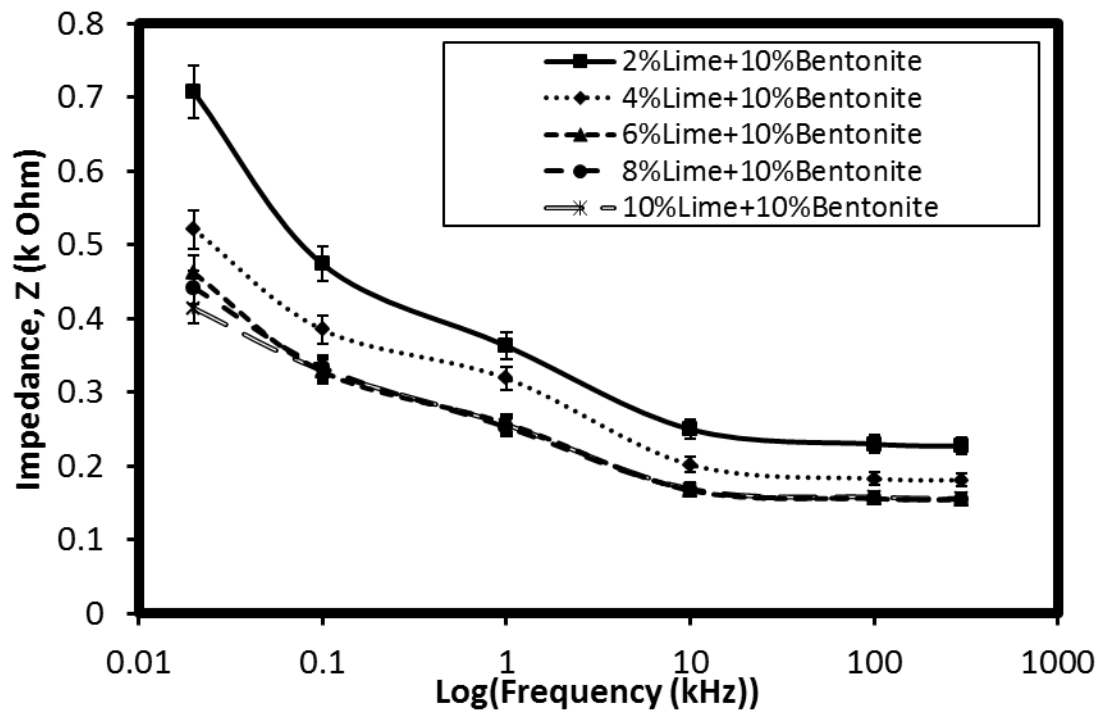


Figure 4.20 Electrical impedance vs. frequency change of 10% bentonite treated soft soil with lime for different lime contents.

4.3.2.Polymer

Soft soil improvement samples were prepared in the laboratory by adding different percentage of acrylamide polymer from 1% to 10% with the 10% bentonite soil at room temperature. The soft soil slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. The modified vane shear device was used to measure the extreme low strength of the prepared mud soft soil. In Figure 4.21, both shear strength and water content of 10% bentonite treated soft soil with polymer vs. polymer content is shown. It is clearly shown that the shear strength increased from 0.17 ± 0.001 kPa to 6.7 ± 0.002 kPa and water content decreased from 90% to 80% as the polymer content increased from 0% to 10% respectively.

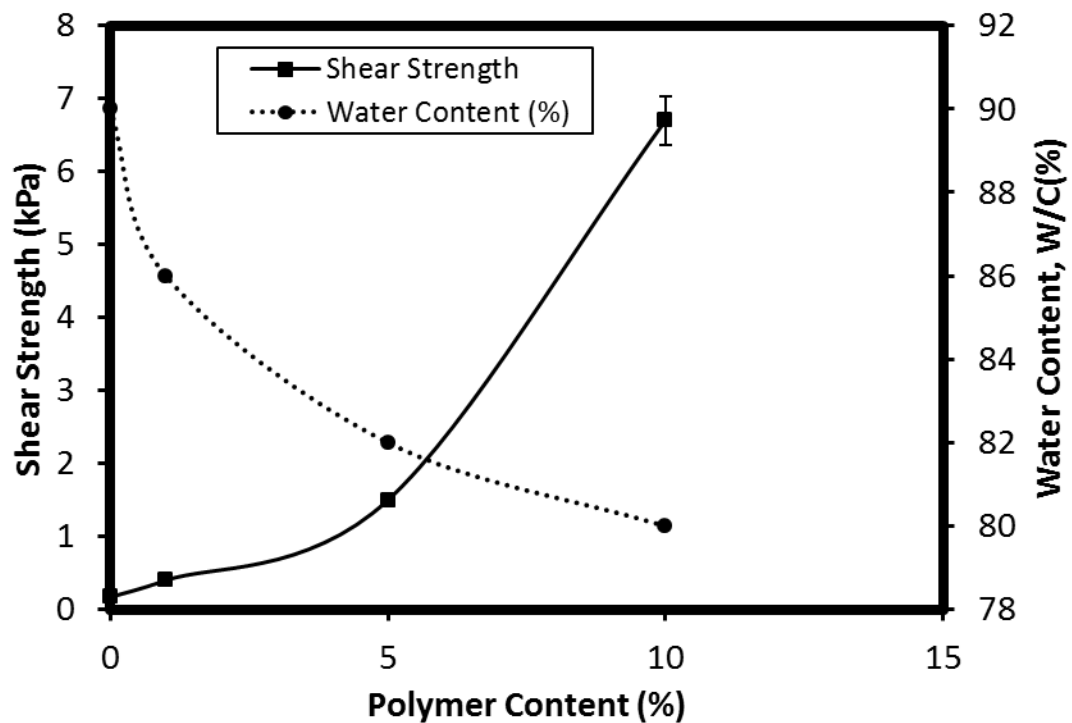


Figure 4.21 Variation of shear strength and water content of 10% bentonite treated soft soil with polymer.

The variations of both initial resistivity and water content of 10% bentonite treated soft soil with polymer vs. polymer content has been studied as shown in Figure 4.22. As water content has decreased from 90% to 80 %, the initial resistivity has decreased from 2.56 ± 0.0001 Ohm.m to 0.47 ± 0.00005 Ohm.m. Due to the increase in the soft soil shear strength with the increase of polymer content, the initial resistivity has decreased because the medium became more conductive. Similar model as Eq. 4.1 is used to predict the relationship between the initial resistivity and polymer content. The proposed model showed a very good agreement with the experimental data ($R^2=0.98$). Table 4.10 summarized the model parameters for Eq. 4.4.

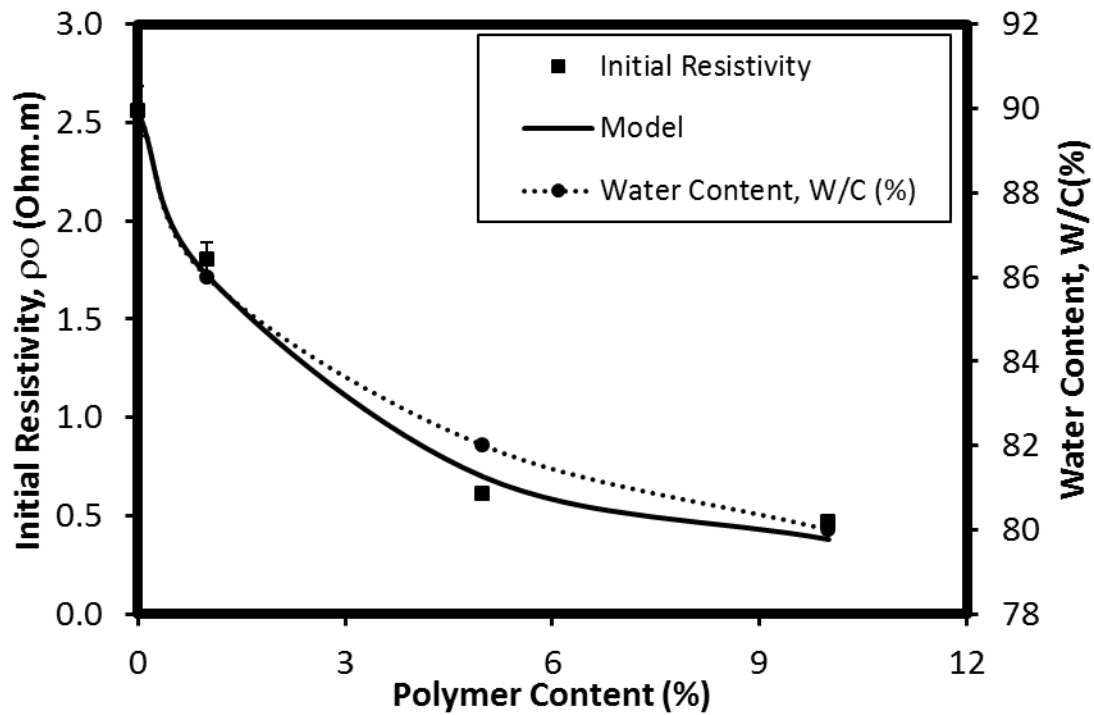


Figure 4.22 Variations of initial resistivity and water content of 10% bentonite treated soft soil with polymer.

Table 4.10 Model parameters similar to Eq. 4.1

A (Ohm.m)	B	R ²
6	-1.2	0.98

The variations of shear strength of 10% bentonite treated soft soil with polymer vs. initial resistivity can be shown in Figure 4.23. It has shown as the shear strength increased, the initial resistivity decreased. A hyperbolic model can be used to predict the relationship between the shear strength and electrical resistivity as

$$\text{shear strength} - (\text{shear strength})_o = \frac{(\rho - \rho_o)}{V + W * (\rho - \rho_o)}, \quad (4.5)$$

where ρ_o is the initial electrical resistivity, V and W are model parameters. The proposed model predicted the experimental data with a very good degree of agreement ($R^2=0.99$).

Table 4.11 summarized the model parameters for Eq. 4.5.

The variation of the soft soil density and water content of 10% bentonite soft soil treated with polymer versus polymer content has drawn in Figure 4.24. As the polymer content increased, the water content decreased due to the increasing of the amount of polymer particles in the same occupied volume. The soft soil density increased from 1.092 gm/cm³ to 1.16 gm/cm³ while the water content decreased from 90% to 80% as the polymer content increased from 0% to 10%.

Table 4.11 Model parameters for Eq. 4.5

(Shear Strength) _o	ρ_o (Ohm.m)	V (Ohm.m)	W	R ²
0.17	0.4	-0.005	-0.146	0.99

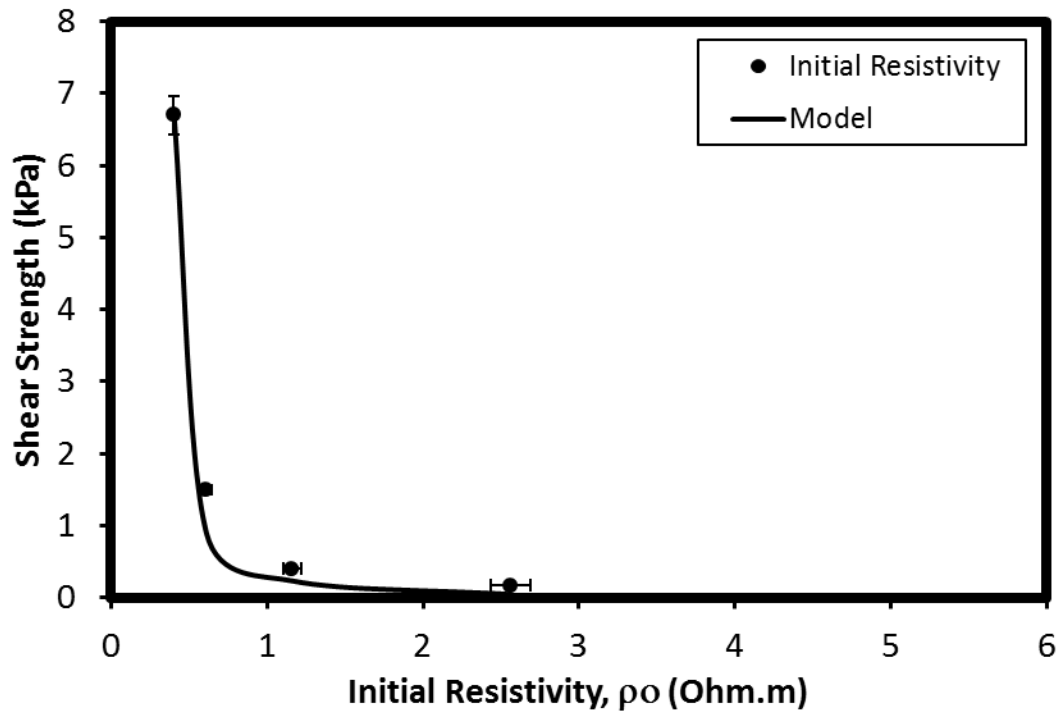


Figure 4.23 Variations of shear strength vs. resistivity of 10% bentonite treated soft soil with polymer.

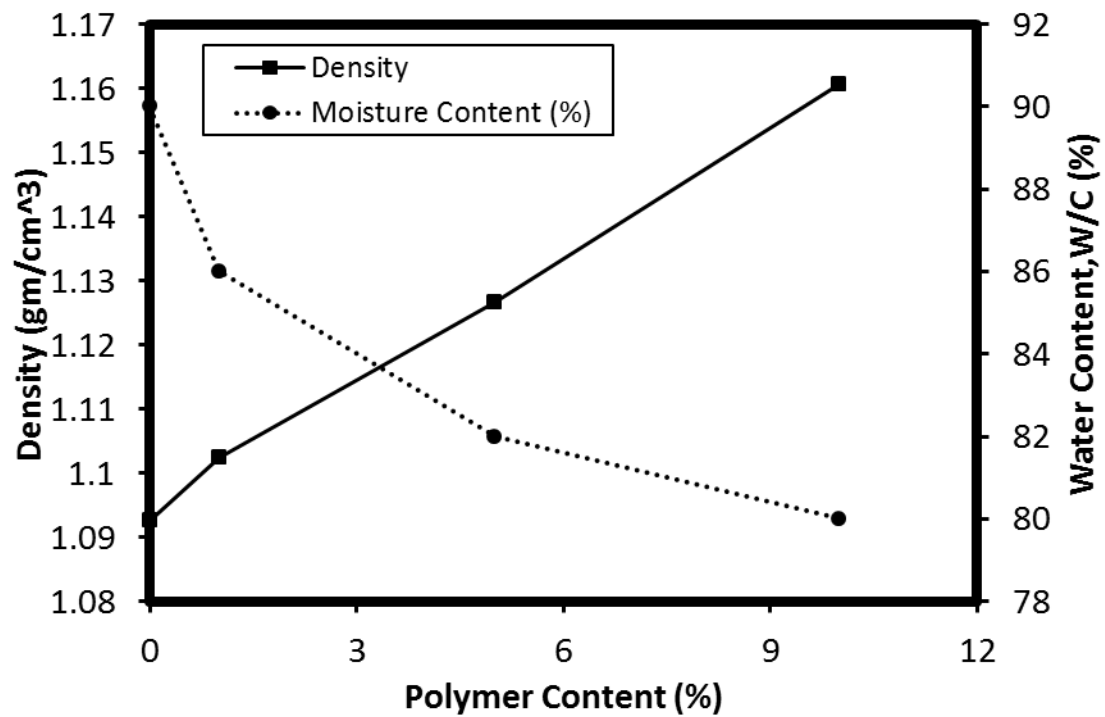


Figure 4.24 The variations of the soft soil density and water content of 10% bentonite treated soft soil with polymer vs. polymer content.

The variations of water content vs. initial resistivity of 10% bentonite treated soft soil with polymer have been studied as shown in Figure 4.25. As the water content increased from 80% to 90%, the initial resistivity increased from 0.4 ± 0.00005 Ohm.m to 2.56 ± 0.0001 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.93$). Table 4.12 summarized the model parameters similar to Eq. 4.3.

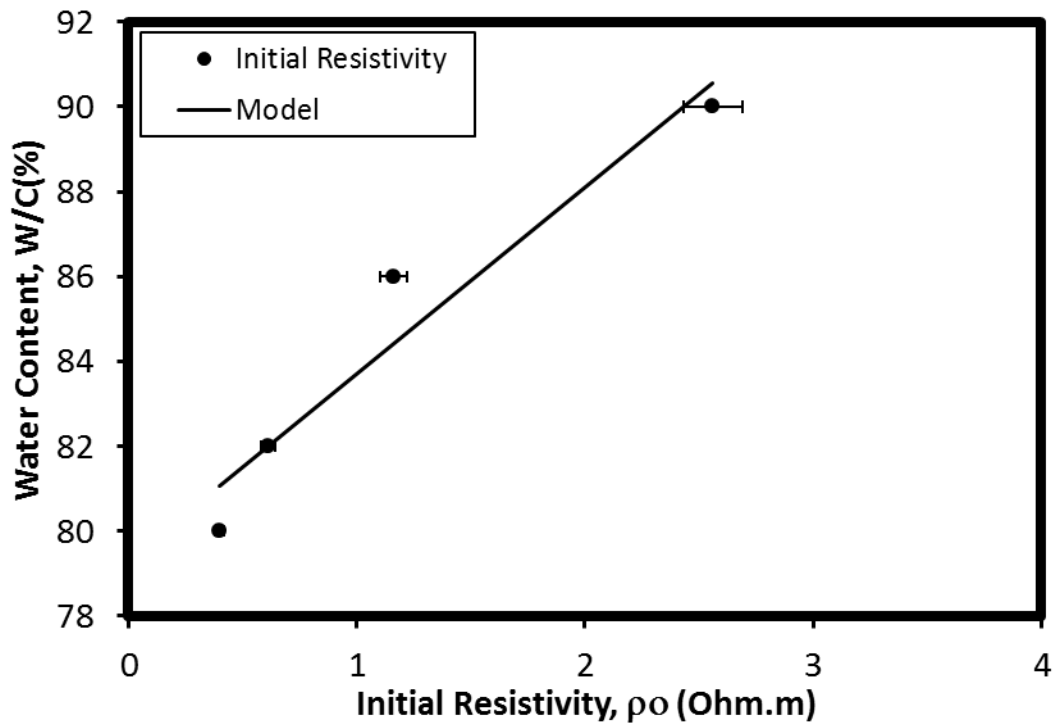


Figure 4.25 The variations of the soft water content of 10% bentonite treated soft soil with polymer vs. initial resistivity.

Table 4.12 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R ²
4.4	79.3	0.93

The electrical impedance versus frequency of 10% bentonite soft soil treated with different polymer contents has been measured using LCR device as shown in Figure 4.26. For all the different soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the polymer content increased due to the increase in the conductivity of the medium with higher polymer content. The drop in the electrical impedance has shown a significant value from 1.71 ± 0.001 k. Ohm to 0.394 ± 0.00005 k. Ohm and 0.607 ± 0.00003 k. Ohm to 0.176 ± 0.00001 k. Ohm at 0.02 kHz and 300 kHz respectively.

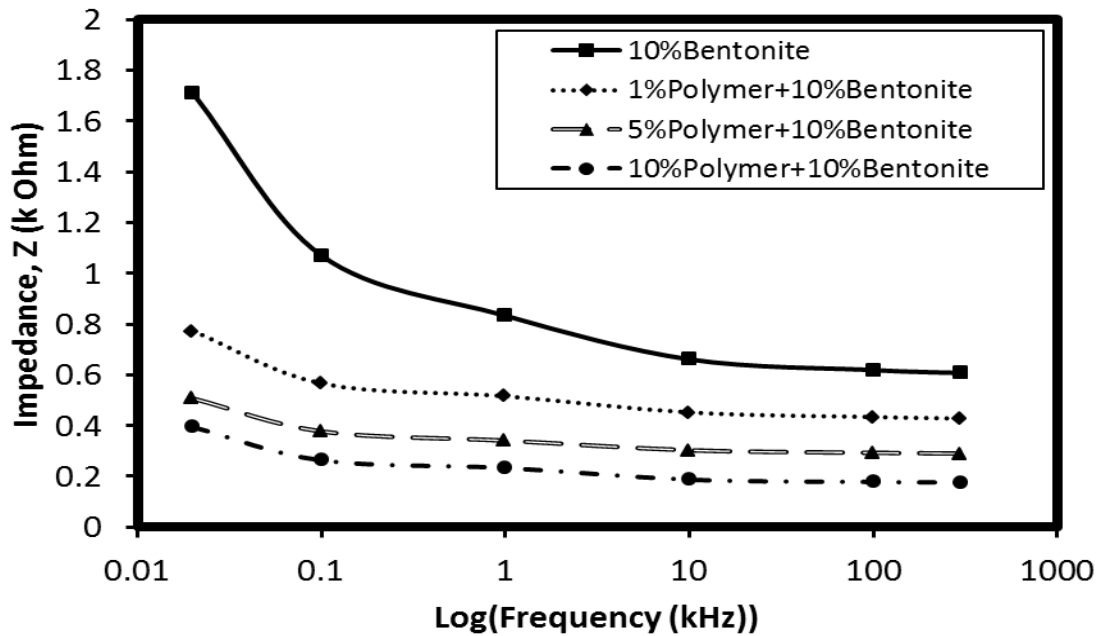


Figure 4.26 Electrical impedance vs. frequency change of 10% bentonite treated soft soil with polymer for different polymer contents.

4.3.3.Sand

Soft soil improvement samples were prepared in the laboratory by adding different percentage of fine sand from 1% to 3% with the 10% bentonite soil at room temperature and mixed for about five minutes. The soft soil slurry mixture was placed in 50 mm (dia.)

x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. The modified vane shear device was used to measure the extreme low strength of the prepared mud soft soil. In Figure 4.27, both shear strength and water content of 10% bentonite treated soft soil with sand vs. sand content is shown. It is clearly shown that both shear strength and water content decreased from 0.17 ± 0.001 kPa to 0.055 ± 0.0005 kPa and water content decreased from 90% to 87% as the sand content increased from % to 3% respectively.

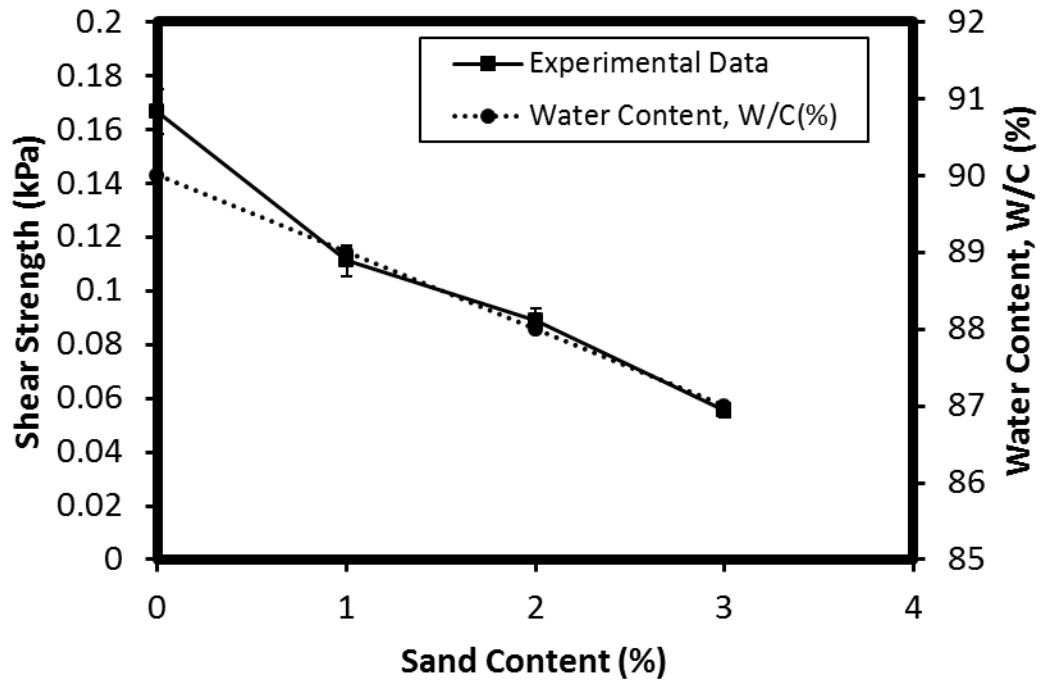


Figure 4.27 Variation of shear strength and water content of 10% bentonite treated soft soil with sand.

The variations of both initial resistivity and water content of 10% bentonite treated soft soil with sand vs. sand content has been studied as shown in Figure 4.28. As water content has decreased from 89% to 87 %, the initial resistivity has increased from 4.5 ± 0.001 Ohm.m to 5.55 ± 0.0005 Ohm.m. Due to the decrease in the soft soil shear

strength with the increase of sand content, the initial resistivity has increased because the medium became less conductive. Similar model as Eq. 4.1 is used to predict the relationship between the initial resistivity and sand content. The proposed model showed a very good agreement with the experimental data ($R^2=0.98$). Table 4.13 summarized the model parameters similar to Eq. 4.1.

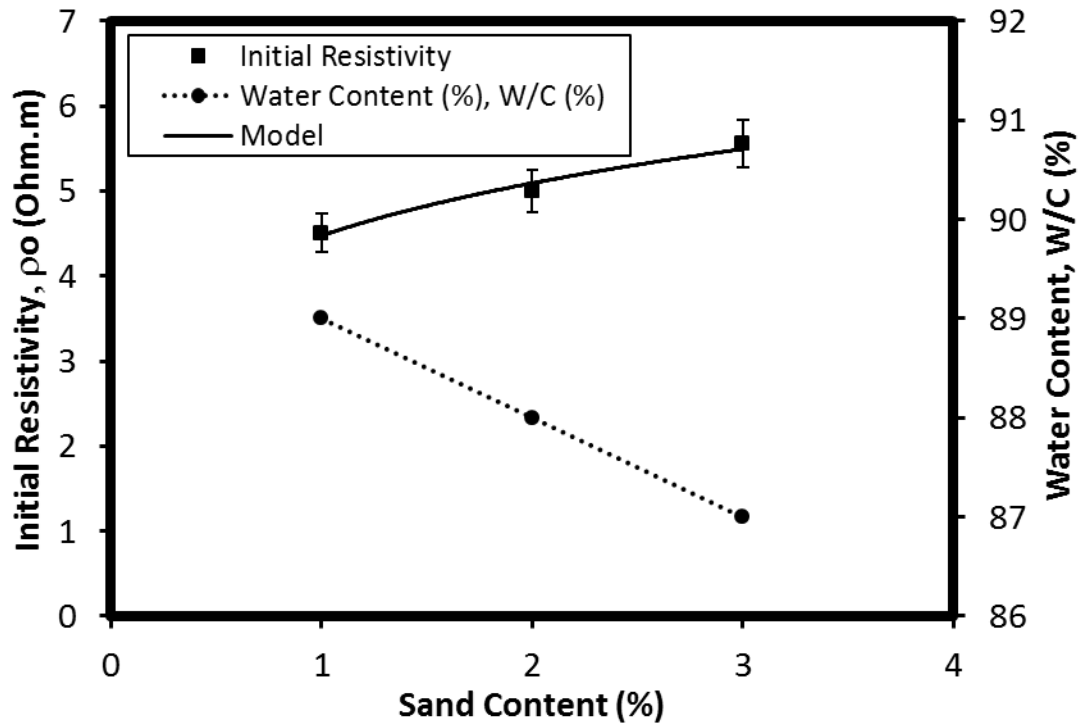


Figure 4.28 Variations of initial resistivity and water content of 10% bentonite treated soft soil with sand.

Table 4.13 Model parameters similar to Eq. 4.1

A (Ohm.m)	B	R^2
4.5	-0.19	0.98

The variations of shear strength of 10% bentonite treated soft soil with sand vs. initial resistivity can be shown in Figure 4.29. It has shown as the shear strength increased, the

initial resistivity decreased. A linear model can be used to predict the relationship between the shear strength and electrical resistivity as

$$\text{shear strength} = V_1 + W_1 * \rho_o, \quad (4.6)$$

where ρ_o is the initial electrical resistivity, V_1 and W_1 are model parameters. The proposed model predicted the experimental data with a very good degree of agreement ($R^2=0.99$). Table 4.14 summarized the model parameters for Eq. 4.6.

The variation of the soft soil density and water content of 10% bentonite soft soil treated with sand versus sand content has drawn in Figure 4.30. As the sand content increased, the water content decreased due to the increasing of the amount of sand particles in the same occupied volume. The soft soil density increased from 1.073 gm/cm³ to 1.085 gm/cm³ while the water content decreased from 89% to 87% as the sand content increased from 0% to 3%.

Table 4.14 Model parameters for Eq. 4.6

V_1 (kPa)	W_1 (kPa/(Ohm.m))	R^2
-0.05	0.35	0.99

The variations of water content vs. initial resistivity of 10% bentonite treated soft soil with sand have been studied as shown in Figure 4.31. As the water content decreased from 89% to 87%, the initial resistivity increased from 4.5 ± 0.001 Ohm.m to 5.55 ± 0.002 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.99$). Table 4.15 summarized the model parameters for Eq. 4.6.

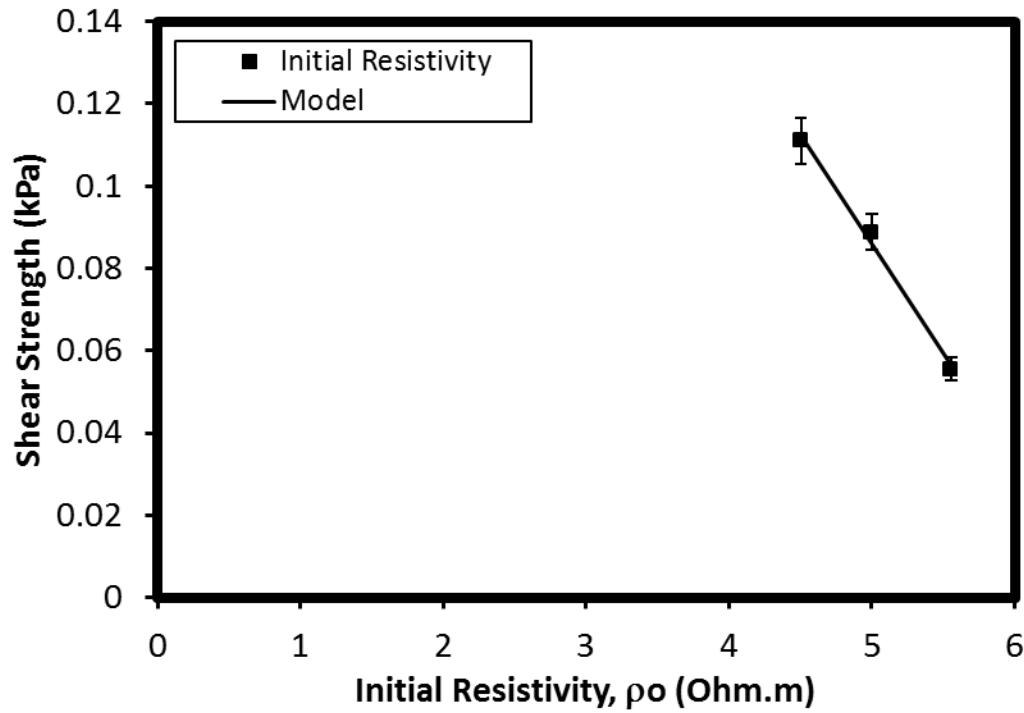


Figure 4.29 Variations of shear strength vs. resistivity of 10% bentonite treated soft soil with sand.

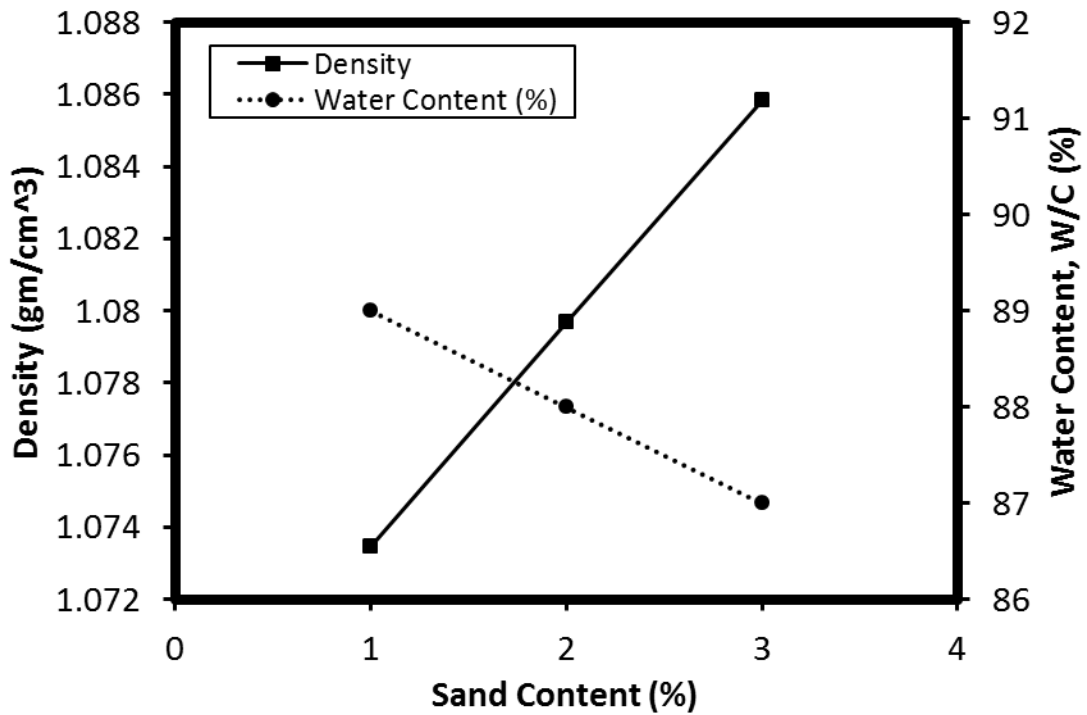


Figure 4.30 The variations of the soft soil density and water content of 10% bentonite treated soft soil with sand vs. sand content.

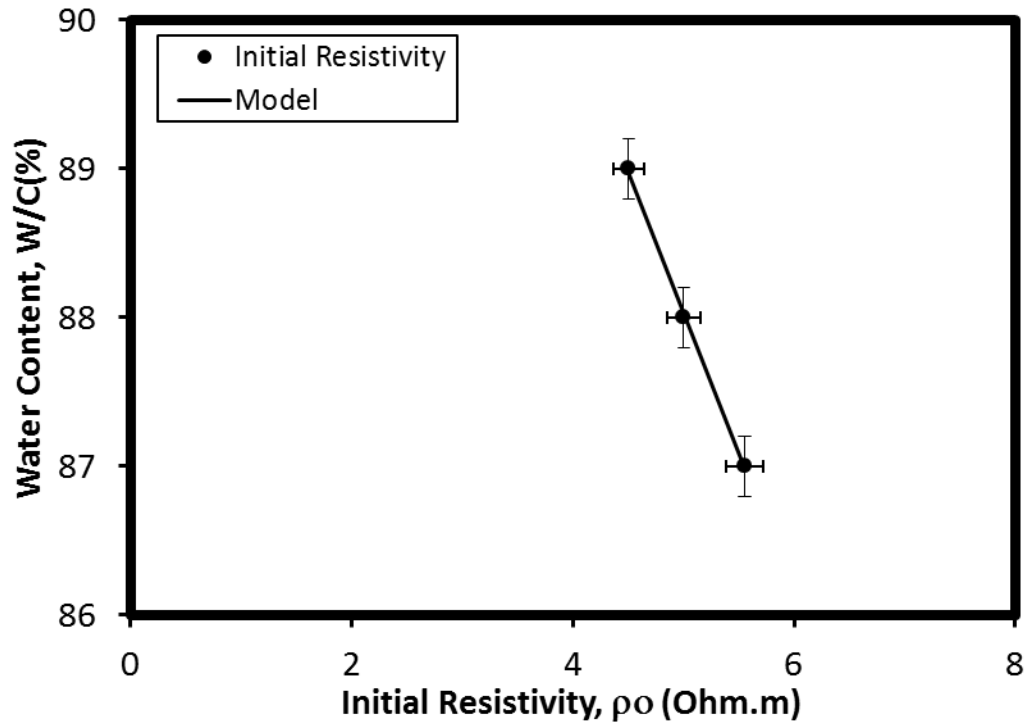


Figure 4.31 The variations of the soft water content of 10% bentonite treated soft soil with sand vs. initial resistivity.

Table 4.15 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R ²
-1.9	97.5	0.99

The electrical impedance versus frequency of 10% bentonite soft soil treated with different sand contents has been measured using LCR device as shown in Figure 4.32. For all the different soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the sand content increased due to the increase in the conductivity of the medium with higher sand content. The drop in the electrical

impedance has shown a significant value from 1.71 ± 0.001 k.Ohm to 0.829 ± 0.001 k.Ohm and 0.607 ± 0.0005 k.Ohm to 0.421 ± 0.0005 k.Ohm at 0.02 kHz and 300 kHz respectively.

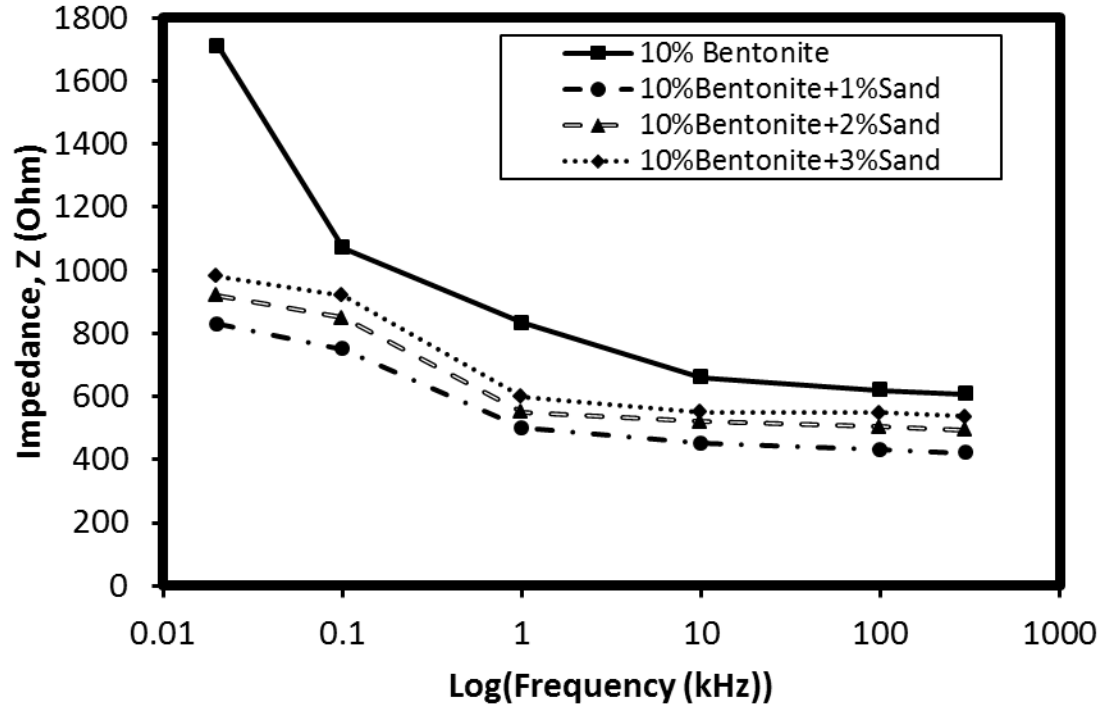


Figure 4.32 Electrical impedance vs. frequency change of 10% bentonite treated soft soil with sand for different sand contents.

4.3.4.Cement

Soft soil improvement samples were prepared in the laboratory by adding different percentage of cement from 1% to 10% with the 10% bentonite soil at room temperature and mixed for about ten minutes. The soft soil slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. The modified vane shear device was used to measure the extreme low strength of the prepared mud soft soil. In Figure 4.33, both shear strength and water content of 10% bentonite treated soft soil with cement vs. cement content is shown. It is clearly shown that the shear strength increased from

0.17±0.001 kPa to 0.233±0.0005 kPa and water content decreased from 90% to 80% as the cement content increased from 0% to 10% respectively.

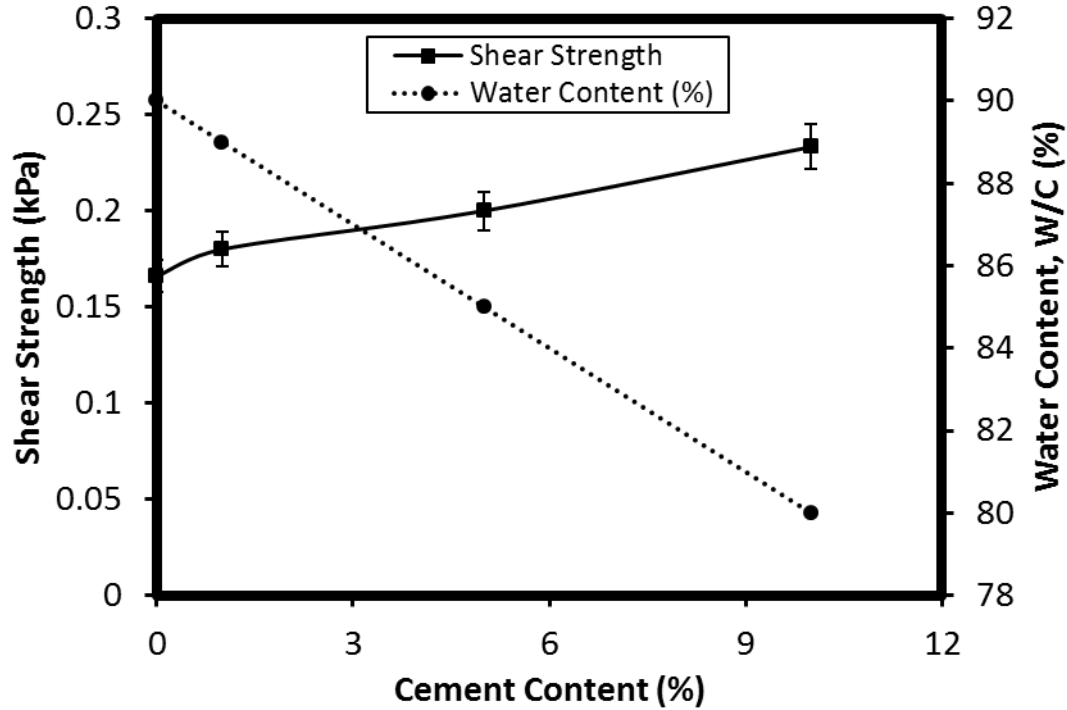


Figure 4.33 Variation of shear strength and water content of 10% bentonite treated soft soil with cement.

The variations of both initial resistivity and water content of 10% bentonite treated soft soil with cement vs. cement content has been studied as shown in Figure 4.34. As water content has decreased from 89% to 80 %, the initial resistivity has decreased from 2.1±0.001 Ohm.m to 0.7±0.0006 Ohm.m. Due to the increase in the soft soil shear strength with the increase of cement content, the initial resistivity has decreased because the medium became more conductive. A linear model is used to predict the relationship between the initial resistivity and cement content as

$$\rho_o = V_2 + W_2 * [\text{cement}(\%)], \quad (4.7)$$

where V_2 and W_2 are model parameters. The proposed model showed a very good agreement with the experimental data ($R^2=0.99$). Table 4.16 summarized the model parameters for Eq. 4.7.

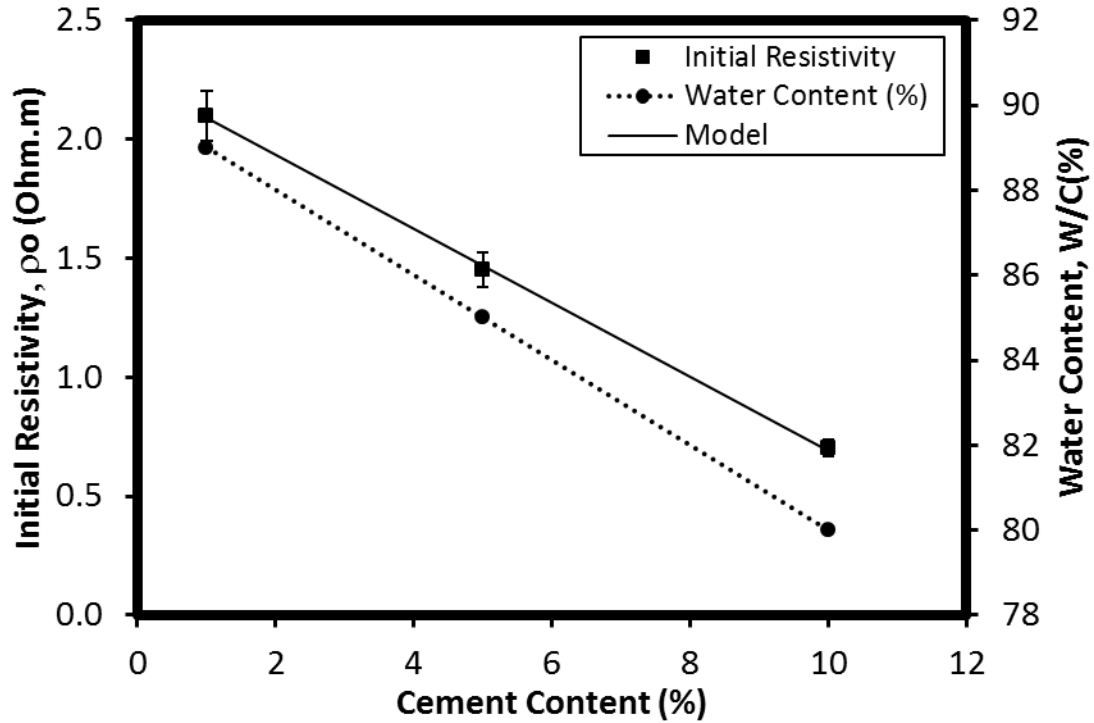


Figure 4.34 Variations of initial resistivity and water content of 10% bentonite treated soft soil with cement.

Table 4.16 Model parameters for Eq. 4.7

V_2 (Ohm.m)	W_2 (Ohm.m)	R^2
-0.16	2.25	0.99

The variations of shear strength of 10% bentonite treated soft soil with cement vs. initial resistivity can be shown in Figure 4.35. It has shown as the shear strength increased, the initial resistivity decreased. Shear strength vs. initial resistivity has shown nonlinear relationship and similar model as Eq. 4.2 can be used to predict such relationship. The proposed model predicted the experimental data with a very good

degree of agreement ($R^2=0.98$). Table 4.16 summarized the model parameters similar to Eq. 4.2.

The variation of the soft soil density and water content of 10% bentonite soft soil treated with cement versus cement content has drawn in Figure 4.36. As the cement content increased, the water content decreased due to the increasing of the amount of cement particles in the same occupied volume. The soft soil density increased from 1.066 gm/cm^3 to 1.135 gm/cm^3 while the water content decreased from 90% to 80% as the cement content increased from 0% to 10%.

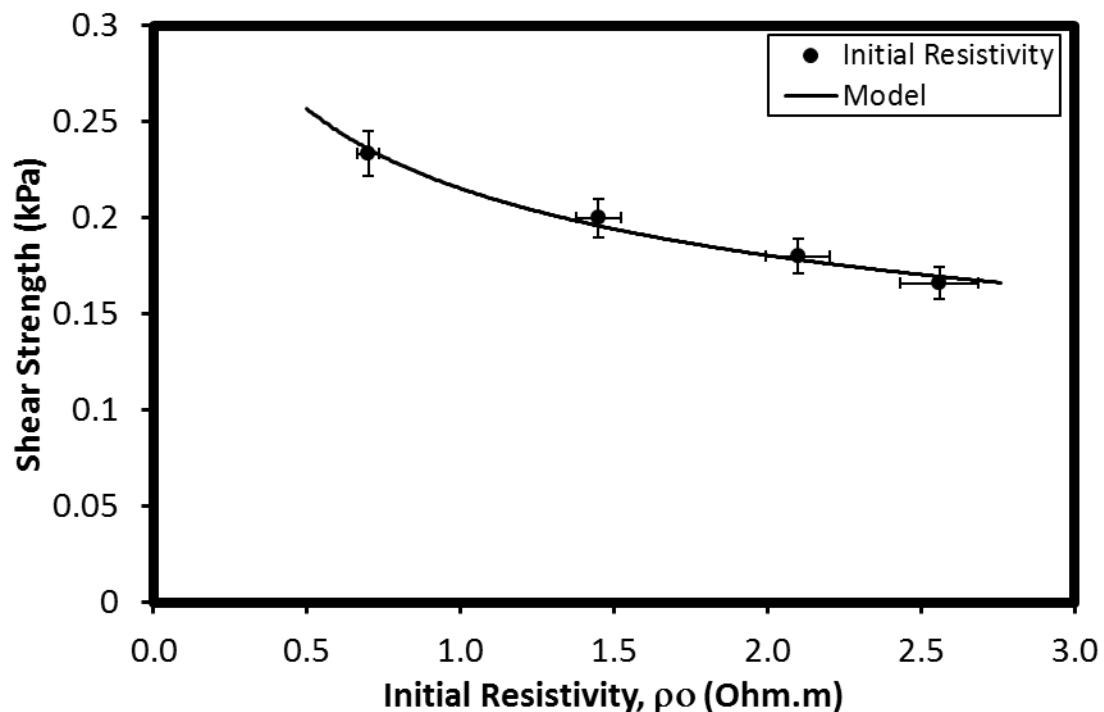


Figure 4.35 Variations of shear strength vs. resistivity of 10% bentonite treated soft soil with cement.

Table 4.17 Model parameters similar to Eq. 4.2

C (kPa/(Ohm.m))	D	R^2
0.22	-0.26	0.98

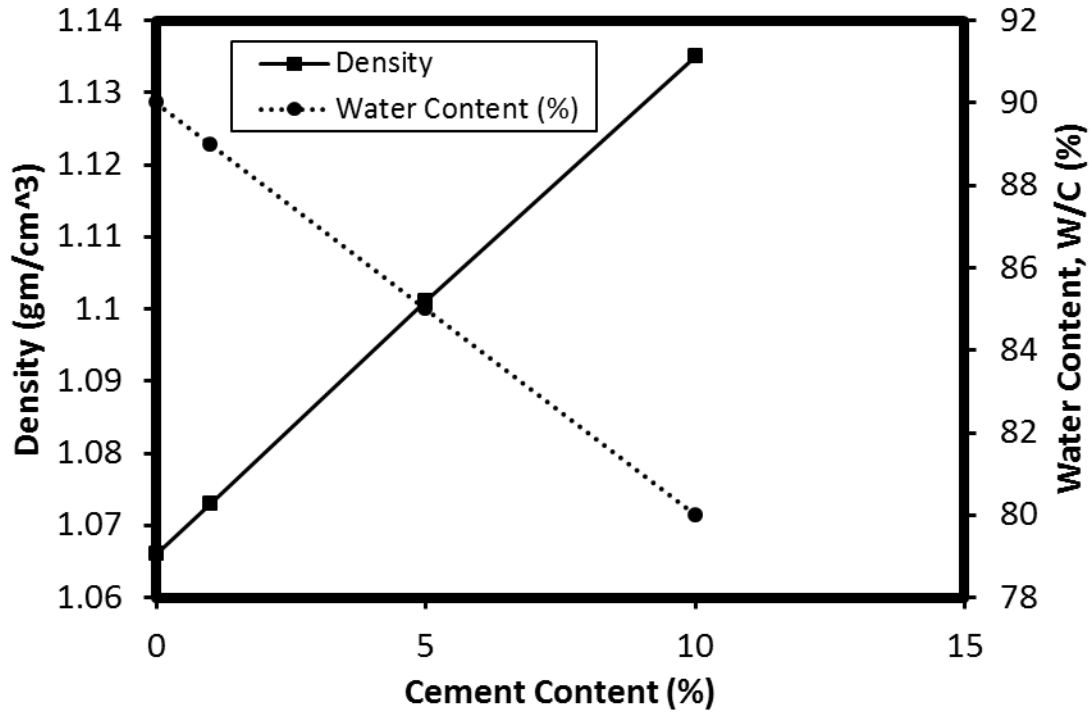


Figure 4.36 The variations of the soft soil density and water content of 10% bentonite treated soft soil with cement vs. cement content.

The variations of water content vs. initial resistivity of 10% bentonite treated soft soil with cement have been studied as shown in Figure 4.37. As the water content increased from 80% to 90%, the initial resistivity increased from 0.7 ± 0.001 Ohm.m to 2.56 ± 0.002 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.98$). Table 4.17 summarized the model parameters similar to Eq. 4.3.

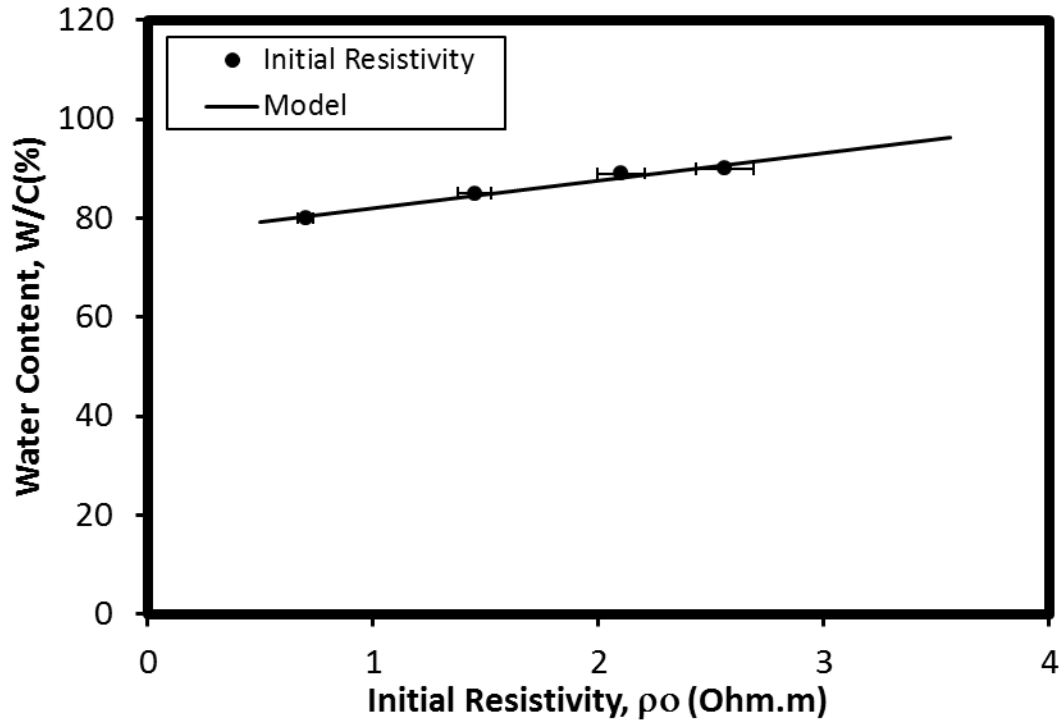


Figure 4.37 The variations of the soft water content of 10% bentonite treated soft soil with cement vs. initial resistivity.

Table 4.17 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R ²
5.6	76.6	0.98

The electrical impedance versus frequency of 10% bentonite soft soil treated with different cement contents has been measured using LCR device as shown in Figure 4.38. For all the different soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the cement content increased due to the increase in the conductivity of the medium with higher cement content. The drop in the electrical impedance has shown a significant value from 1.71 ± 0.0001 k. Ohm to 1.077 ± 0.0001 k. Ohm and 0.607 ± 0.0001 k. Ohm to 0.382 ± 0.00005 k. Ohm at 0.02 kHz and 300 kHz respectively.

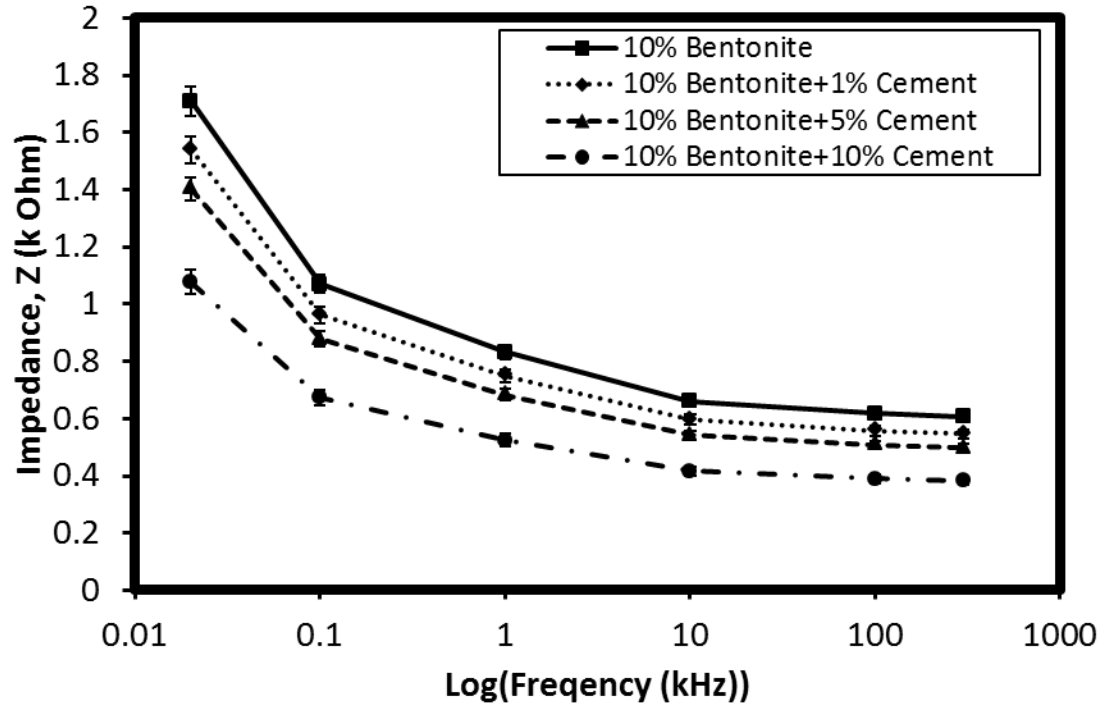


Figure 4.38 Electrical impedance vs. frequency change of 10% bentonite treated soft soil with cement for different cement contents.

4.3.5. Kaolinite

Soft soil improvement samples were prepared in the laboratory by adding different percentage of kaolinite from 2% to 10% with the 10% bentonite soil at room temperature and mixed for about ten minutes. The soft soil slurry mixture was placed in 50 mm (dia.) x 100 mm (height) cylindrical plastic mold and tested through modified vane shear device to measure the mud shear strength. The modified vane shear device was used to measure the extreme low strength of the prepared mud soft soil. In Figure 4.39, both shear strength and water content of 10% bentonite treated soft soil with kaolinite vs. kaolinite content is shown. It is clearly shown that the shear strength increased from 0.15 ± 0.0001 kPa to 0.26 ± 0.0002 kPa and water content decreased from 90% to 80% as the kaolinite content increased from 0% to 10% respectively.

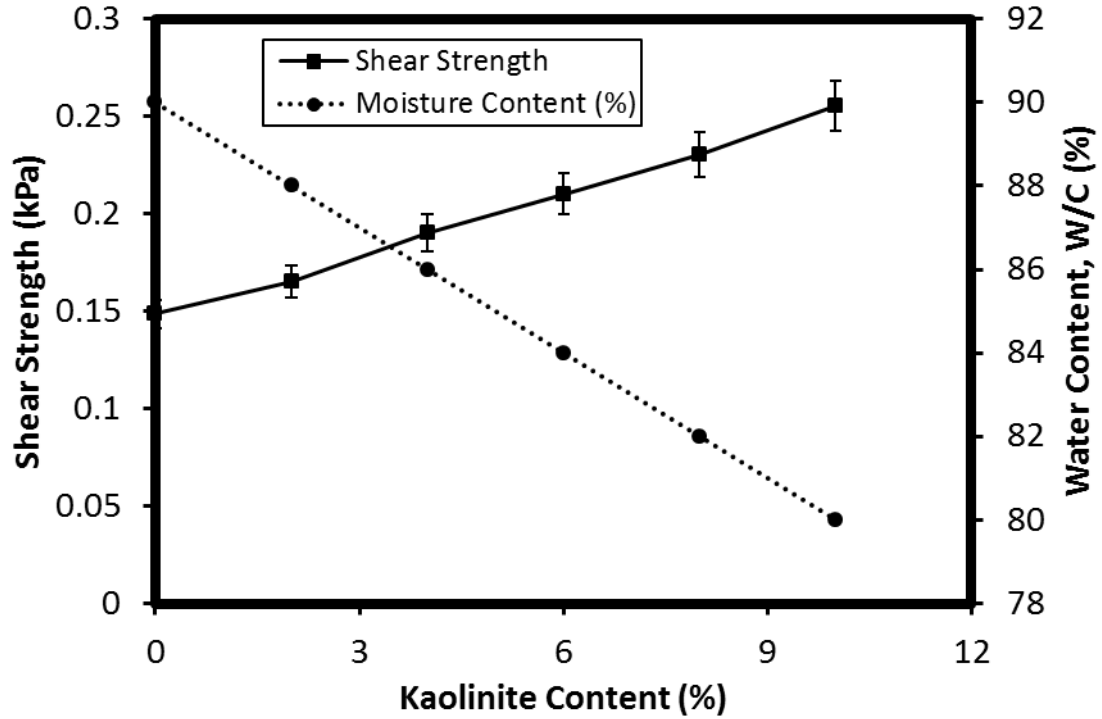


Figure 4.39 Variation of shear strength and water content of 10% bentonite treated soft soil with kaolinite.

The variations of both initial resistivity and water content of 10% bentonite treated soft soil with kaolinite vs. kaolinite content has been studied as shown in Figure 4.40. As water content has decreased from 88% to 80 %, the initial resistivity has decreased from 2.15 ± 0.002 Ohm.m to 1.05 ± 0.001 Ohm.m. Due to the increase in the soft soil shear strength with the increase of kaolinite content, the initial resistivity has decreased because the medium became more conductive. A nonlinear model is used to predict the relationship between the initial resistivity and kaolinite content and similar model as Eq. 4.1 can be used to predict such a relationship. The proposed model showed a very good agreement with the experimental data ($R^2=0.97$). Table 4.16 summarized the model parameters similar to Eq. 4.1.

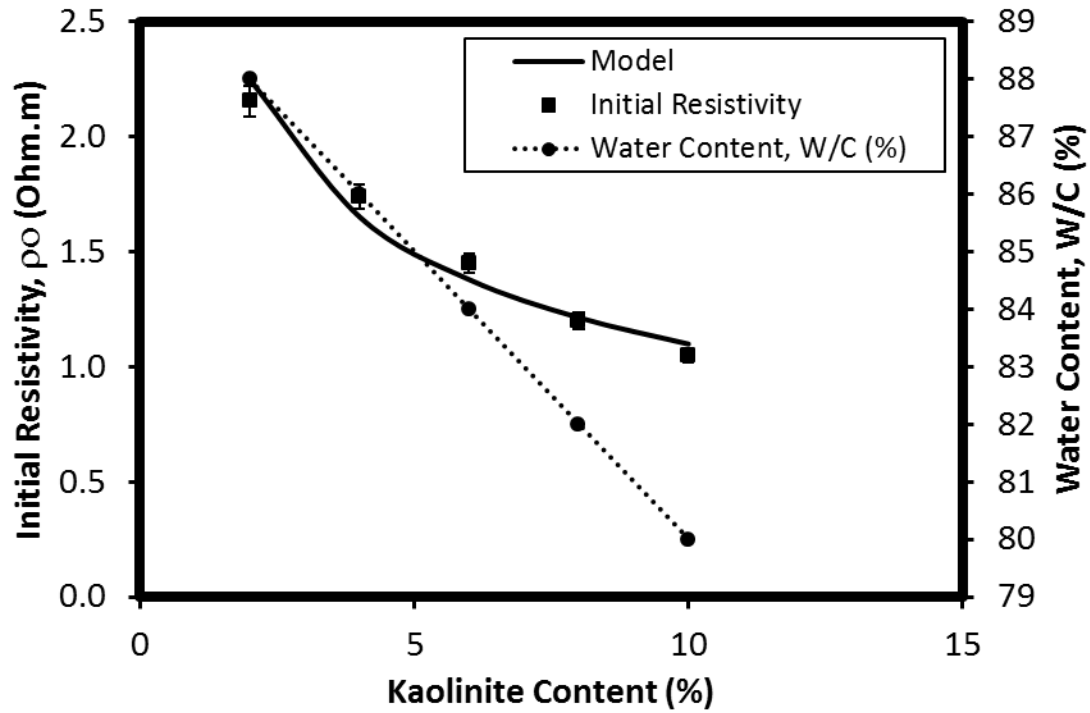


Figure 4.40 Variations of initial resistivity and water content of 10% bentonite treated soft soil with kaolinite.

Table 4.18 Model parameters similar to Eq. 4.1

A (Ohm.m)	B	R ²
3.06	-0.45	0.97

The variations of shear strength of 10% bentonite treated soft soil with kaolinite vs. initial resistivity can be shown in Figure 4.41. It has shown as the shear strength increased, the initial resistivity decreased. Shear strength vs. initial resistivity has shown nonlinear relationship and similar model as Eq. 4.2 can be used to predict such relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2=0.99$). Table 4.19 summarized the model parameters similar to Eq. 4.2.

The variation of the soft soil density and water content of 10% bentonite soft soil treated with kaolinite versus kaolinite content has drawn in Figure 4.42. As the kaolinite

content increased, the water content decreased due to the increasing of the amount of kaolinite particles in the same occupied volume. The soft soil density increased from 1.078 gm/cm^3 to 1.125 gm/cm^3 while the water content decreased from 88% to 80% as the kaolinite content increased from 2% to 10%.

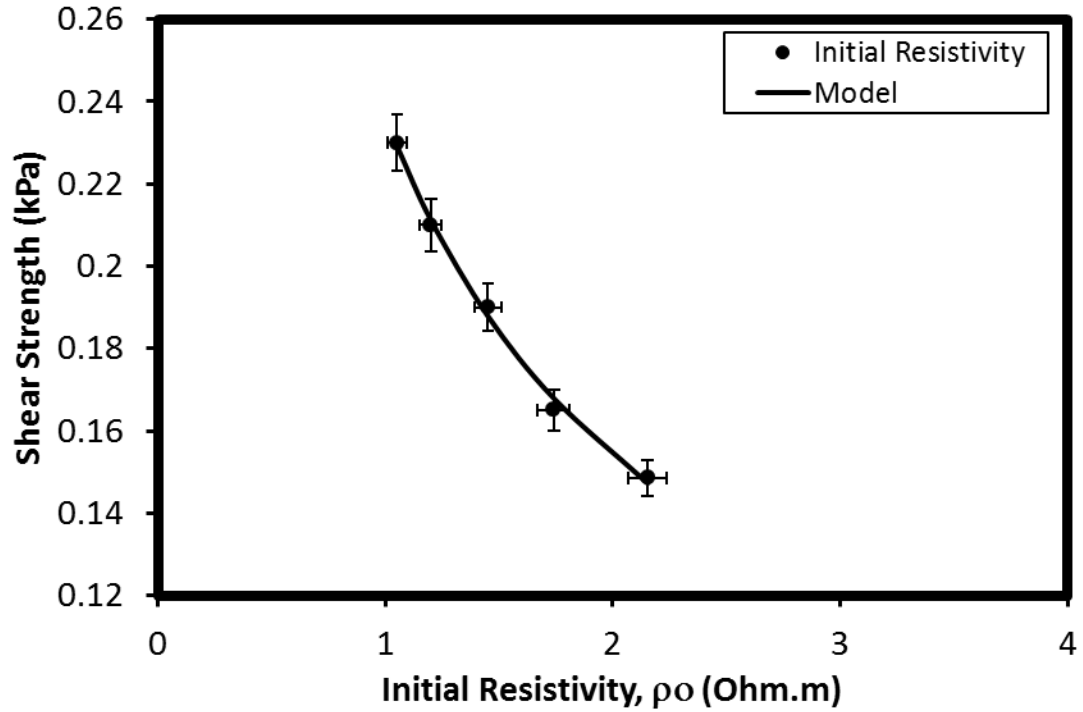


Figure 4.41 Variations of shear strength vs. resistivity of 10% bentonite treated soft soil with kaolinite.

Table 4.19 Model parameters similar to Eq. 4.2

C (kPa/(Ohm.m))	D	R ²
0.24	-0.62	0.99

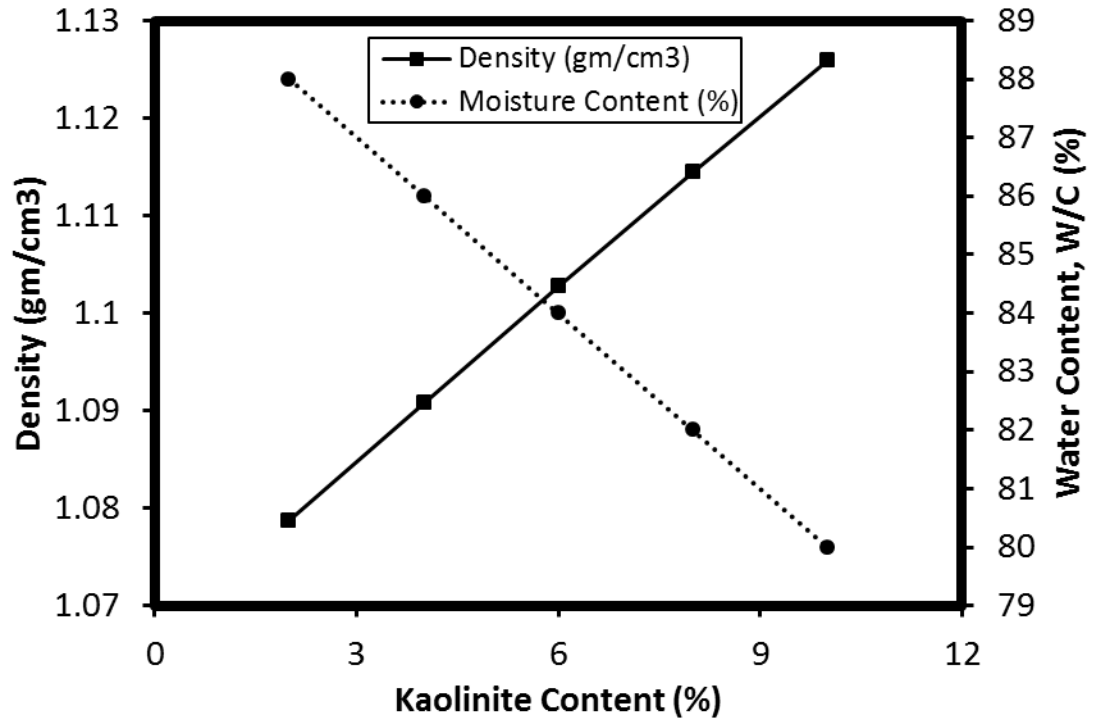


Figure 4.42 The variations of the soft soil density and water content of 10% bentonite treated soft soil with kaolinite vs. kaolinite content.

The variations of water content vs. initial resistivity of 10% bentonite treated soft soil with kaolinite have been studied as shown in Figure 4.43. As the water content increased from 80% to 90%, the initial resistivity increased from 1.05 ± 0.001 Ohm.m to 2.15 ± 0.002 Ohm.m. Water content vs. resistivity has shown a linear relationship and similar model as Eq. 4.3 can be used to predict such a relationship. The proposed model predicted the experimental data with a very good degree of agreement ($R^2 = 0.97$). Table 4.20 summarized the model parameters similar to Eq. 4.3.

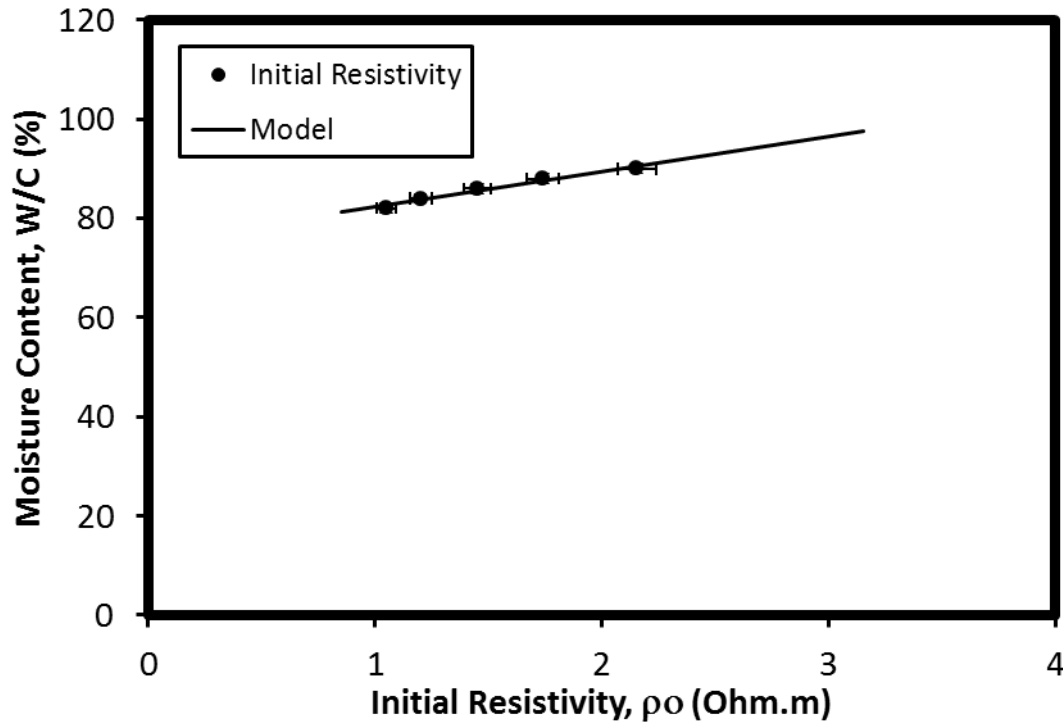


Figure 4.43 The variations of the soft water content of 10% bentonite treated soft soil with kaolinite vs. initial resistivity.

Table 4.20 Model parameters similar to Eq. 4.3

E (1/(Ohm.m))	F	R ²
7.06	75.3	0.97

The electrical impedance versus frequency of 10% bentonite soft soil treated with different kaolinite contents has been measured using LCR device as shown in Figure 4.44. For all the different soft soil compositions, as the frequency increased the electrical impedance decreased due to the minimizing of the effect of the contact resistance. The electrical impedance decreased as the kaolinite content increased due to the increase in the conductivity of the medium with higher kaolinite content. The drop in the electrical impedance has shown a significant value from 1.05 ± 0.001 k.Ohm to 0.8 ± 0.001 k.Ohm and 0.64 ± 0.0005 k.Ohm to 0.534 ± 0.0005 k.Ohm at 0.02 kHz and 300 kHz respectively.

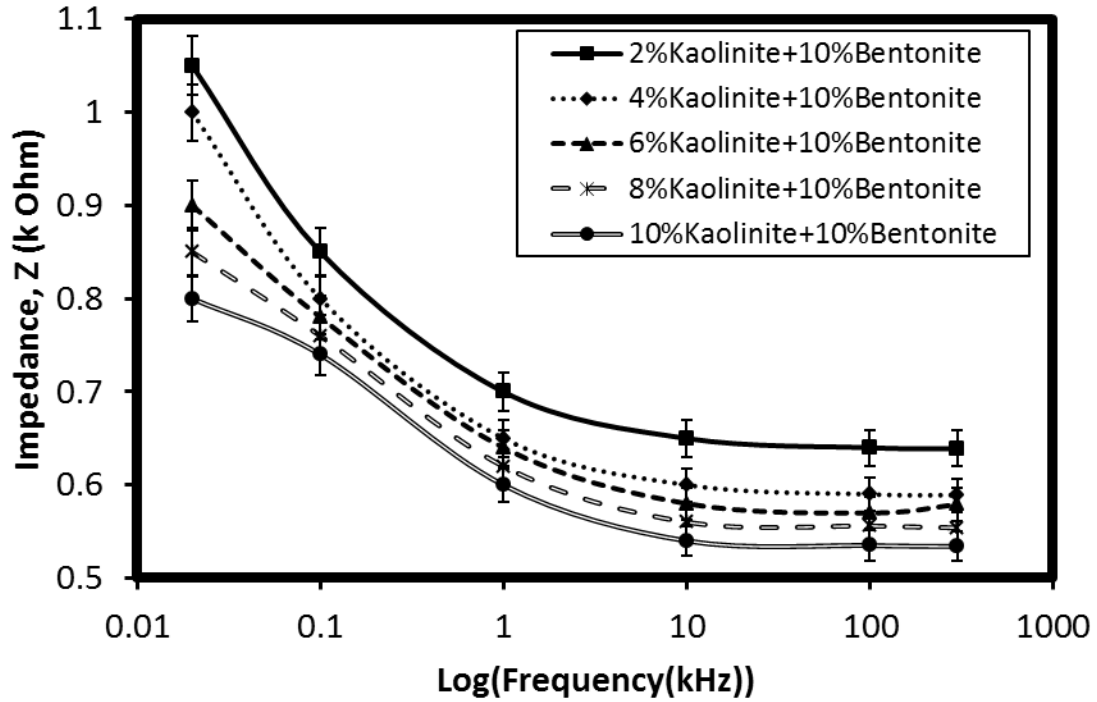


Figure 4.44 Electrical impedance vs. frequency change of 10% bentonite treated soft soil with kaolinite for different kaolinite contents.

4.4. Interface tests

Several small scale-tests have been performed to quantify the interface shear strength of soft soil using small concrete solid cylinders. A thin steel rod (2 mm (dia.)) was embedded in the small concrete cylinder with dimensions of 50 mm (dia.) x 100 mm (height) then attached to the modified vane shear device replacing the position of the vane blades. A container with dimensions of 250 mm (dia.) x 500 mm (height) was filled with the soft soil, then the concrete cylinder was lowered to the container until it was fully penetrated the soft soil followed by starting the test at a very small displacement rate (0.1 mm / sec). The schematic diagram for the interface testing details can be idealized in Figure 4.45.

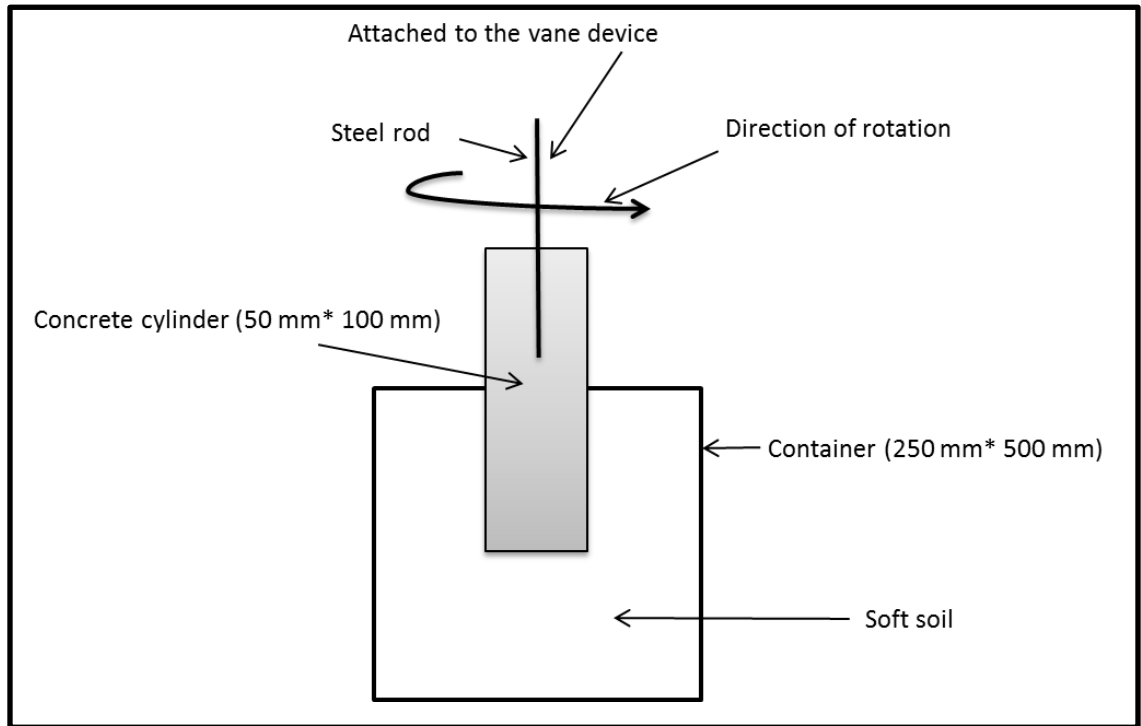


Figure 4.45 The schematic diagram for the small-scale interface testing details.

Several soft soil compositions, including 10% bentonite soft soil, 10% bentonite soft soil treated with 5% to 20% lime, and 10% bentonite soft soil treated with 1% to 10% polymer were tested to quantify the maximum induced interface shear stress (shear strength). The variations of the shear strength and interface shear for the soft soil vs. lime content is identified in Figure 4.46. The interface shear is almost 50% of the soft soil shear strength and similar trends can be exhibited for both shear strength and interface shear.

The variation of the interface shear vs. water content for the soft soil can be shown in Figure 4.47. As the lime content increased from 0% to 20%, the interface shear decreased from 0.085 ± 0.001 kPa to 0.074 ± 0.001 kPa with optimum interface shear of 0.094 ± 0.001 kPa at 5% lime content while the water content decreased from 90% to 70%.

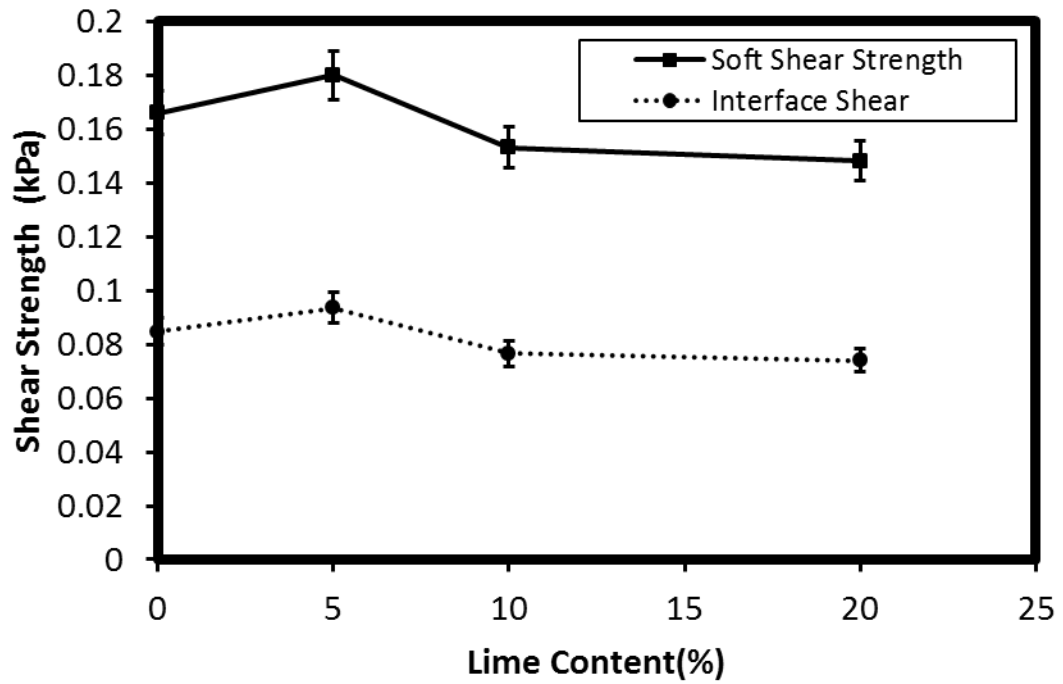


Figure 4.46 Comparison of soft soil shear strength and interface shear with different lime content.

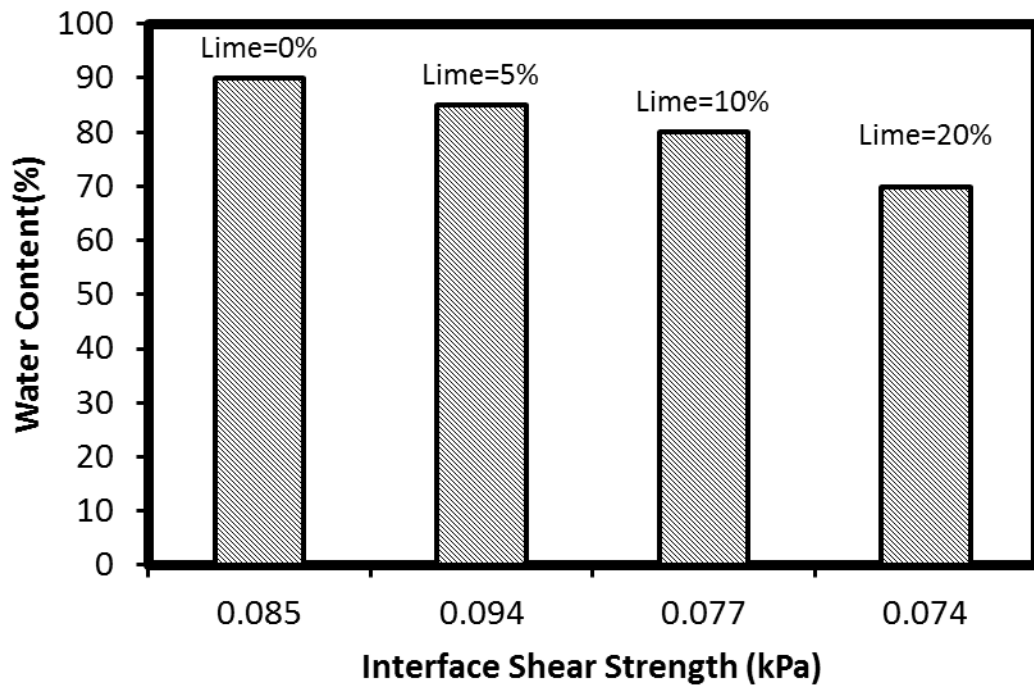


Figure 4.47 Variation of soft soil interface shear vs. water and lime contents.

Similarly, the variations of the shear strength and interface shear for the soft soil vs. polymer content is identified in Figure 4.48. Similar trends can be exhibited for both shear strength and interface shear. The interface shear is almost 53% of the soft soil shear strength. As the polymer content increased from 0% to 10%, the shear strength and interface shear increased from 0.17 ± 0.001 kPa to 6.7 ± 0.01 kPa and 0.1 ± 0.001 kPa to 3.6 ± 0.0007 kPa respectively.

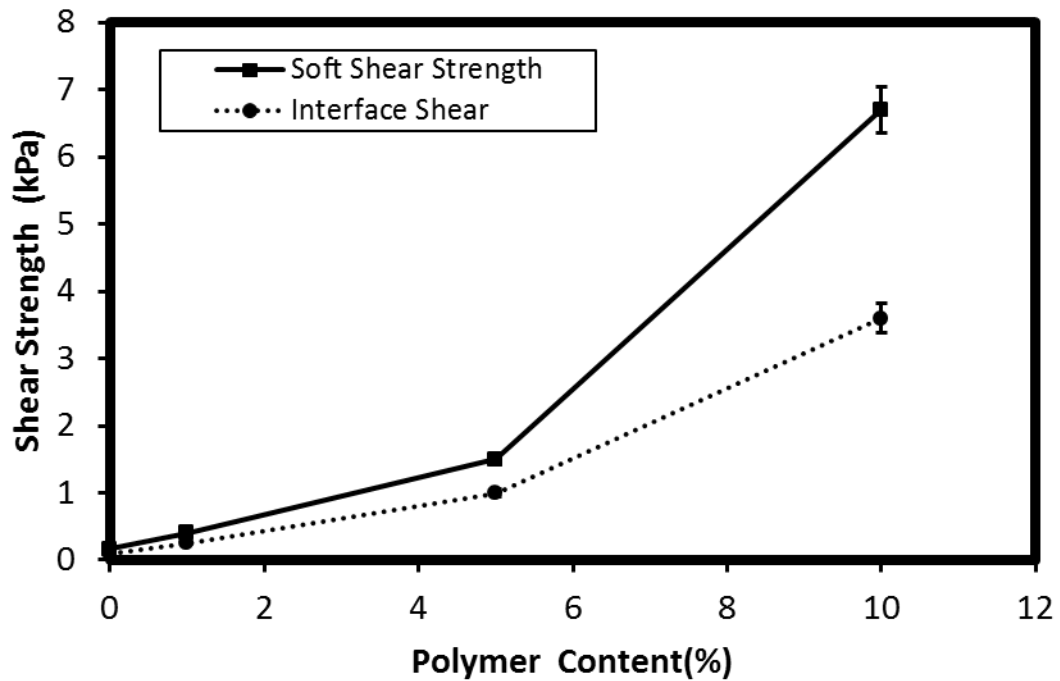


Figure 4.48 Comparison of soft soil shear strength and interface shear with different polymer content.

The variation of the interface shear vs. water content for the soft soil can be shown in Figure 4.49. As the polymer content increased from 0% to 10%, the interface shear increased from 0.1 kPa to 3.6 kPa and the water content decreased from 90% to 80 %.

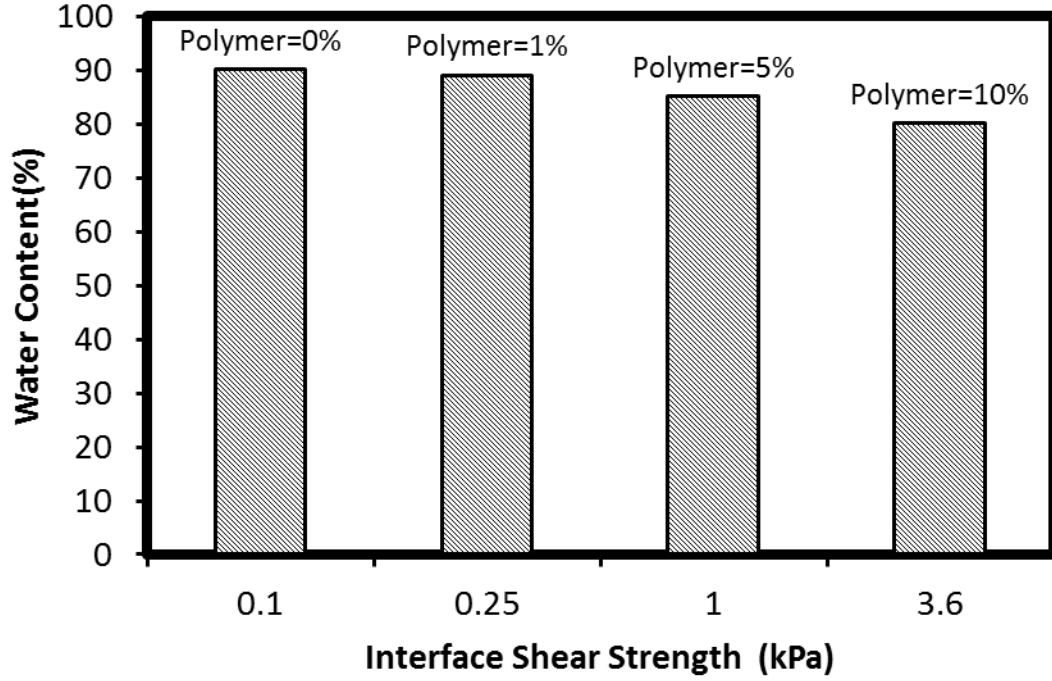


Figure 4.49 Variation of soft soil interface shear vs. water and polymer contents.

The relationship between the interface shear and soft soil shear strength of soft soil treated with lime is shown in Figure 4.50. There is a linear relationship between the interface shear and soft soil strength and it can be modeled as

$$\tau_{\text{int}} = A_1 + B_1 \tau_{\text{soil}}, \quad (4.8)$$

where τ_{int} and τ_{soil} are soft soil interface shear and shear strength. A_1 and B_1 are model parameters. The proposed model predicted the experimental data with very good agreement ($R^2=0.99$). Table 4.21 summarized the model parameters for Eq. 4.8.

Likewise, the relationship between the interface shear and soft soil shear strength of soft soil treated with polymer is shown in Figure 4.51. Similar model as Eq. 4.8 is used to predict such a relationship with a very good degree of agreement with the experimental data ($R^2=0.99$). Table 4.21 summarized the model parameters similar to Eq. 4.8.

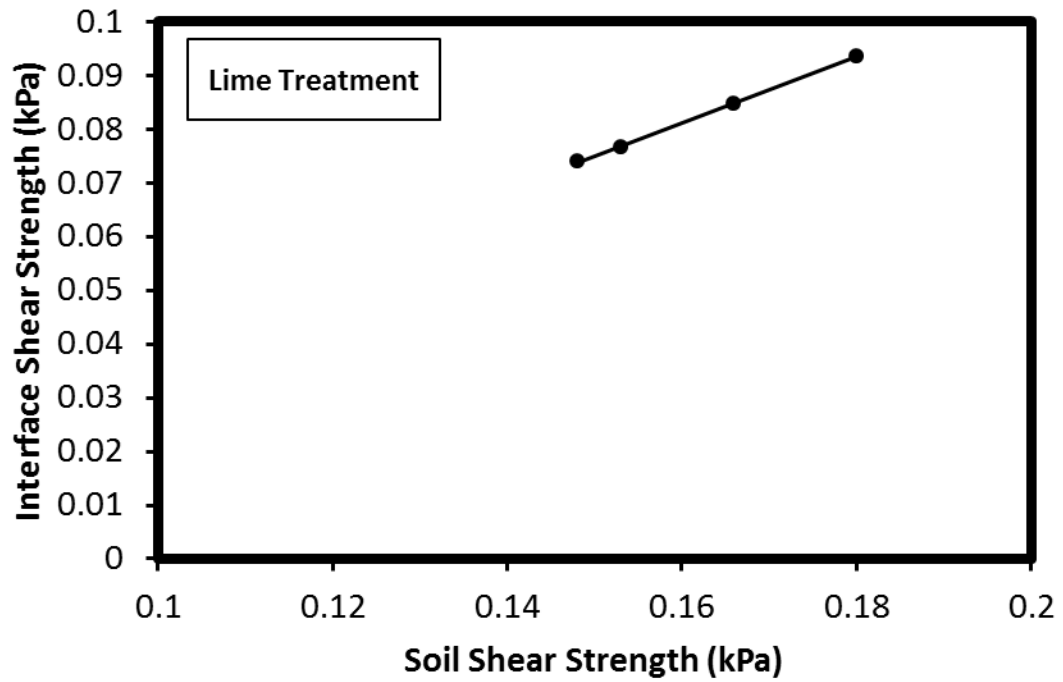


Figure 4.50 The relationship of interface shear and shear strength of lime treated soft soil.

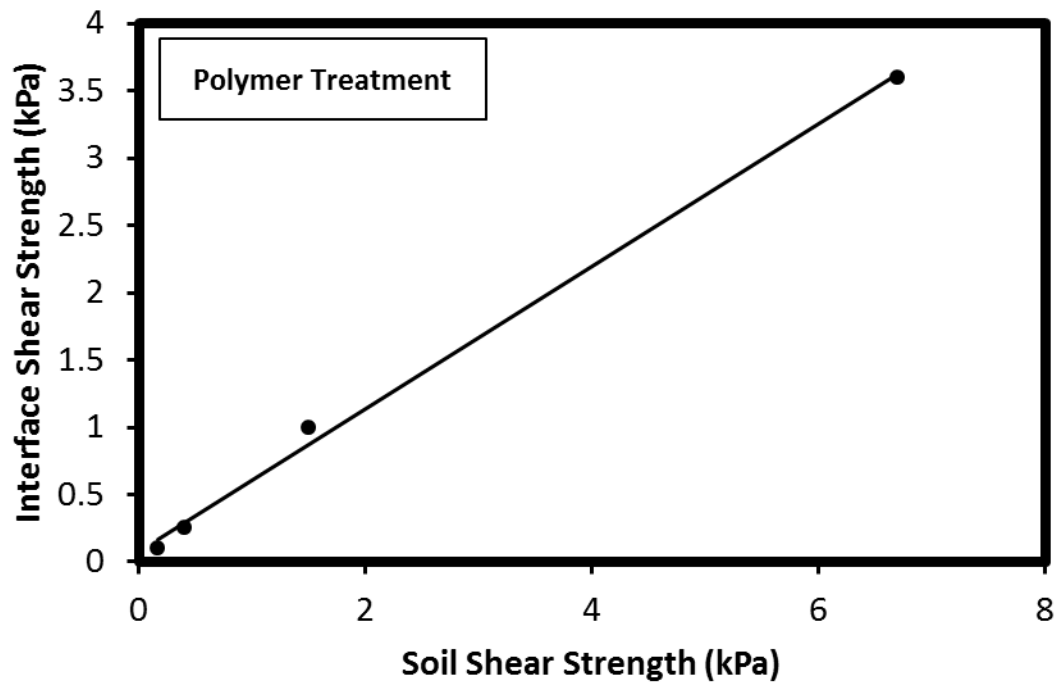


Figure 4.51 The relationship of interface shear and shear strength of polymer treated soft soil.

Table 4.21 Model parameters for Eq. 4.8

Treatment Type	A ₁ (kPa)	B ₁	R ²
Lime	0.62	-0.018	0.99
Polymer	0.53	0.076	0.99

A comparison of soft soil shear strength vs. interface shear for soft soil treated with lime and polymer is shown in Figure 4.52. The effectiveness of the polymer over the lime in terms of induced shear strength is noticeably presented. The interface shear strength of treated soft soil with 10% polymer was 49 times greater than the interface shear strength of treated soft soil with 20% lime.

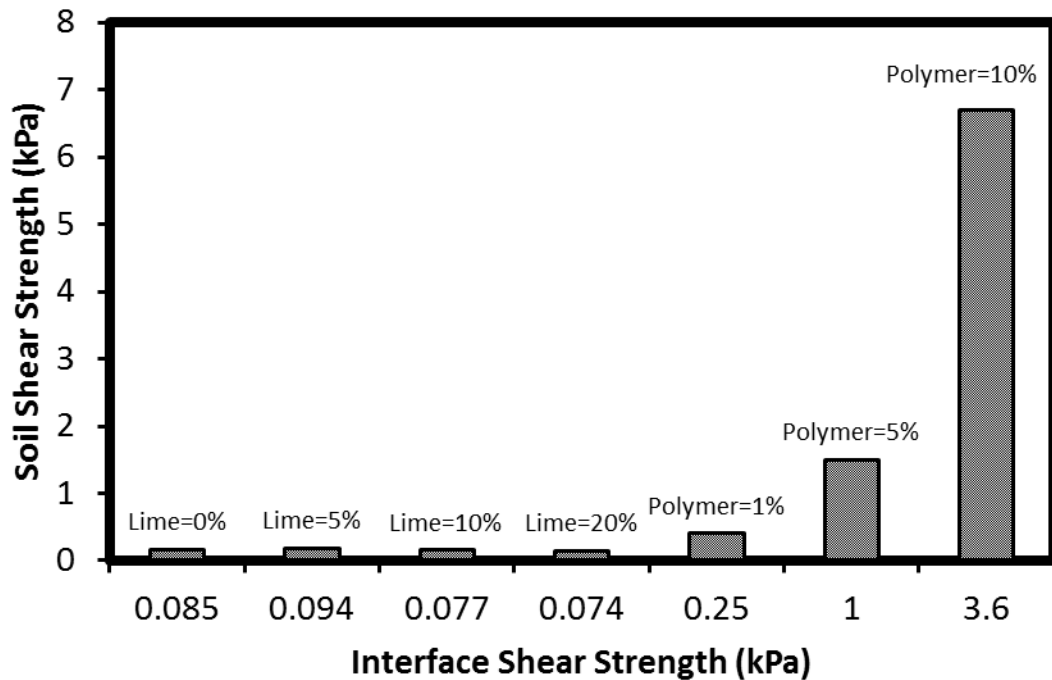


Figure 4.52 Comparison of shear strength and interface shear for soft soil treated with lime and polymer.

A comparison of soft soil water content vs. interface shear for soft soil treated with lime and polymer is shown in Figure 4.53. The reduction in the water content in the

treated soft soil with more polymer content increased the interface shear strength while more lime contents decreased both water content and interface shear strength. As the polymer content increased from 1% to 10%, the interface shear strength increased by 14 times and the water content decreased by 10%. As the lime content increased from 5% to 20%, the interface shear strength decreased by 1.3 times and the water content decreased by 18%.

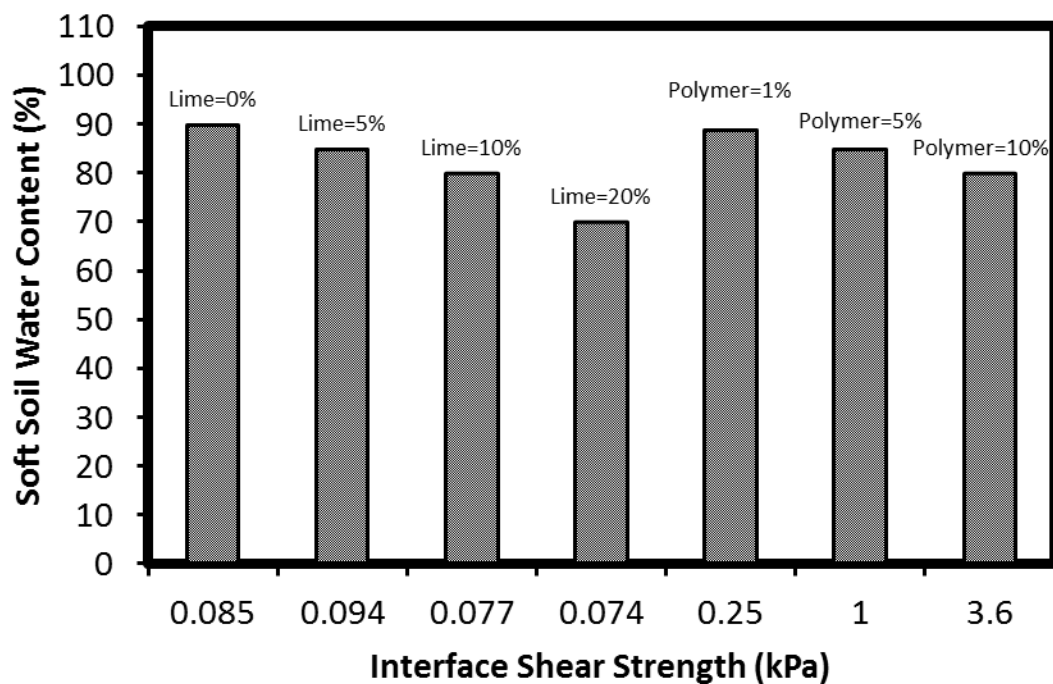


Figure 4.53 Comparison of water content and interface shear for soft soil treated with lime and polymer.

4.5.Soft soil profile characterization

One of the most critical challenges in the field is to characterize the soft soil profile with depth in terms of shear strength. Electrical resistivity can be used as one of the methods to provide such kind of profile due to its strong correlation with the shear

strength. In addition, electrical resistivity is used to monitor the particle settling over the time and it can predict the solid content in any level at any time.

To further investigate the particle settling, a container with dimensions of 20 cm (dia.) and 50 cm (height) is prepared with embedded wires in different levels to monitor the changes in electrical resistivity with the hydrometer test in parallel. The container is filled with 2% bentonite soft soil and the hydrometer and electrical resistivity measurement were taken for up to 5800 min. In Figure 4.54, the hydrometer results for 2% bentonite is shown where the percent of finer vs. the particle diameter is identified. It is obviously shown that the hydrometer is able to quantify the particle settlement even for long-term measurement where 0.00072 mm particle diameter is settled after 5800 minutes. However, hydrometer test is not able to quantify the long-term percent of finer where the lowest recorded percent of finer was 6.02% after 376 minutes.

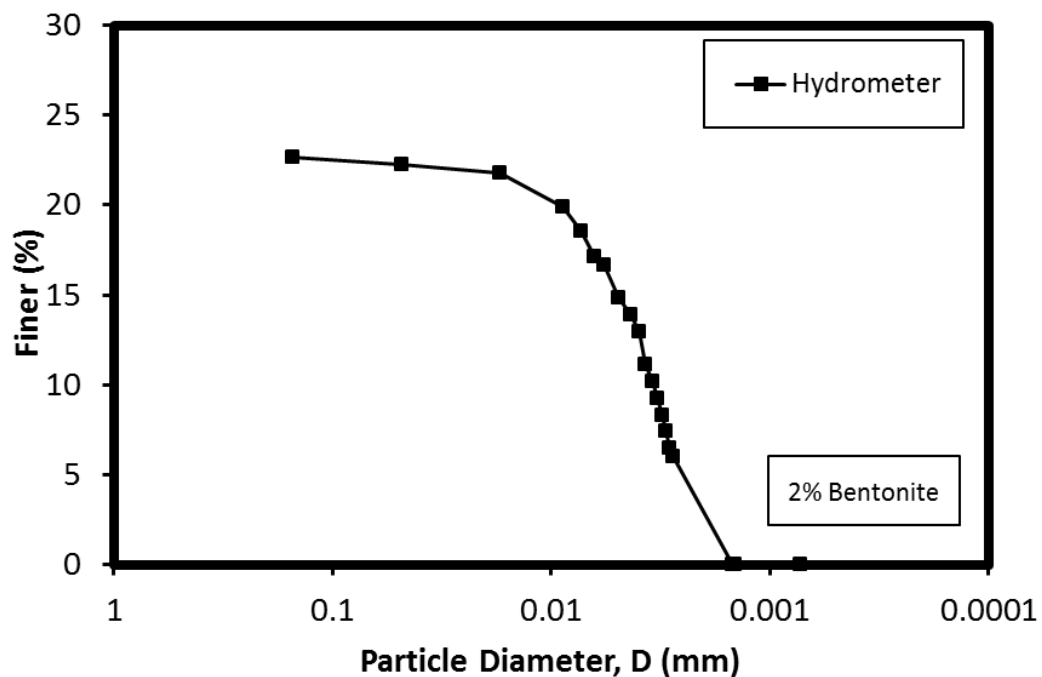


Figure 4.54 The percent finer vs. particle diameter for 2% bentonite soft soil.

The variation of both electrical resistance and particle diameter for 2% bentonite over the time through the hydrometer test is shown in Figure 4.55. The electrical resistance is measured at different levels and the changes in the resistance is clearly indicating the particle settling where hydrometer reading is incorporated as well. Similarly, the variation of both electrical resistance and percent finer for 2% bentonite over the time through hydrometer is drawn in Figure 4.56. The percent finer showed zero value after 376 minutes only while the process of particle settling continued for up to 5800 minutes.

The particle diameters and percent finer decreased from 0.15 mm to 0.00072 mm and 22.69% to 0% as the time increased from 0 minutes to 5800 minutes respectively.

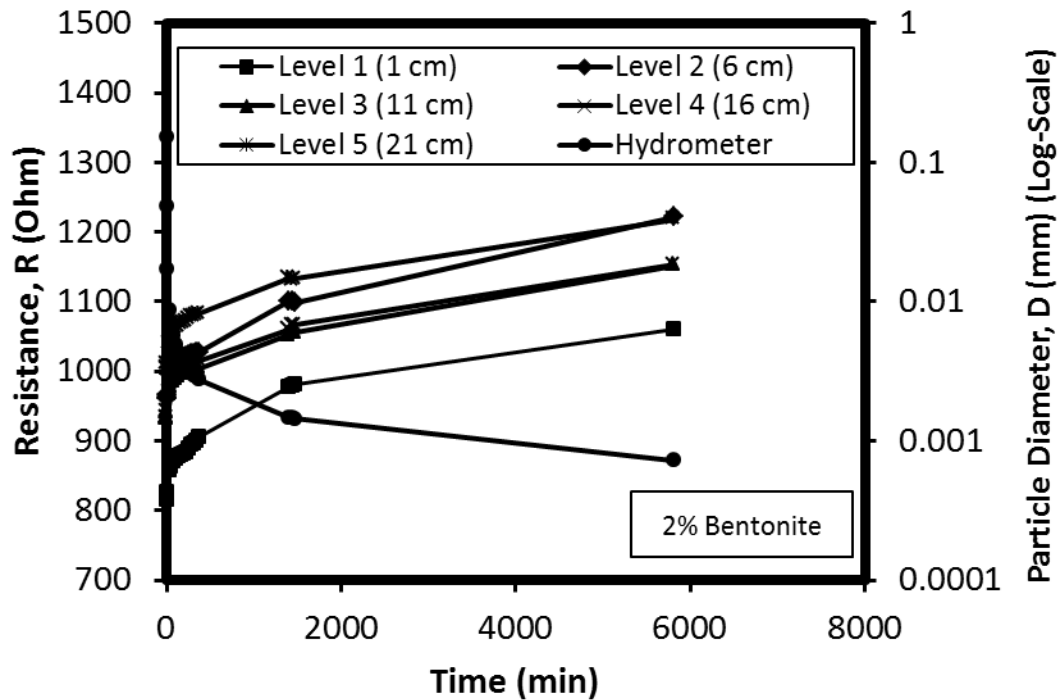


Figure 4.55 Comparison of electrical resistance and particle diameter variation over the time during hydrometer test and up to 5800 min for 2% bentonite soft soil.

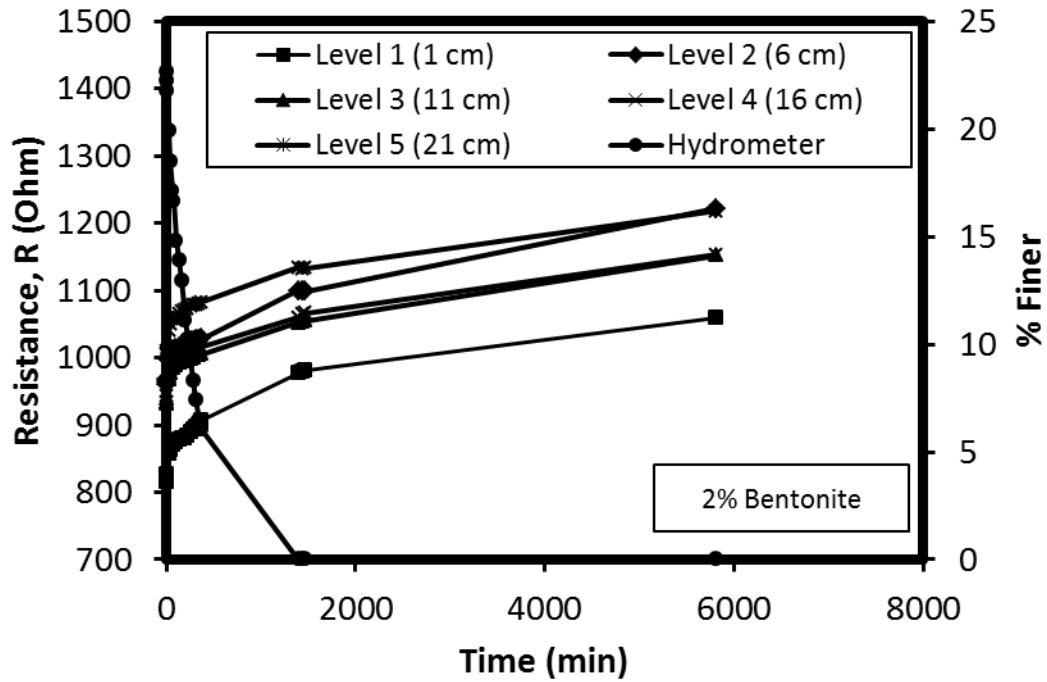


Figure 4.56 Comparison of electrical resistance and % finer variation over the time during hydrometer test and up to 5800 min for 2% bentonite soft soil.

The variation of the electrical resistance with the depth over the time for 2% bentonite is shown in Figure 4.57. The electrical resistance increased with the time due to the particle settling that made the medium less conductive. The overall variation of the recorded electrical resistance is ranged from 816 ± 1 Ohm to 1218 ± 2 Ohm at 1 cm and 21 cm depth respectively. Similarly, the variation of the electrical resistivity with the depth over the time 2% bentonite is identified in Figure 4.58. The resistivity increased over the time for all the tested depths due to the decrease in the conductivity because of bentonite particle settling. The overall variation of the resistivity is ranged from 6.29 ± 0.003 Ohm.m at time zero and depth 1 cm to 7.62 ± 0.001 Ohm.m at time 5800 min and depth 21 cm.

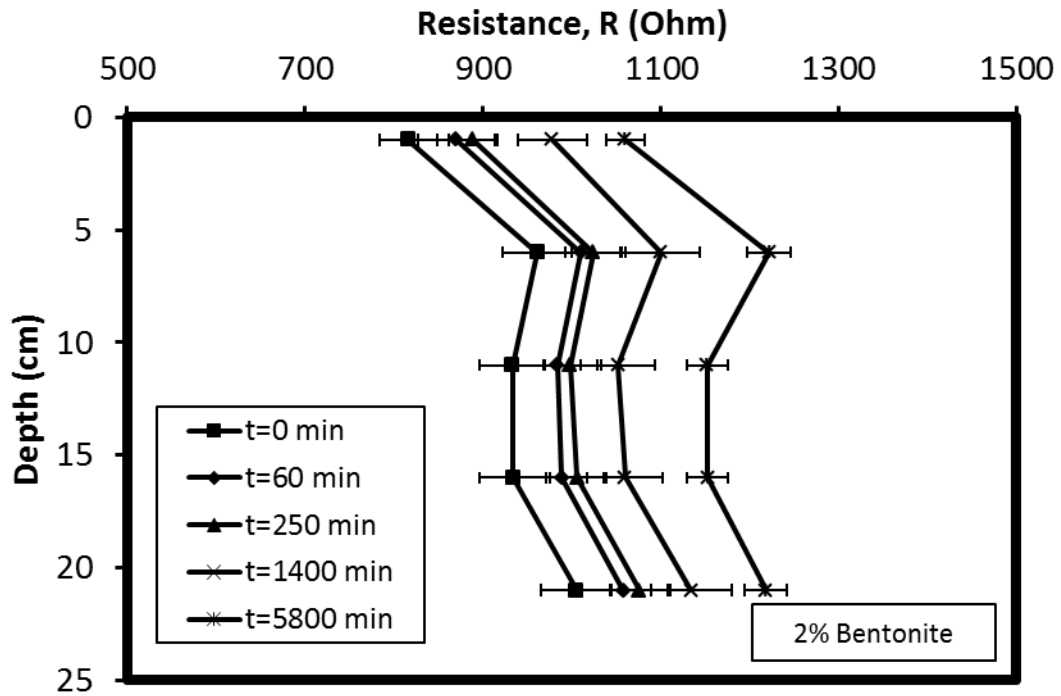


Figure 4.57 The variation of the electrical resistance with the depth over the time for 2% bentonite soft soil.

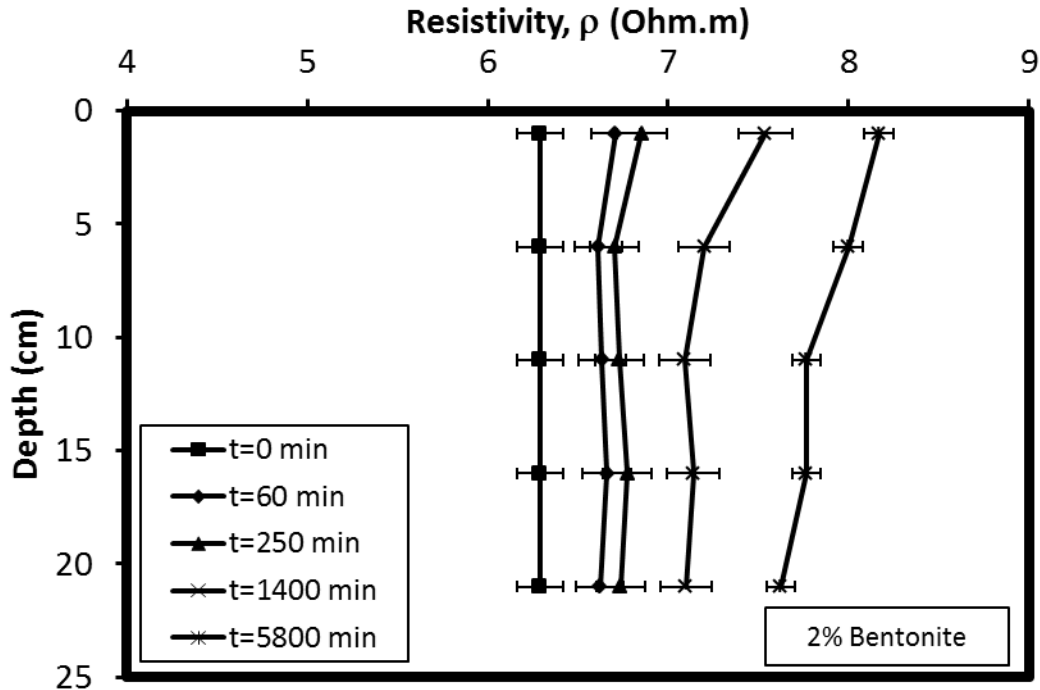


Figure 4.58 The variation of the electrical resistivity with the depth over the time for 2% bentonite soft soil.

The variation of bentonite concentration with the depth over the time is shown in Figure 4.59. As the time increased from 0 minutes to 5800 minutes, the bentonite percent decreased from 2% to 1.27% and 2% to 1.43% at 1 cm and 21 cm depth respectively. The decrease in concentration is due to particle settling over the time.

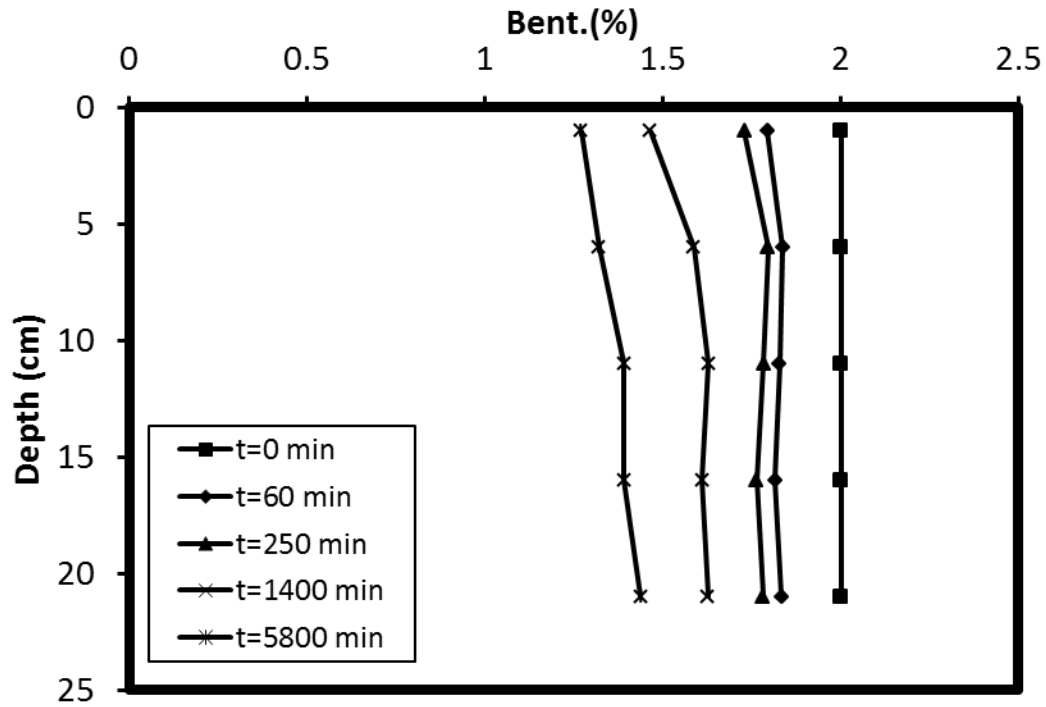


Figure 4.59 The variation of the bentonite content with the depth over the time for 2% bentonite soft soil.

The variation of the shear strength profile for 2% bentonite soft soil with the depth over the time is drawn in Figure 4.60. As the time increased from 0 minutes to 5800 minutes, the shear strength decreased from 0.011 ± 0.000001 kPa to 0.0039 ± 0.000001 kPa and 0.0048 ± 0.000001 kPa at 1 cm and 21 cm depth respectively. The decrease in the shear strength is due to particle settling over the time.

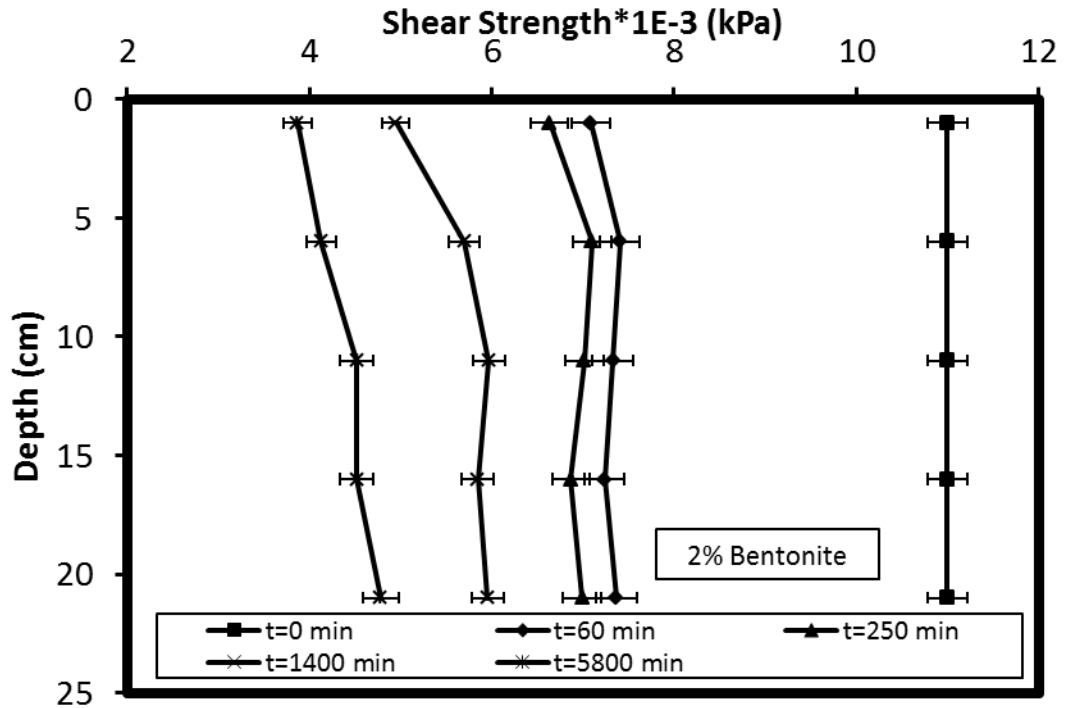


Figure 4.60 The variation of the shear strength with the depth over the time for 2% bentonite soft soil.

In Figure 4.61, the hydrometer results for 2% kaolinite is shown where the percent of finer vs. the particle diameter is identified. It is obviously shown that the hydrometer is able to quantify the particle settlement even for long-term measurement where 0.0014 mm particle diameter is settled after 1410 minutes. However, hydrometer test is not able to quantify the long-term percent of finer where the lowest recorded percent of finer was 0.9 % after 24 minutes.

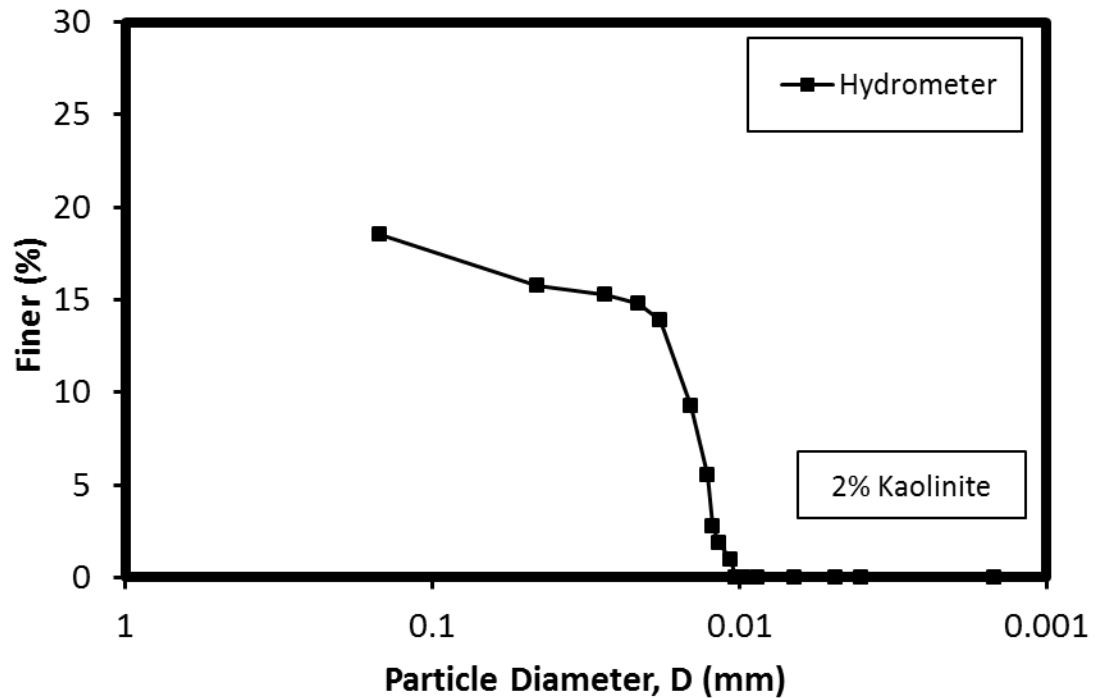


Figure 4.61 The percent finer vs. particle diameter for 2% bentonite soft soil.

The variation of both electrical resistance and particle diameter for 2% kaolinite over the time through the hydrometer test is shown in Figure 4.62. The electrical resistance is measured at different levels and the changes in the resistance is clearly indicating the particle settling where hydrometer reading is incorporated as well. Similarly, the variation of both electrical resistance and percent finer for 2% kaolinite over the time through hydrometer is drawn in Figure 4.63. The percent finer showed zero value after 24 minutes only while the process of particle settling continued for up to 1410 minutes. The particle diameters and percent finer decreased from 0.15 mm to 0.0014 mm and 18.51% to 0% as the time increased from 0 minutes to 1410 minutes respectively.

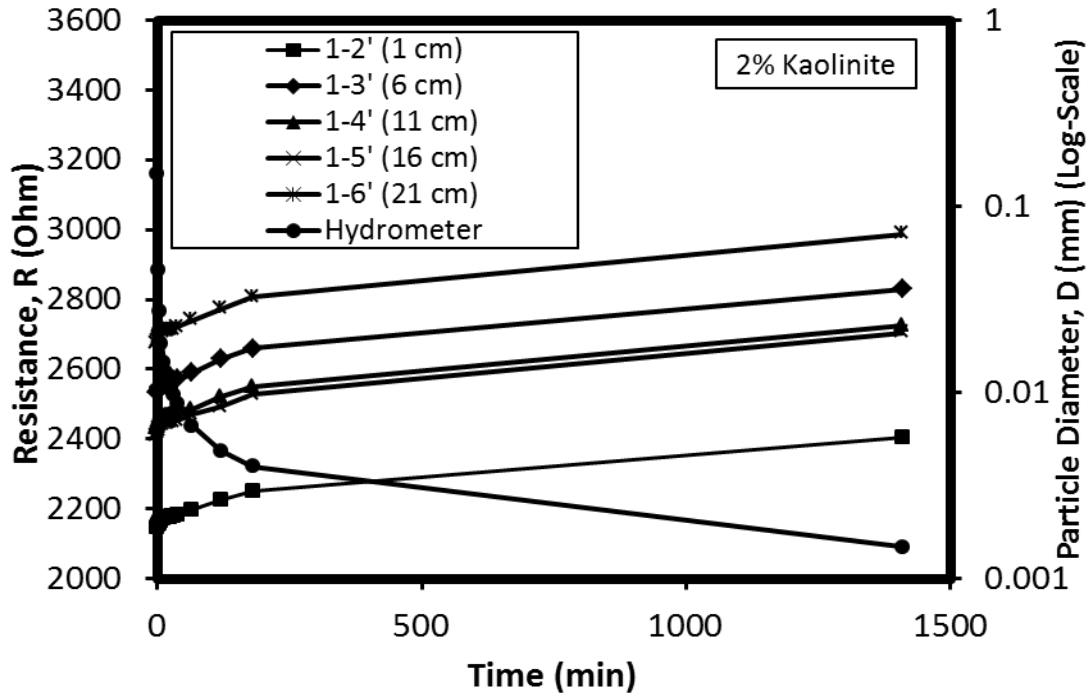


Figure 4.62 Comparison of electrical resistance and particle diameter variation over the time during hydrometer test and up to 1410 min for 2% kaolinite soft soil.

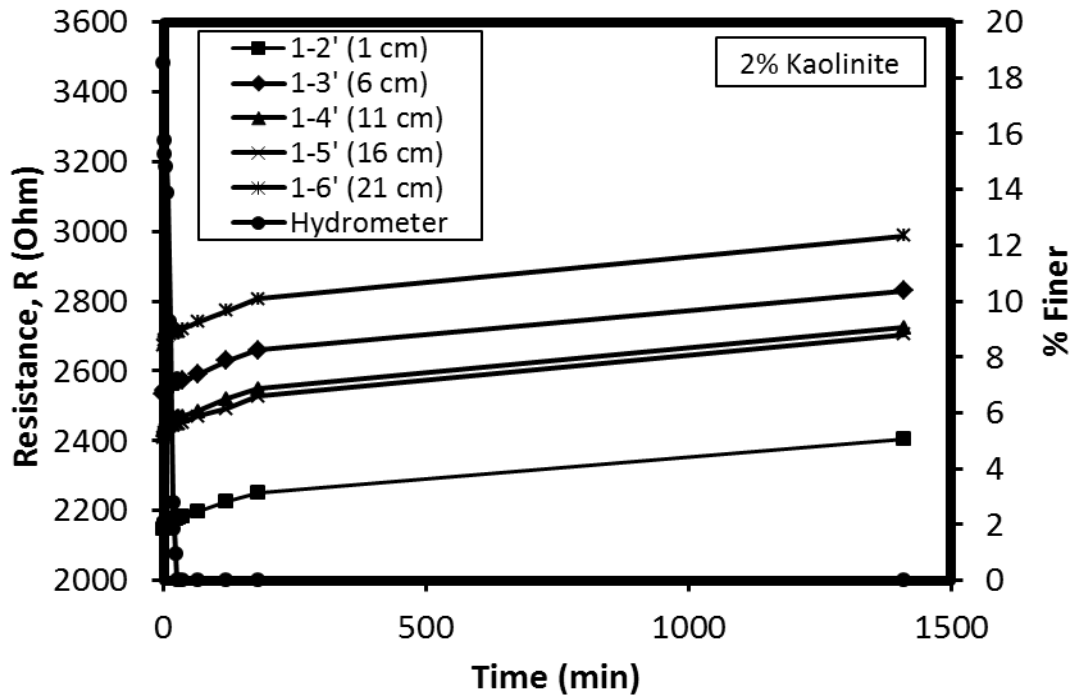


Figure 4.63 Comparison of electrical resistance and % finer variation over the time during hydrometer test and up to 1410 min for 2% kaolinite soft soil.

The variation of the electrical resistance with the depth over the time for 2% kaolinite is shown in Figure 4.64. The electrical resistance increased with the time due to the particle settling that made the medium less conductive. The overall variation of the recorded electrical resistance is ranged from 2146 ± 3 Ohm to 2988 ± 4 Ohm at 1 cm and 21 cm depth respectively. Similarly, the variation of the electrical resistivity with the depth over the time for 2% kaolinite is identified in Figure 4.65. The resistivity increased over the time for all the tested depths due to the decrease in the conductivity because of kaolinite particle settling. The overall variation of the resistivity is ranged from 23.9 ± 0.01 Ohm.m at time zero and depth 1 cm to 26.7 ± 0.01 Ohm.m at time 1410 min and depth 21 cm.

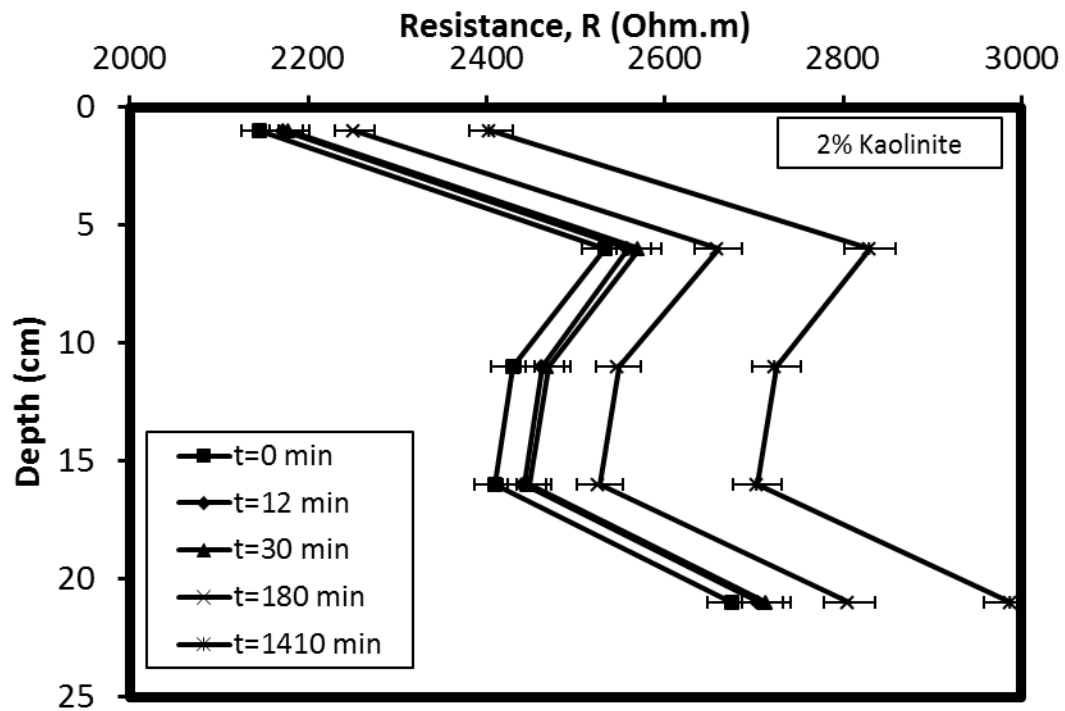


Figure 4.64 The variation of the electrical resistance with the depth over the time for 2% kaolinite soft soil.

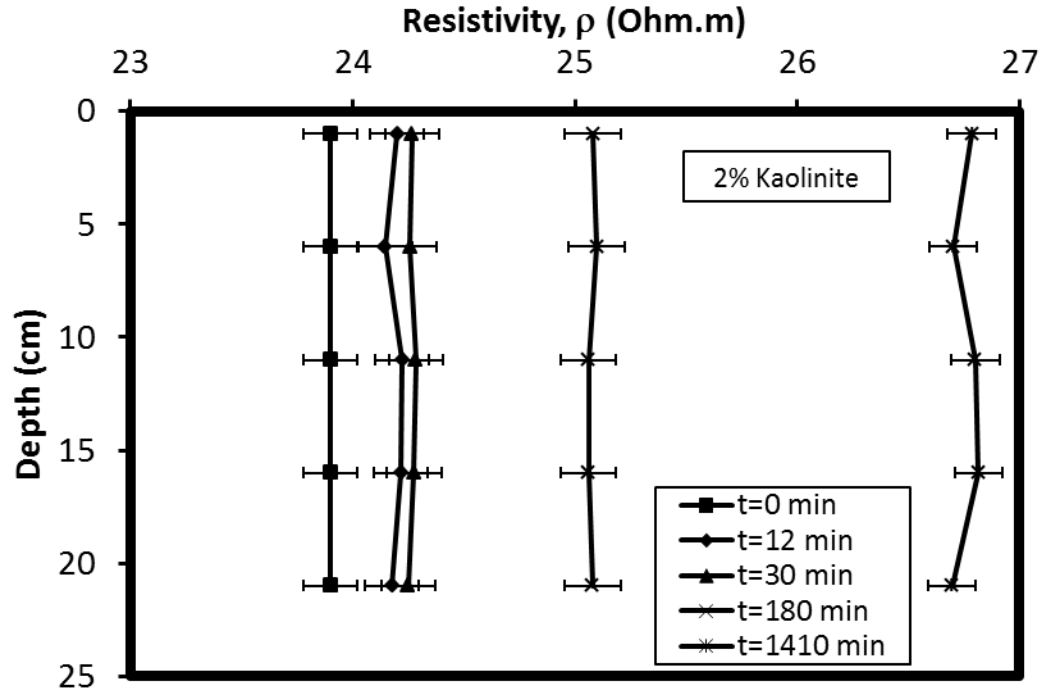


Figure 4.65 The variation of the electrical resistivity with the depth over the time for 2% kaolinite soft soil.

The variation of kaolinite concentration with the depth over the time is shown in Figure 4.66. As the time increased from 0 minutes to 1410 minutes, the kaolinite percent decreased from 2% to 0.1% and 2% to 0.15% at 1 cm and 21 cm depth respectively. The decrease in concentration is due to particle settling over the time.

The variation of the shear strength profile for 2% kaolinite soft soil with the depth over the time is drawn in Figure 4.67. As the time increased from 0 minutes to 1410 minutes, the shear strength decreased from 0.0097 ± 0.00001 kPa to 0.00641 ± 0.0000001 kPa along the whole depth. The decrease in the shear strength is due to particle settling over the time.

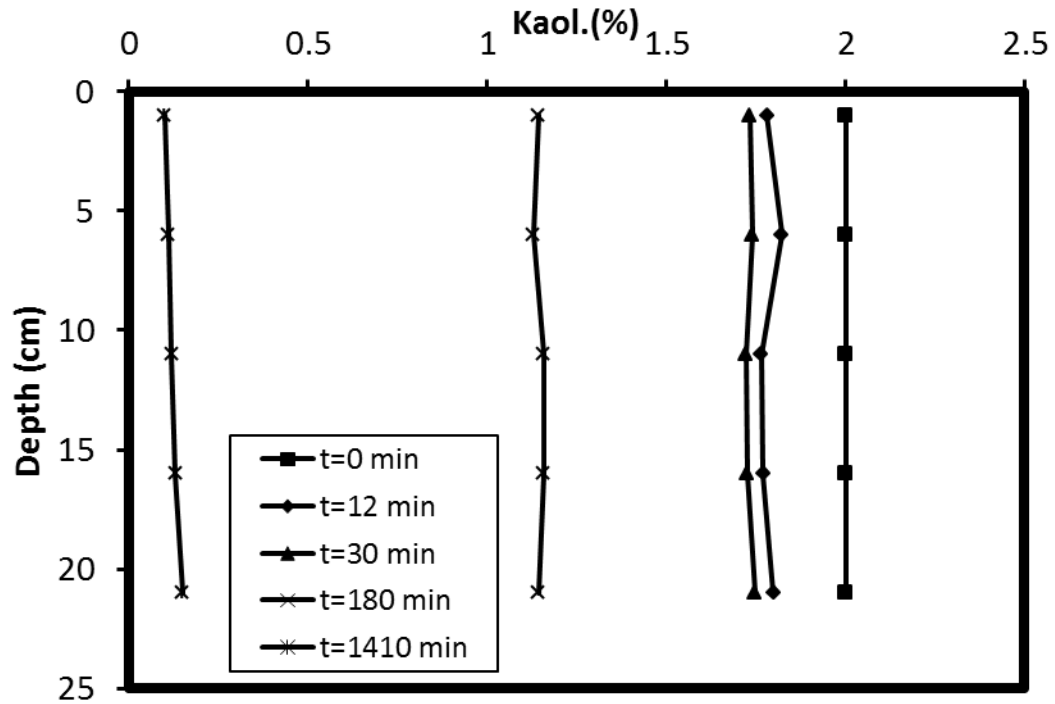


Figure 4.66 The variation of the kaolinite content with the depth over the time for 2% kaolinite soft soil.

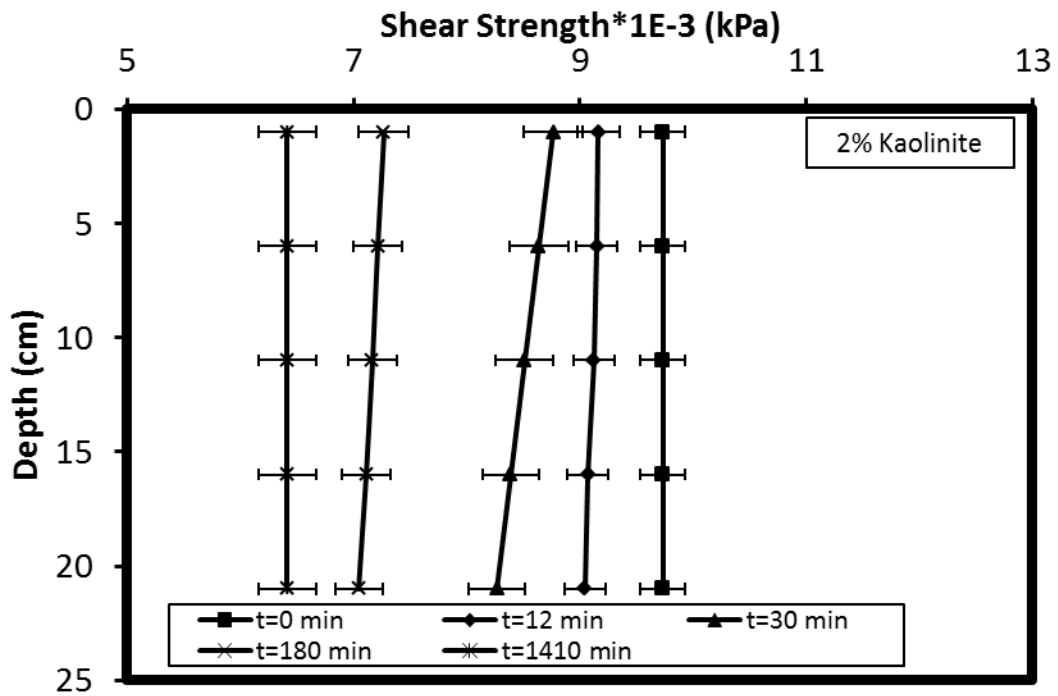


Figure 4.67 The variation of the shear strength with the depth over the time for 2% kaolinite soft soil.

Large-scale testing is performed to quantify the shear strength profile of 5% bentonite soft soil using a container with dimensions of 40 cm (dia.) and 100 cm (height). A series of wires were embedded in the vertical direction and in all four corners as shown in Figure 4.68.

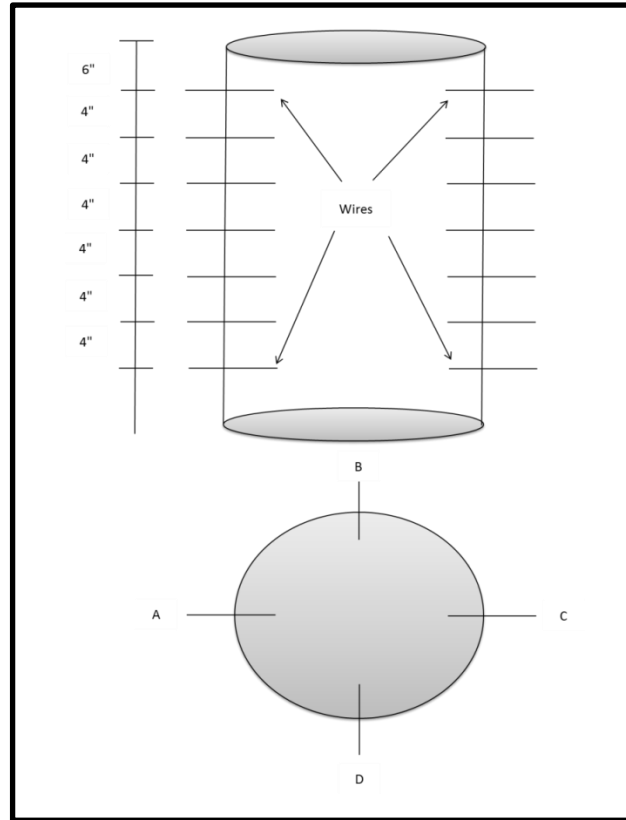


Figure 4.68 Schematic diagram for the large scale testing for 5% bentonite soft soil.

The variation of the electrical resistance with the depth over the time for 5% bentonite is shown in Figure 4.69. The electrical resistance increased with the time due to the particle settling that made the medium less conductive. The overall variation of the recorded electrical resistance is ranged from 420 ± 2 Ohm to 700 ± 4 Ohm at 15 cm and 65 cm depth respectively. Similarly, the variation of the electrical resistivity with the depth

over the time for 5% bentonite is identified in Figure 4.70. The resistivity increased over the time for all the tested depths due to the decrease in the conductivity because of bentonite particle settling. The overall variation of the resistivity is ranged from 5 ± 0.01 Ohm.m at time zero and depth 15 cm to 6.15 ± 0.02 Ohm.m at time 2 days and depth 65 cm.

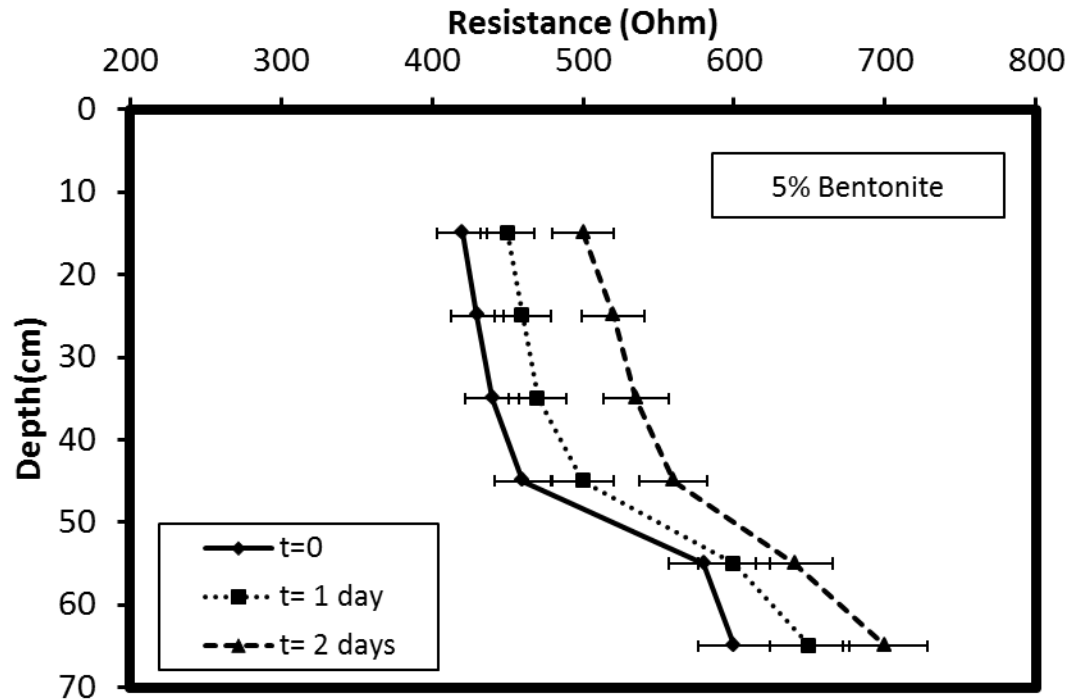


Figure 4.69 The variation of the electrical resistance with the depth over the time for 5% bentonite soft soil.

The variation of bentonite concentration with the depth over the time is shown in Figure 4.71. As the time increased from 0 minutes to 2 days, the bentonite percent decreased from 5% to 3.24% and 5% to 3.12% at 15 cm and 65 cm depth respectively. The decrease in concentration is due to particle settling over the time.

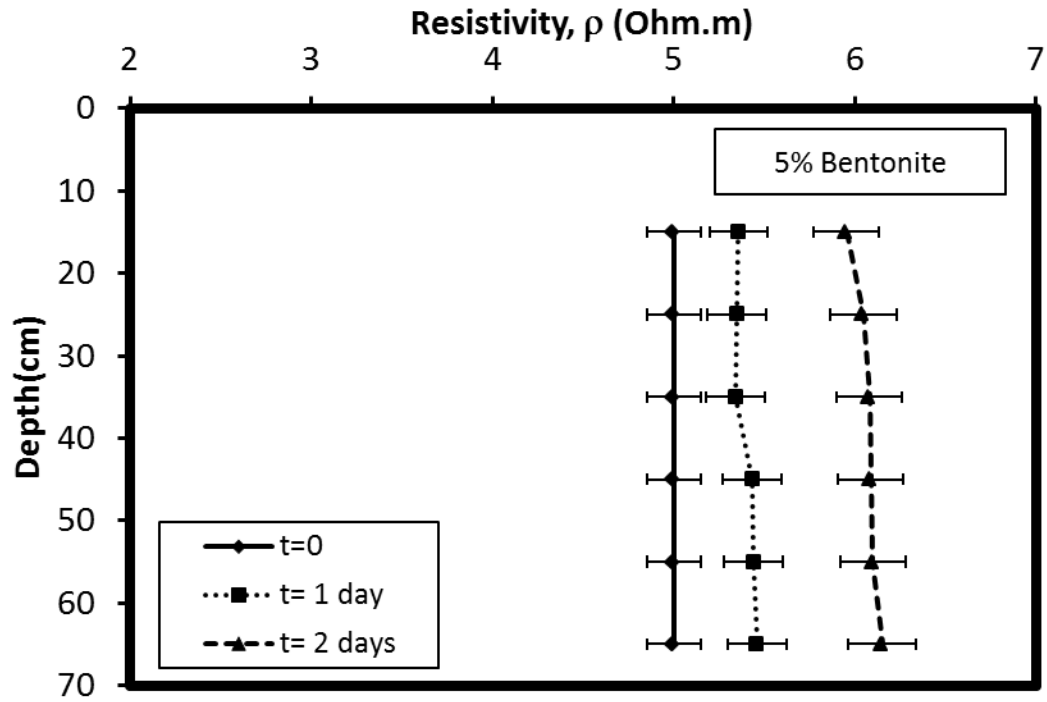


Figure 4.70 The variation of the electrical resistivity with the depth over the time for 2% bentonite soft soil.

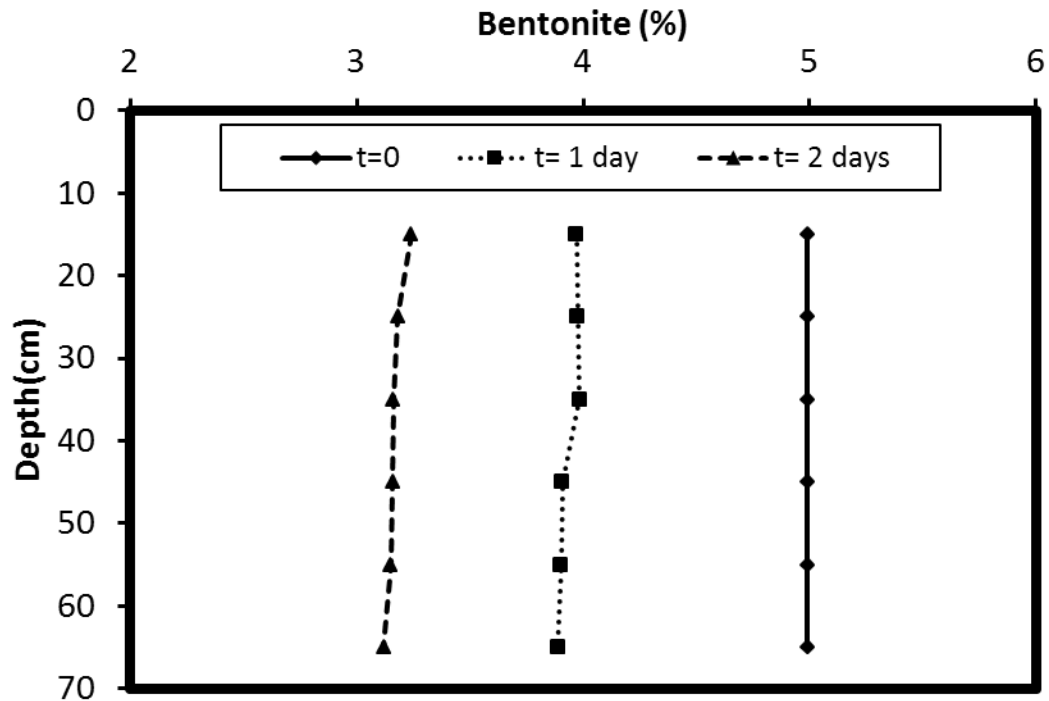


Figure 4.71 The variation of the bentonite content with the depth over the time for 5% bentonite soft soil.

The variation of the shear strength profile for 5% bentonite soft soil with the depth over the time is drawn in Figure 4.72. As the time increased from 0 minutes to 2 days, the shear strength decreased from 0.017 ± 0.00001 kPa to 0.0102 ± 0.000001 kPa and 0.0121 ± 0.000001 kPa at 15 cm and 65 cm depth respectively. The decrease in the shear strength is due to particle settling over the time.

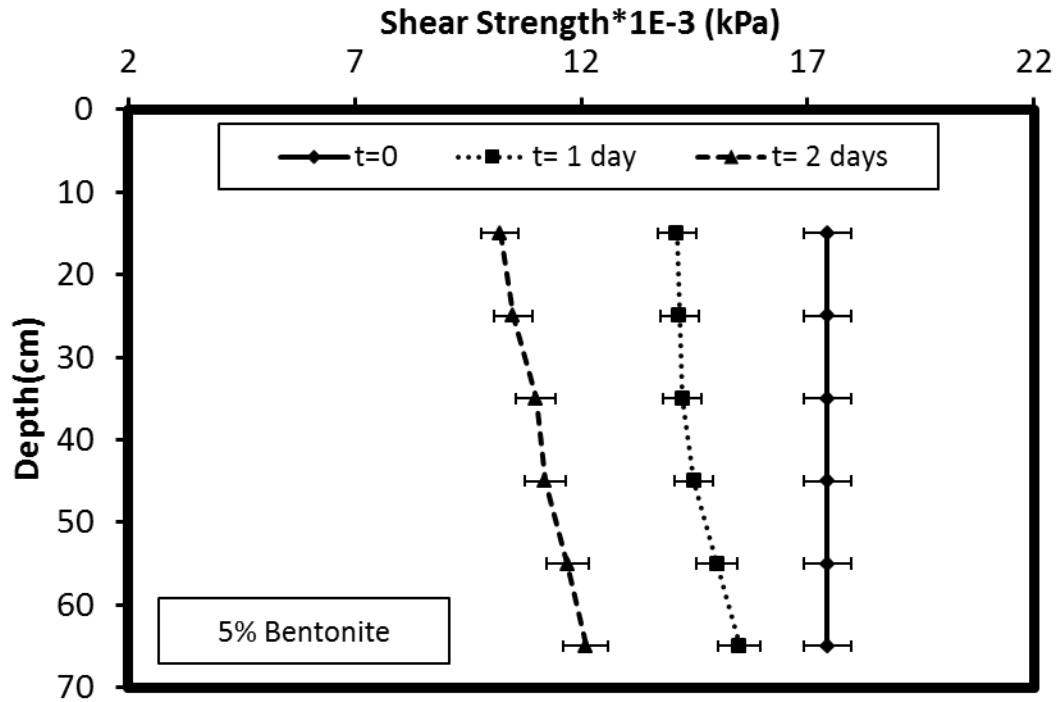


Figure 4.72 The variation of the shear strength with the depth over the time for 5% bentonite soft soil.

To test the sensing ability of the electrical resistivity method, a full-scale field test simulating oil well with dimensions of 50 cm diameter and 12 m depth was performed. A series of wires in longitudinal direction was attached to the surface of steel casing (30 cm diameter and 12 m depth) towards the spacing between the casing and the well with complete insulation from steel to prevent electricity flow through the steel casing. A schematic configuration for the oil well and steel casing is shown in Figure 4.73. The

testing procedure started by digging the well up to the required diameter and depth then an instrumented steel casing is placed carefully in the oil well. Then, the electrical resistance readings were taken at different depths where air was filling the gap between the well and the casing for the upper 6 m depth while drilling mud was filling the gap between the well and the casing for the lower 6 m depth. When the first round of readings was completed, the first cement slurry mixture was placed in the well where the cement positioned in the lower part of the well pushing the mud to be on the top of the cement slurry due to the differences in the density between the cement and mud. Then, the second round of electrical resistance readings was recorded. Finally, the second cement slurry mixture was placed in the well and the mud was raised to higher level in the well. Then, the third round of electrical resistance readings was recorded. The variation of the electrical resistance with the depth for the field oil well at $t = 0$ is represented in Figure 4.74. The electrical resistance for the upper 6 m is constant and 60 times the electrical resistance for the lower 6 m where air and drilling mud is exist in the upper and lower parts of the oil well respectively. There are some variations in the electrical resistance for the lower 6 m depth and the upper and lower limits are 300 ± 0.1 Ohm and 180 ± 0.1 Ohm at 7.2 m and 6.3 m depth respectively. The variation of the electrical resistance with the depth for the field oil well at $t = 0$ is represented in Figure 4.74. The electrical resistance for the upper 6 m is constant and 60 times the electrical resistance for the lower 6 m where air and drilling mud is exist in the upper and lower parts of the oil well respectively. There are some variations in the electrical resistance for the lower 6 m depth and the upper and lower limits are 300 ± 0.1 Ohm and 180 ± 0.1 Ohm at 7.2 m and 6.3 m depth respectively.

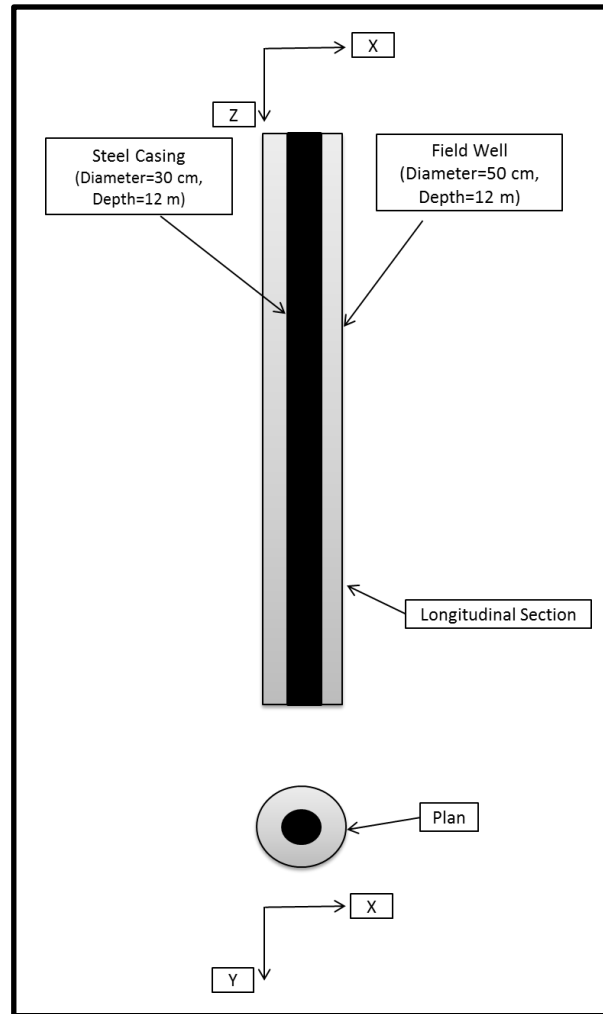


Figure 4.73 Schematic configuration of the field oil well with steel casing.

The variation of the electrical resistance with the depth for the field oil well at $t = 0.6$ hr is represented in Figure 4.75. The material distribution in the well were air, drilling mud and cement at depths 0 m to 1.8 m, 2.7 m to 8.1 m, and 9 m to 12 m respectively. The electrical resistance for the upper 1.8 m is constant and 72 times the electrical resistance of the drilling mud zone and 364 times the electrical resistance of the cement zone. The electrical resistance variations for the drilling mud were 250 ± 1 Ohm and 149.7 ± 0.5 Ohm at 4.5 m and 8.1 m respectively. The electrical resistance variations for the cement were 49.4 ± 0.1 Ohm and 39.8 ± 0.1 Ohm at 12 m and 10.8 m respectively.

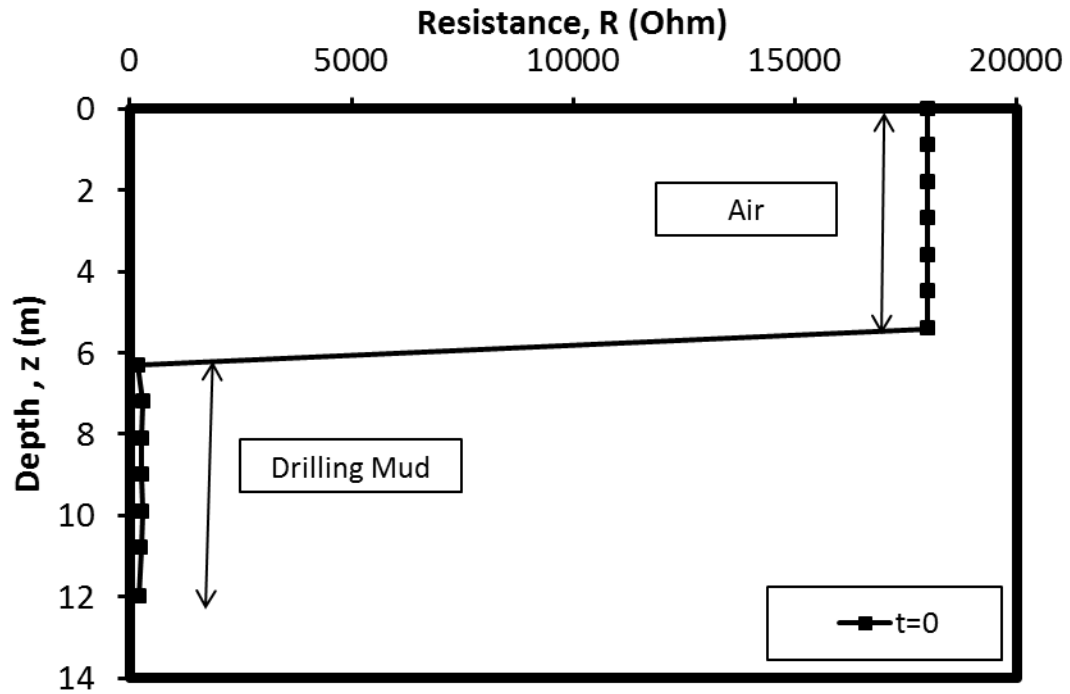


Figure 4.74 Variation of the electrical resistance with the depth for the field oil well at $t = 0$.

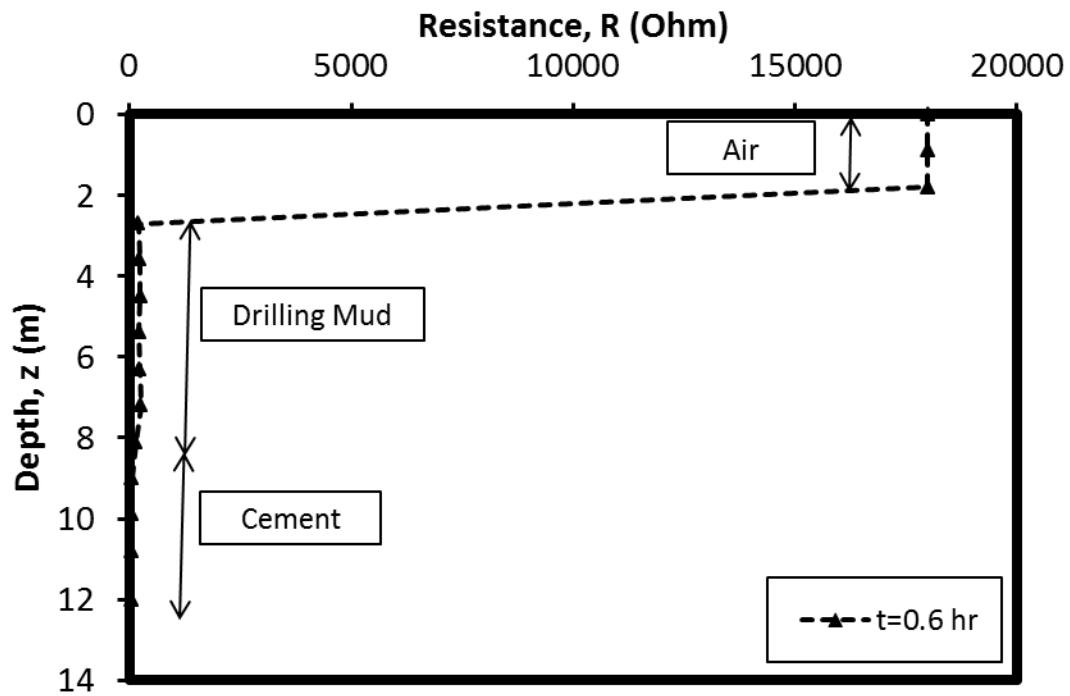


Figure 4.75 Variation of the electrical resistance with the depth for the field oil well at $t = 0.6$ hr.

The variation of the electrical resistance with the depth for the field oil well at $t = 1$ hr is represented in Figure 4.76. The material distribution in the well were drilling mud and cement at depths 0 m to 4.5 m, and 5.4 m to 12 m respectively. The maximum electrical resistance for the drilling mud zone is 8.4 times the maximum electrical resistance of the cement zone. The electrical resistance variations for the drilling mud were 240 ± 1 Ohm and 150 ± 1 Ohm at 2.7 m and 0 m respectively. The electrical resistance variations for the cement were 28.6 ± 0.1 Ohm and 22.9 ± 0.1 Ohm at 12 m and 6.3 m respectively.

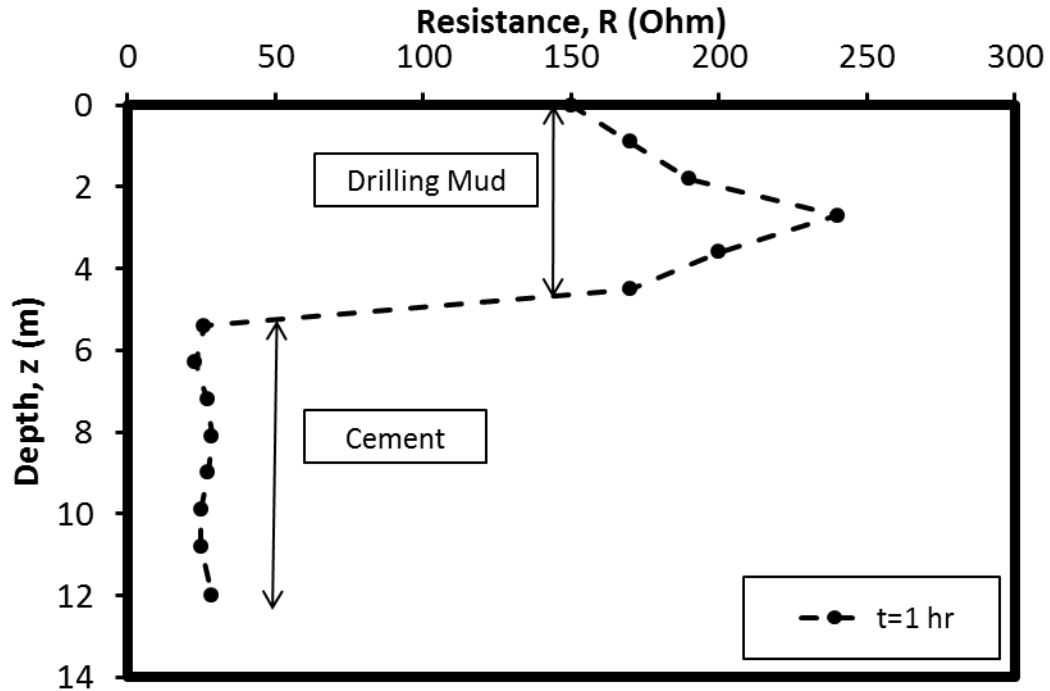


Figure 4.76 Variation of the electrical resistance with the depth for the field oil well at $t = 1$ hr.

From Figure 4.74, Figure 4.75, and Figure 4.76, the sensitivity of using the electrical resistance can be concluded where the variation of the air, drilling mud, and cement depths in the field oil well were captured precisely.

4.6.Long-term soft soil improvement

The efficiency for any material treatment is measured by its ability for long-term performance. Hence, it is required to study the effect of the soft soil treatment agents such as lime, and polymer for long-term evaluation. The most important factors can be studied through the effect of the lime and polymer on both the shear strength and water content for the 10% bentonite treated soft soil and up to 28 days. The treated samples were studied under two different curing systems that are air-closed (closed-curing) and air-curing systems. The long-term variation of the shear strength with the lime content for the 10% bentonite soft soil treated with the lime under closed-curing system is shown in Figure 4.77. Generally, as the lime content increased, the shear strength for the 10% bentonite soft soil increased and higher shear strength can be noticed for longer curing time. As the curing time increased from 0 day to 28 days, the shear strength increased by 95% and 1959% for 0% and 20% lime contents respectively. As the lime content increased from 0% to 20%, the shear strength increased by 14% and 1103% at 0 day and 28 days curing time respectively. The long-term variation of the water content with the lime content for the 10% bentonite soft soil treated with the lime under closed-curing system is shown in Figure 4.78. Generally, as the lime content increased, the water content for the 10% bentonite soft soil decreased due to the increase for the solid content in the medium. As the curing time increased from 0 day to 28 days, the water content decreased by 7% and 37% for 0% and 20% lime contents respectively. As the lime content increased from 0% to 20%, the water content decreased by 22% and 47% at 0 day and 28 days curing time respectively.

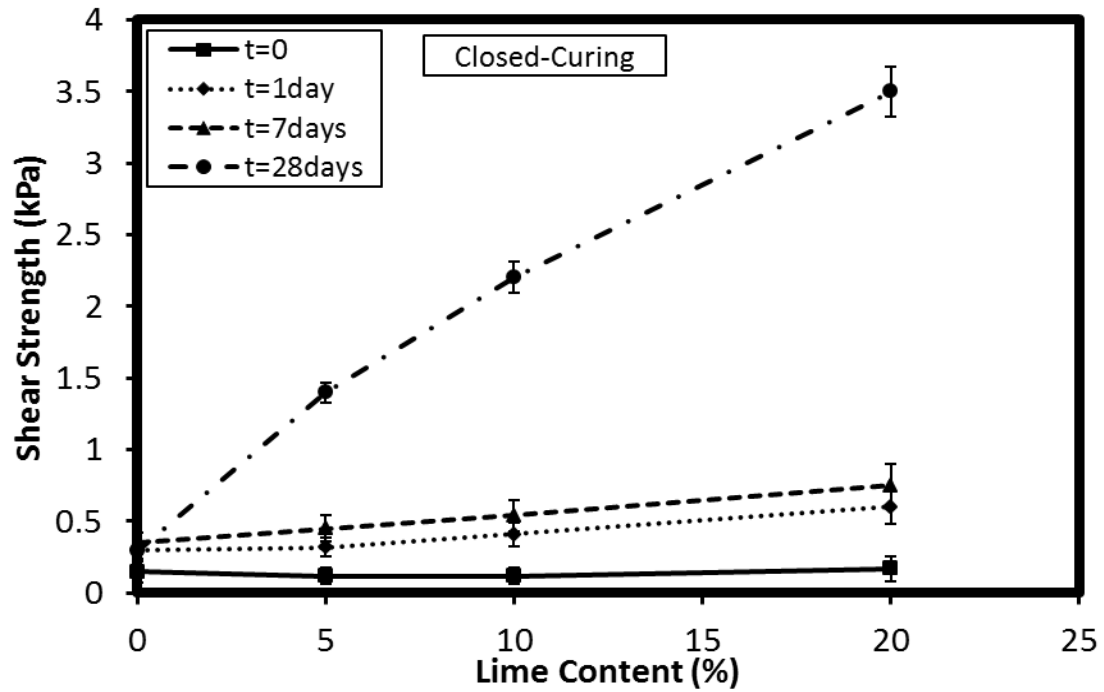


Figure 4.77 Long-term variation of shear strength with lime content for 10% bentonite soft soil treated with lime in closed-air curing system.

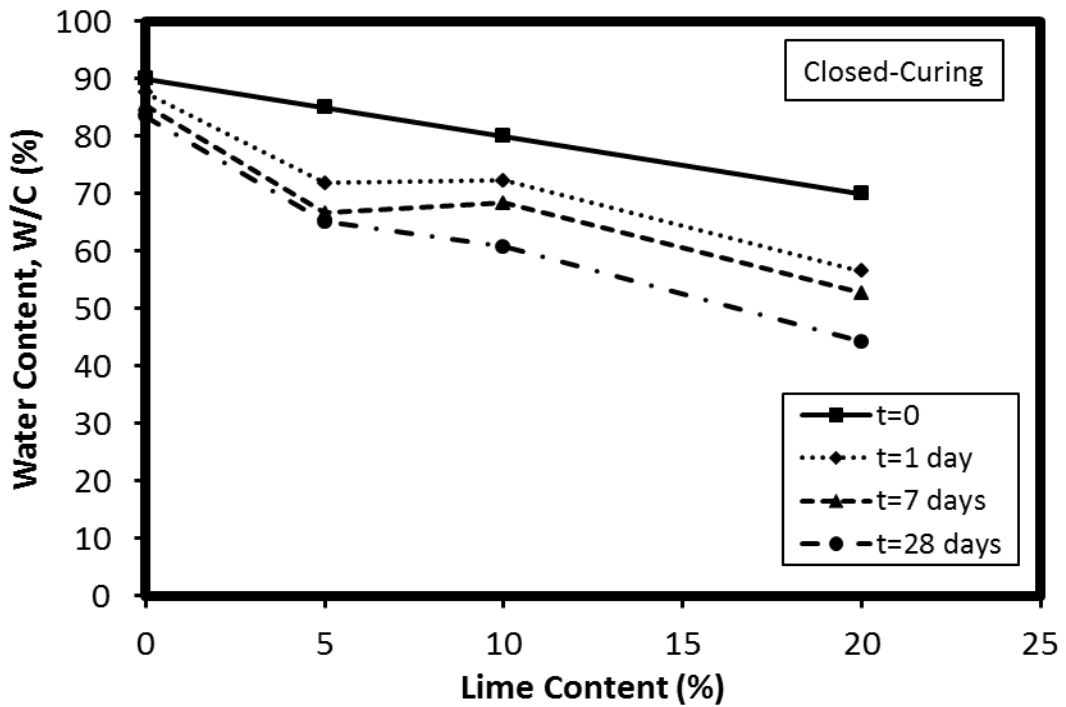


Figure 4.78 Long-term variation of water content with lime content for 10% bentonite soft soil treated with lime in closed-air curing system.

The long-term variation of the shear strength with the lime content for the 10% bentonite soft soil treated with the lime under air-curing system is shown in Figure 4.79. Generally, as the lime content increased, the shear strength for the 10% bentonite soft soil increased and higher shear strength can be noticed for longer curing time. As the curing time increased from 0 day to 28 days, the shear strength increased by 487% and 3005% for 0% and 20% lime contents respectively. As the lime content increased from 0% to 20%, the shear strength increased by 14% and 504% at 0 day and 28 days curing time respectively. The long-term variation of the water content with the lime content for the 10% bentonite soft soil treated with the lime under air-curing system is shown in Figure 4.80. Generally, as the lime content increased, the water content for the 10% bentonite soft soil decreased due to the increase for the solid content in the medium. As the curing time increased from 0 day to 28 days, the water content decreased by 33% and 73% for 0% and 20% lime contents respectively. As the lime content increased from 0% to 20%, the water content decreased by 68% and 87% at 0 day and 28 days curing time respectively. As the curing condition changed from closed-curing to air-curing, the shear strength increased by 200% and 51% for 0% and 20% lime contents at 28 days curing time respectively. As the curing condition changed from closed-curing to air-curing, the water content decreased by 27% and 57% for 0% and 20% lime contents at 28 days curing time respectively.

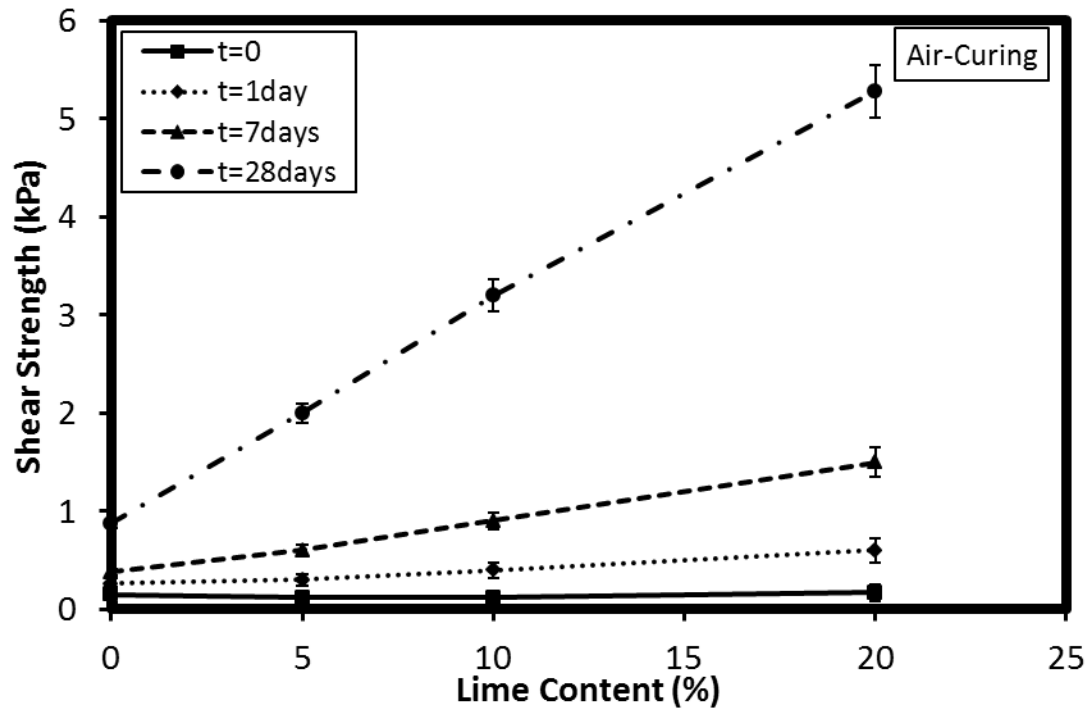


Figure 4.79 Long-term variation of shear strength with lime content for 10% bentonite soft soil treated with lime in air-curing system.

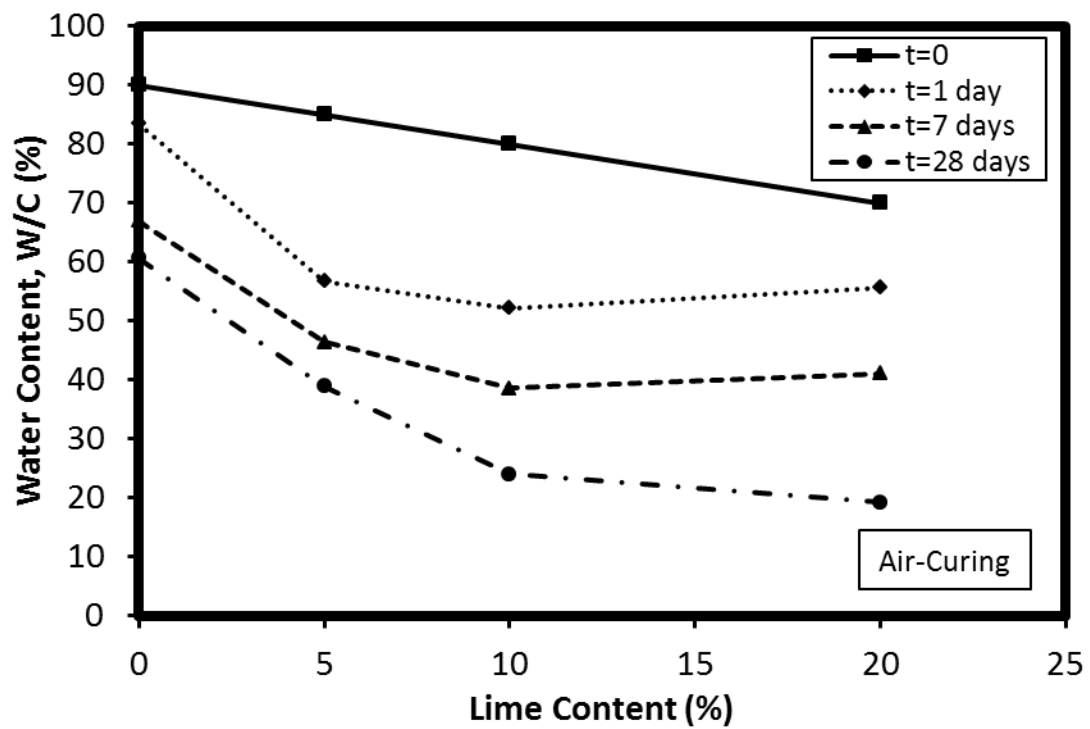


Figure 4.80 Long-term variation of water content with lime content for 10% bentonite soft soil treated with lime in air-curing system.

The long-term variation of the shear strength with the polymer content for the 10% bentonite soft soil treated with the polymer under closed-curing system is shown in Figure 4.81. Generally, as the polymer content increased, the shear strength for the 10% bentonite soft soil increased and higher shear strength can be noticed for longer curing time. As the curing time increased from 0 day to 28 days, the shear strength increased by 95% and 313% for 0% and 5% polymer contents respectively. As the polymer content increased from 0% to 5%, the shear strength increased by 632% and 1446% at 0 day and 28 days curing time respectively. The long-term variation of the water content with the polymer content for the 10% bentonite soft soil treated with the polymer under closed-curing system is shown in Figure 4.82. Generally, as the polymer content increased, the water content for the 10% bentonite soft soil decreased. As the curing time increased from 0 day to 28 days, the water content decreased by 7% and 25% for 0% and 5% polymer contents respectively. As the polymer content increased from 0% to 5%, the water content decreased by 6% and 23% at 0 day and 28 days curing time respectively. As the treatment agent changed from 20% lime to 5% polymer in closed-curing system, the shear strength increased by 541% and 29% at 0 day and 28 days curing time respectively. As the treatment agent changed from 20% lime to 5% polymer in closed-curing system, the water content increased by 21% and 44% at 0 day and 28 days curing time respectively.

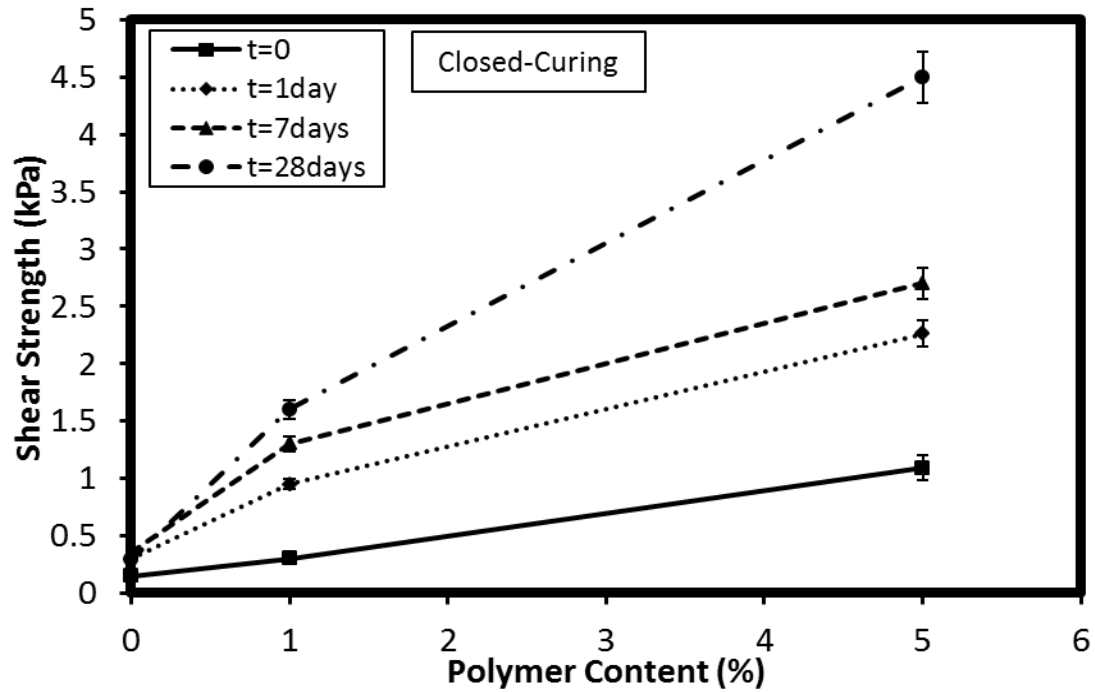


Figure 4.81 Long-term variation of shear strength with polymer content for 10% bentonite soft soil treated with lime in closed-air curing system.

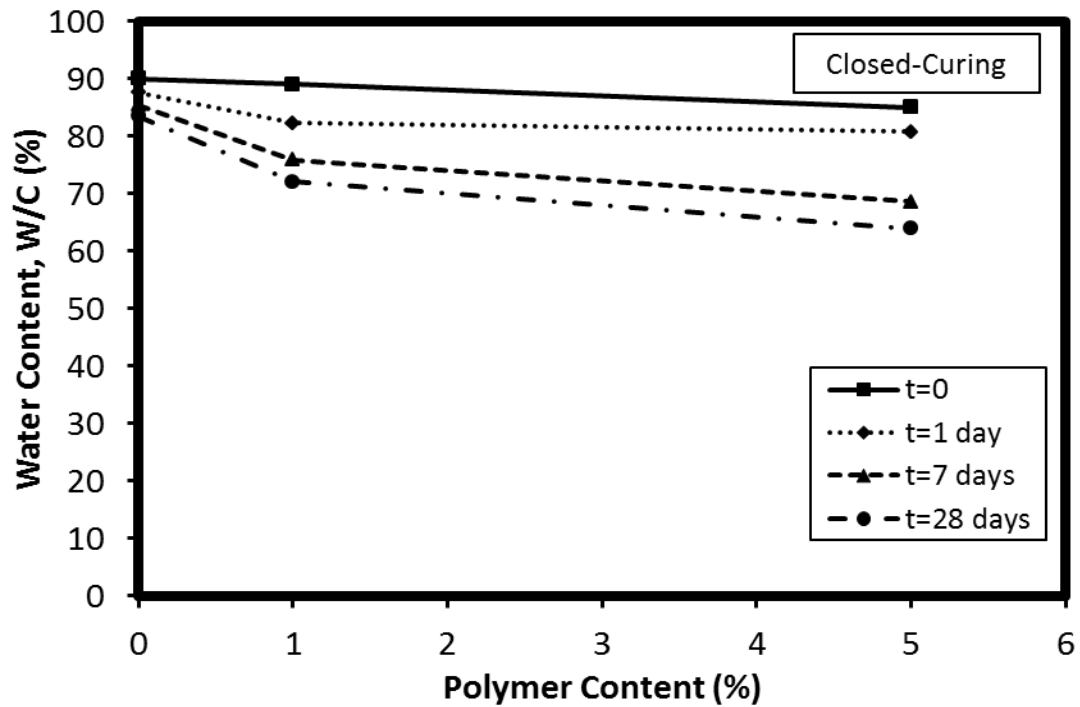


Figure 4.82 Long-term variation of water content with polymer content for 10% bentonite soft soil treated with lime in closed-air curing system.

The long-term variation of the shear strength with the polymer content for the 10% bentonite soft soil treated with the polymer under air-curing system is shown in Figure 4.83. Generally, as the polymer content increased, the shear strength for the 10% bentonite soft soil increased and higher shear strength can be noticed for longer curing time. As the curing time increased from 0 day to 28 days, the shear strength increased by 571% and 537% for 0% and 5% polymer contents respectively. As the polymer content increased from 0% to 5%, the shear strength increased by 636% and 600% at 0 day and 28 days curing time respectively. The long-term variation of the water content with the polymer content for the 10% bentonite soft soil treated with the polymer under air-curing system is shown in Figure 4.84. Generally, as the polymer content increased, the water content for the 10% bentonite soft soil decreased. As the curing time increased from 0 day to 28 days, the water content decreased by 33% and 46% for 0% and 5% polymer contents respectively. As the polymer content increased from 0% to 5%, the water content decreased by 6% and 25% at 0 day and 28 days curing time respectively. As the curing condition changed from closed-curing to air-curing, the shear strength increased by 244% and 56% for 0% and 5% polymer contents at 28 days curing time respectively. As the curing condition changed from closed-curing to air-curing, the water content decreased by 27% and 28% for 0% and 5% polymer contents at 28 days curing time respectively. As the treatment agent changed from 20% lime to 5% polymer in air-curing system, the shear strength increased by 545% and 33% at 0 day and 28 days curing time respectively. As the treatment agent changed from 20% lime to 5% polymer in air-curing system, the water content decreased by 8% and 137% at 0 day and 28 days curing time respectively.

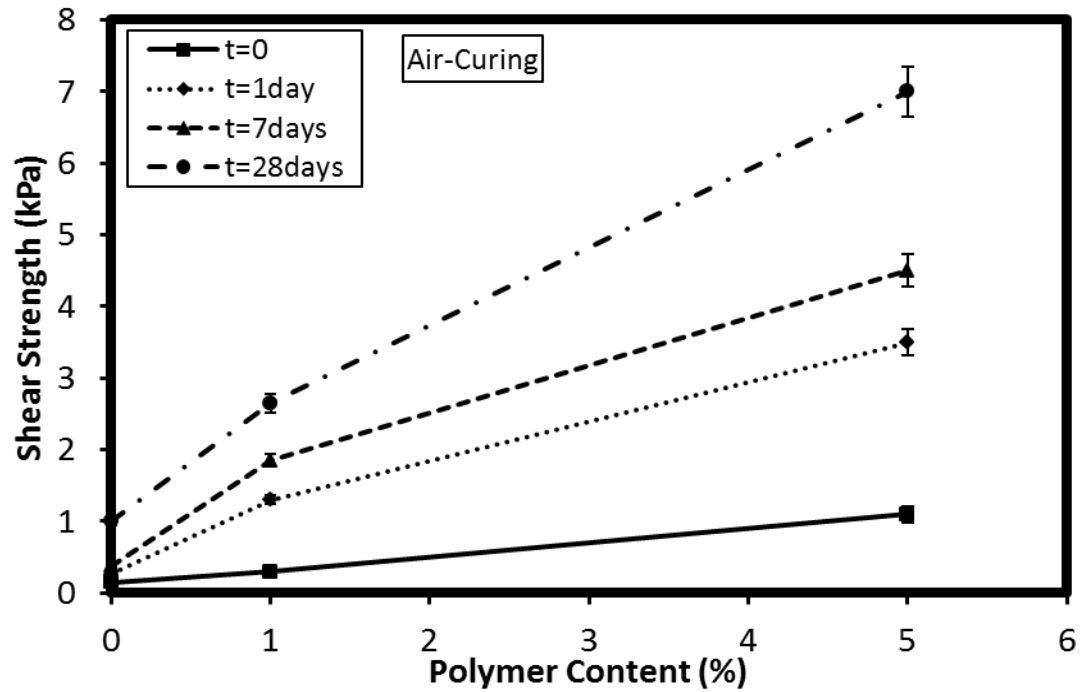


Figure 4.83 Long-term variation of shear strength with polymer content for 10% bentonite soft soil treated with polymer in air-curing system.

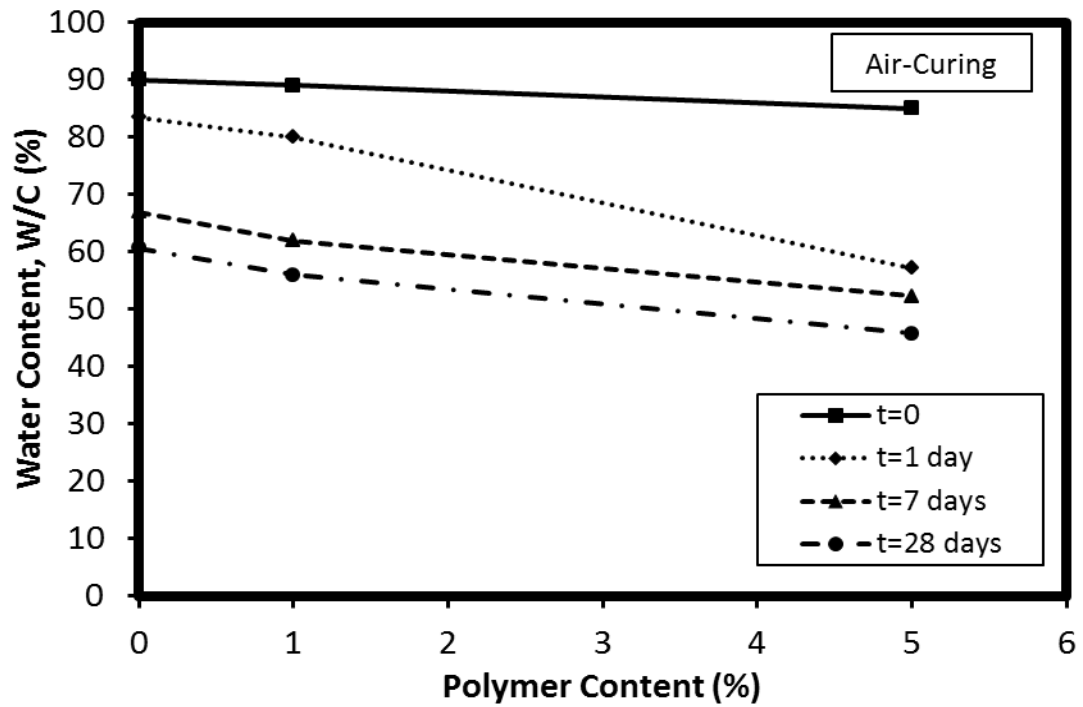


Figure 4.84 Long-term variation of water content with polymer content for 10% bentonite soft soil treated with polymer in air-curing system.

Due the popularity of using lime as soil improvement agent, the study of the 10% bentonite treated soft soil with lime is extended up to 180 days in closed-curing system. Both the shear strength and water content for 10% bentonite treated soft soil with lime is investigated. The comparison between the short and long-term variations of the shear strength versus lime content for 10% bentonite soft soil treated with lime in closed-curing system is shown in Figure 4.85. As the curing time increased from 0 day to 180 days, the shear strength increased by 645% and 10125% for 0% and 10% lime content respectively. As the lime content increased from 0% to 10%, the shear strength was almost constant at 0 day curing time. As the lime content increased from 0% to 10%, the shear strength increased by 1273% at 180 days curing time. The comparison between the short and long-term variations of the water content versus shear strength for 10% bentonite soft soil treated with lime in closed-curing system is shown in Figure 4.86. As the lime content increased from 0% to 10%, the water content decreased by 69% at 28 days curing time. The long-term results have shown that the treated soft soil shear strength is increasing with the time, the moisture content is decreasing with time, and the polymer is more effective than the lime as a treatment agent.

4.7. Miniature CIGMAT Penetrometer

In this section, the penetrations of CIGMAT penetrometer with different weights on the shear strength of both untreated and treated ultra-soft soil has been investigated. The untreated ultra-soft soil consisted of 2% to 10% bentonite while the treated ultra-soft soil consisted of 10% bentonite treated with different percentage of lime and polymer. The CIGMAT penetrometer weights were 7 gm, 14 gm, 21 gm, and 28 gm.

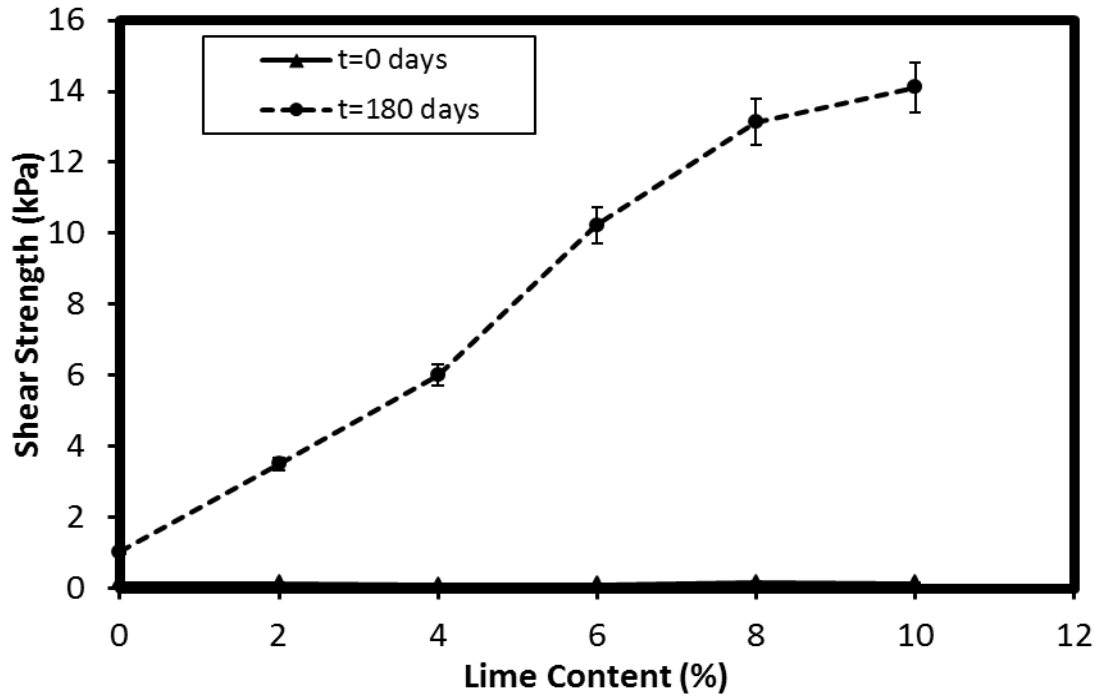


Figure 4.85 Comparison between the short and long-term variations of the shear strength versus lime content for 10% bentonite soft soil treated with lime in closed-curing system.

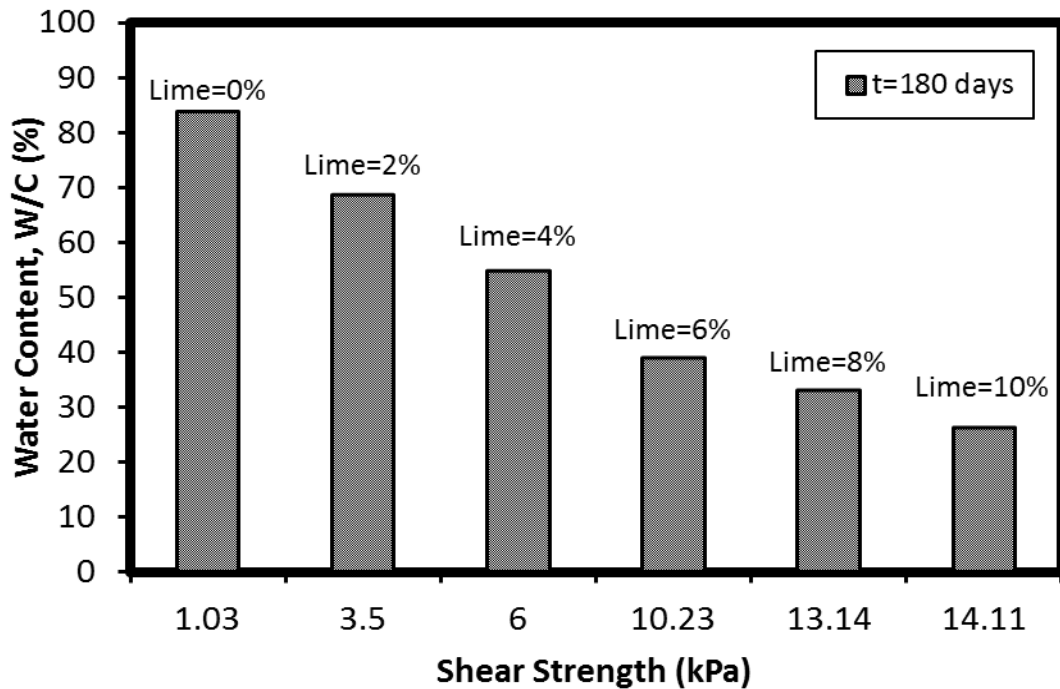


Figure 4.86 Comparison between the short and long-term variations of the water content versus shear strength for 10% bentonite soft soil treated with lime in closed-curing system.

4.7.1. Penetration versus weight

(i) Untreated ultra-soft soil

The variation of CIGMAT penetrometer penetration versus different weights of CIGMAT penetrometer of untreated ultra-soft soil has been shown in Figure 4.87. As the penetrometer weight increased the penetration increased regardless of the solid content in the ultra-soft soil while for a constant penetrometer weight, the penetration decreased as the bentonite content increased due to the increase in ultra-soft soil strength. As the penetrometer weight increased from 7 gm to 28 gm, the penetration increased by 146%, 157%, 160%, 300% and 400% for 2%, 4%, 6%, 8% and 10% bentonite content respectively. As the bentonite content increased from 2% to 10%, the penetrometer penetration decreased by 85%, 79%, 71% and 69% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.

(ii) Treated ultra-soft soil with lime

The variation of CIGMAT penetrometer penetration versus different weights of CIGMAT penetrometer of treated bentonite ultra-soft soil with lime has been shown in Figure 4.88. As the penetrometer weight increased from 7 gm to 28 gm, the penetration increased by 67%, 56%, 55%, 48% and 44% for 2%, 4%, 6%, 8% and 10% lime content respectively. As the lime content increased from 2% to 10%, the penetrometer penetration increased by 73%, 61%, 57% and 50% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.

(iii) Treated ultra-soft soil with polymer

The variation of CIGMAT penetrometer penetration versus different weights of CIGMAT penetrometer of treated bentonite ultra-soft soil with polymer has been shown in Figure 4.89. As the penetrometer weight increased from 7 gm to 28 gm, the penetration

increased by 50%, 56%, and 500% for 1%, 5% and 10% polymer content respectively. As the polymer content increased from 1% to 10%, the penetrometer penetration decreased by 96%, 87%, 84% and 83% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.

(iii) Correlation

A linear correlation between the CIGMAT miniature penetration and weight has been proposed as

$$\delta(\text{mm}) = A_2 + B_2 * P(\text{gm}), \quad (4.9)$$

where δ (mm) is the CIGMAT penetration, A_2 (mm) & B_2 (mm/gm) are model parameters, and P (gm) is CIGMAT penetrometer weight.

A summary of proposed correlation (Eq. 4.9) details for both untreated and treated ultra-soft soil is summarized in Table 4. 22.

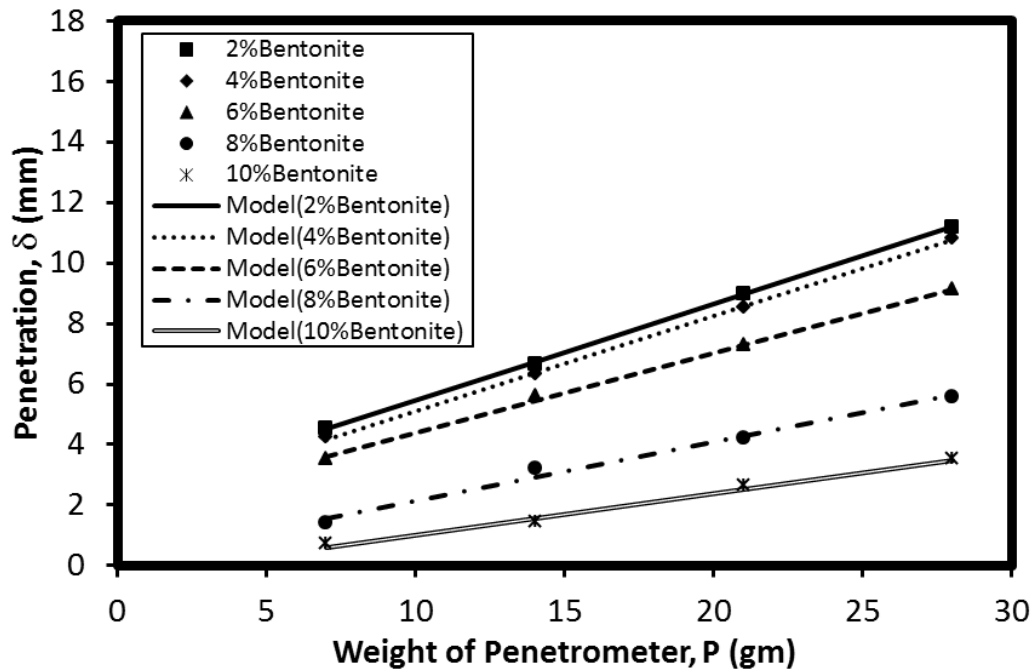


Figure 4.87 Variation of CIGMAT penetration and CIGMAT weight of untreated ultra-soft soil.

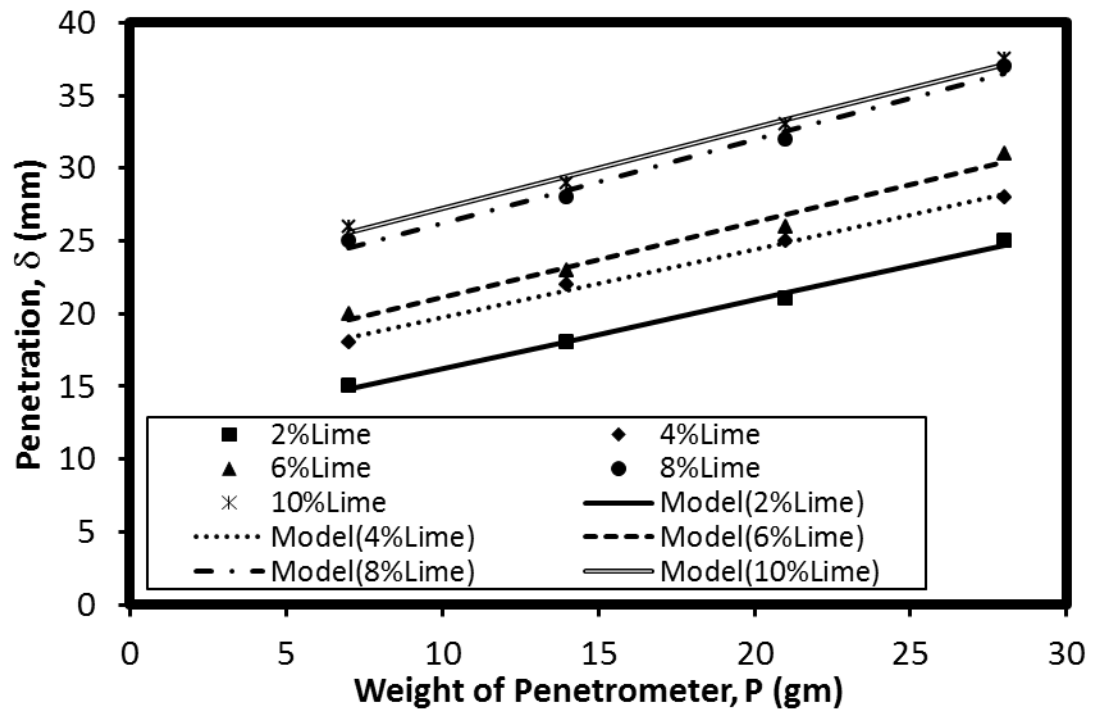


Figure 4.88 Variation of CIGMAT penetration and CIGMAT weight of treated ultra-soft soil with lime.

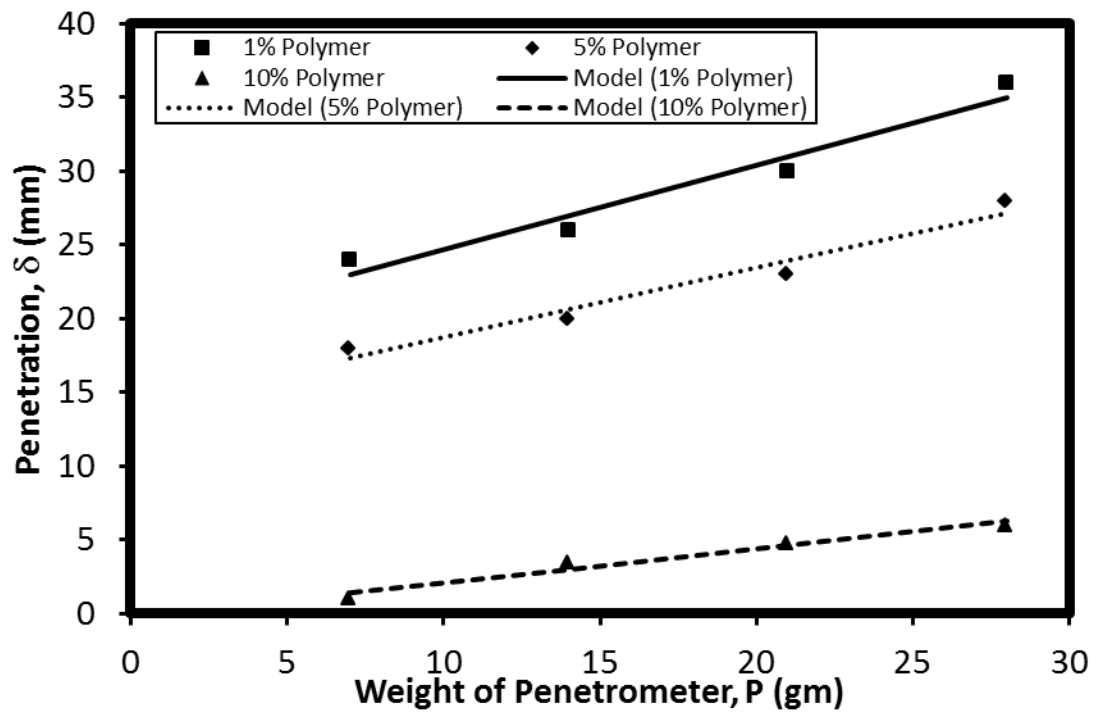


Figure 4.89 Variation of CIGMAT penetration and CIGMAT weight of treated ultra-soft soil with polymer.

Table 4. 22 Correlation parameters (Eq. 4.9) of untreated and treated ultra-soft soil.

Untreated ultra-soft soil	A₂ (mm)	B₂ (mm/gm)	R²	RMSE (mm)
2% bentonite	2.28	0.39	0.99	0.051
4% bentonite	1.95	0.31	0.99	0.05
6% bentonite	1.75	0.26	0.99	0.095
8% bentonite	0.2	0.19	0.99	0.167
10% bentonite	-0.35	0.14	0.99	0.102
Treated ultra-soft soil (10% bentonite+lime)	A₂ (mm)	B₂ (mm/gm)	R²	RMSE (mm)
2% lime	11.5	0.47	0.99	0.271
4% lime	15	0.47	0.99	0.273
6% lime	16	0.51	0.99	0.548
8% lime	20.5	0.57	0.99	0.5
10% lime	21.8	0.55	0.99	0.379
Treated ultra-soft soil (10% bentonite+polymer)	A₂ (mm)	B₂ (mm/gm)	R²	RMSE (mm)
1% polymer	19	0.57	0.95	1
5% polymer	14	0.47	0.99	0.758
10% polymer	-0.25	0.23	0.97	0.347

4.7.2. Shear strength versus penetration

(i) Untreated ultra-soft soil

The variation of the shear strength versus CIGMAT penetration of untreated ultra-soft soil has been shown in Figure 4.90. As the shear strength increased, the penetration decreased regardless of the penetrometer weight, and for a constant shear strength, the penetration increased as the CIGMAT penetrometer weight increased. As the penetrometer weight increased from 7 gm to 28 gm, the penetration increased by 44% and 50% for a shear strength of 0.011 kPa and 0.17 kPa respectively. As the shear strength increased from 0.011 kPa to 0.17 kPa, the CIGMAT penetration decreased by 78% and 77% for CIGMAT penetrometer weights of 7 gm and 28 gm respectively.

(ii) Lime treatment

The variation of the shear strength versus CIGMAT penetration of treated ultra-soft soil with lime has been shown in Figure 4.91. As the penetrometer weight increased from 7 gm to 28 gm, the penetration increased by 48% and 66% for a shear strength of 0.14 kPa and 0.27 kPa respectively. As the shear strength increased from 0.14 kPa to 0.27 kPa, the CIGMAT penetration decreased by 40% and 32% for CIGMAT penetrometer weights of 7 gm and 28 gm respectively.

(iii) Polymer treatment

The variation of the shear strength versus CIGMAT penetration of treated ultra-soft soil with polymer has been shown in Figure 4.92. As the penetrometer weight increased from 7 gm to 28 gm, the penetration increased by 50% and 500% for a shear strength of 0.15 kPa and 6.8 kPa respectively. As the shear strength increased from 0.15 kPa to 6.8 kPa, the CIGMAT penetration decreased by 96% and 85% for CIGMAT penetrometer weights of 7 gm and 28 gm respectively.

(iv) Correlation

A linear correlation between the shear strength and CIGMAT penetrometer penetration has been proposed as

$$\text{Shear strength (kPa)} = D_1 + E_1 * \Delta(\text{mm}), \quad (4.10)$$

where $\Delta(\text{mm})$ is the CIGMAT penetration, D (kPa) & E (kPa/mm) are model parameters.

A summary of proposed correlation (Eq.5) details for both untreated and treated ultra-soft soil parameters are summarized in Table 4.23.

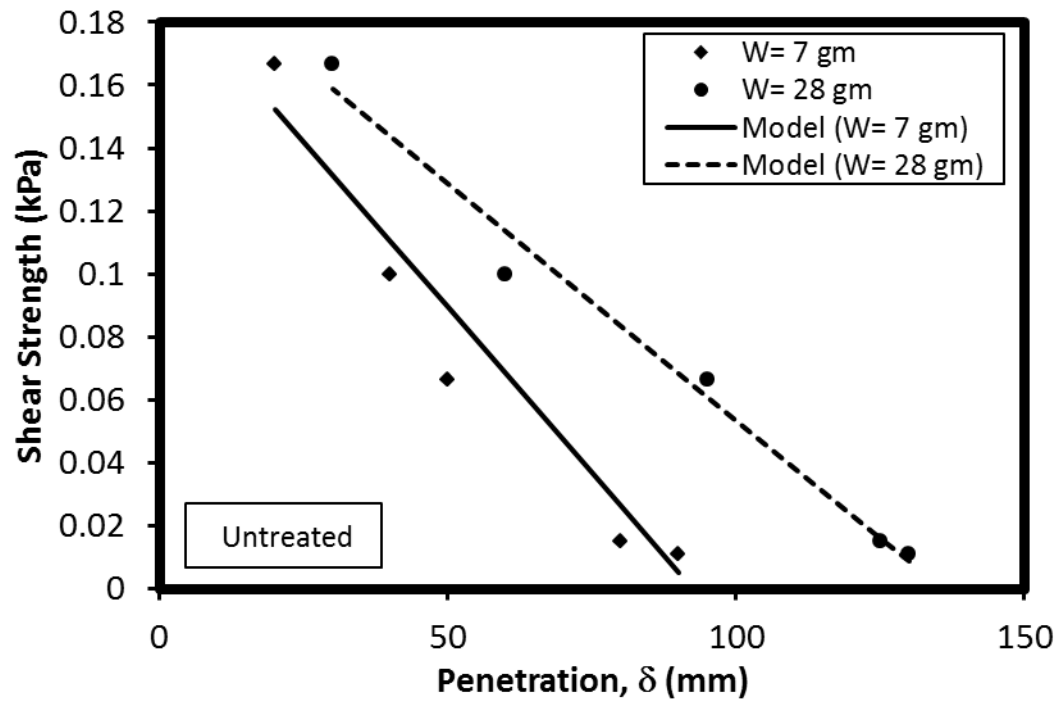


Figure 4.90 Variation of shear strength and CIGMAT penetrometer penetration of untreated ultra-soft soil.

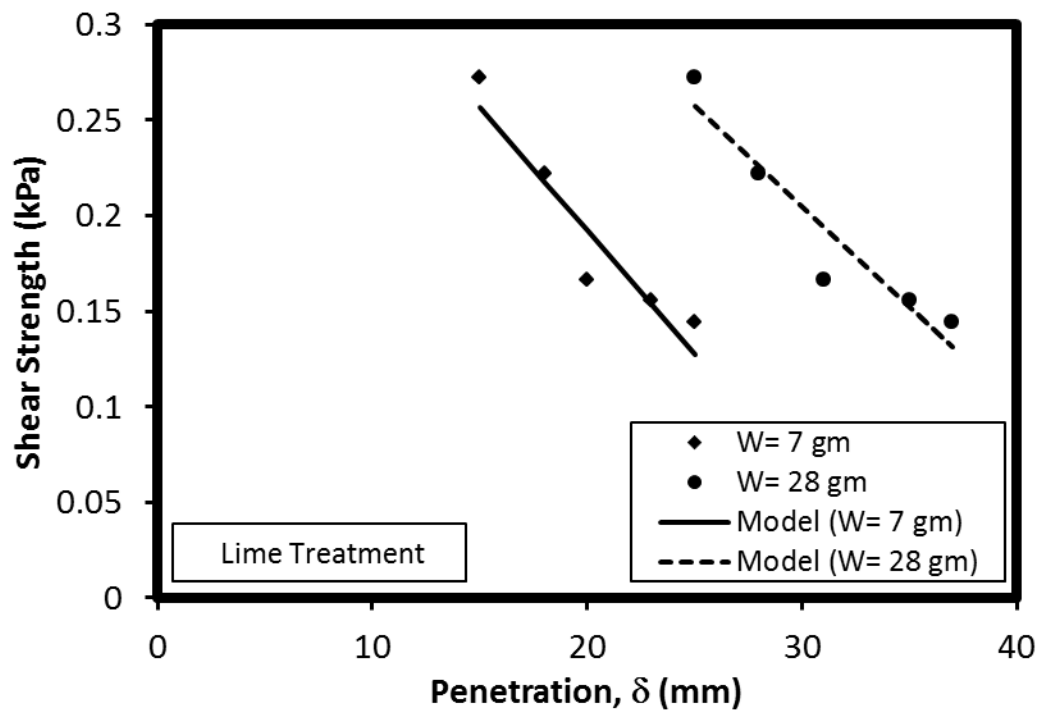


Figure 4.91 Variation of shear strength and CIGMAT penetrometer penetration of treated ultra-soft soil with lime.

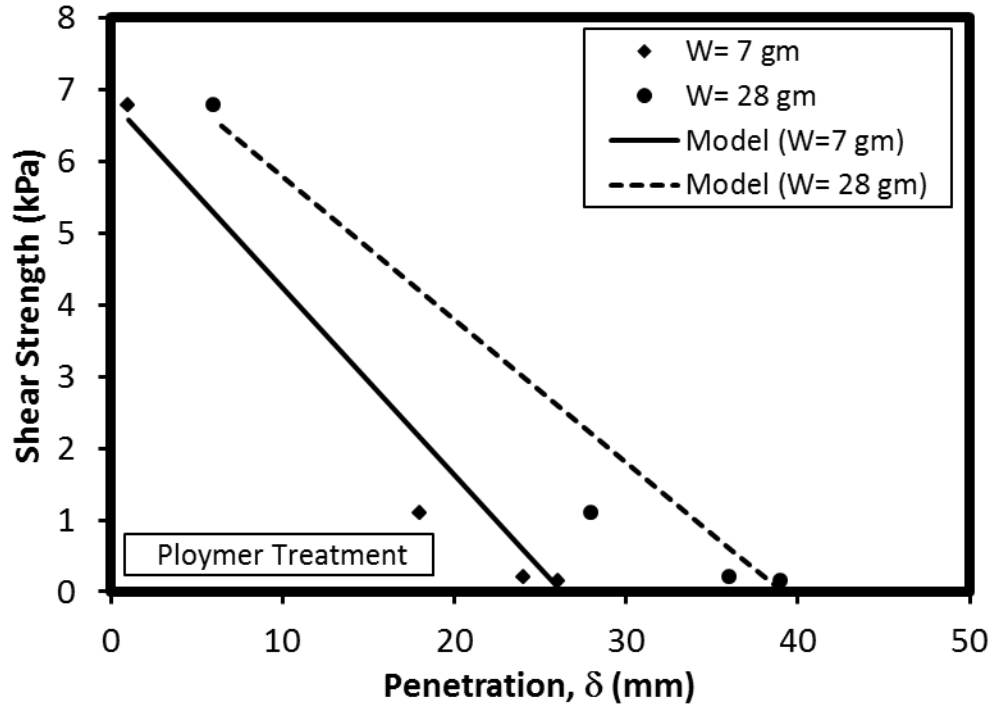


Figure 4.92 Variation of shear strength and CIGMAT penetrometer penetration of treated ultra-soft soil with polymer.

Table 4.23 Correlation parameters (Eq. 4.10) of untreated and treated ultra-soft soil

Untreated ultra-soft soil	D_1 (kPa)	E_1 (kPa/mm)	R^2	RMSE (kPa)
CIGMAT penetrometer weight (gm)				
7	0.19	-0.0021	0.94	0.014
28	0.20	-0.0015	0.98	0.0075
Treated ultra-soft soil (10% bentonite+lime)	D_1 (kPa)	E_1 (kPa/mm)	R^2	RMSE (kPa)
CIGMAT penetrometer weight (gm)				
7	0.45	-0.0129	0.90	0.0154
28	0.52	-0.0105	0.89	0.0153
Treated ultra-soft soil (10% bentonite+polymer)	D_1 (kPa)	E_1 (kPa/mm)	R^2	RMSE (kPa)
CIGMAT penetrometer weight (gm)				
7	6.83	-0.26	0.96	0.512
28	7.77	-0.20	0.95	0.536

4.7.3.Finite element modeling

CIGMAT miniature penetrometer

An elasto-plastic Mohr-Coulomb model was selected to represent the non-linear behavior of ultra-soft soils in this study. When subjected to primary deviatoric loading, the cohesive soil showed a decreasing stiffness and irreversible plastic strains. Limiting states of stress are described by means of the undrained shear strength of the ultra-soft soil. The initial stresses in the case of ultra-soft soil were generated using Jaky's formula which gives the at rest earth pressure coefficient $K_0 = 1 - \sin\phi$ where ϕ is the friction angle in terms of effective stress. Values of soil parameters used in this investigation is summarized in Table 4.24. Finite element modeling was performed on the CIGMAT penetrometer using 3-D axisymmetric analyses. During the generation of the mesh, 545 elements of 15-noded triangular elements with 975 nodes having an average element size of 0.42 mm to provide sufficient accuracy in stress evaluation. The meshes were chosen to match the corresponding prototype geometries in the experimental model test. Also, there is full fixity at the base of the geometry and smooth conditions at the vertical sides. Loading was applied in very small increments up to the total load.

The stages of finite element modeling for the CIGMAT miniature penetrometer can be shown in Figure 4.93, and Figure 4.94 respectively. CIGMAT miniature penetrometer penetration with element and nodes details under the impact of applied force can be shown in Figure 4.93. The generated stressed zone under the CIGMAT miniature penetrometer due to different applied loads can be identified in Figure 4.94.

Table 4.24 Values of soil parameters used in FEM analyses

Parameter	Ultra-soft soil
Unit weight, γ (kN/m ³)	11
Elastic modulus (kN/m ²)	100
Untreated shear strength (kN/m ²)	0.17
Treated shear strength (kN/m ²)	6.8
Poisson's ratio μ	0.45
At rest earth pressure coefficient K_0	1

Shear strength-penetration relationship

Finite element modeling of shear strength versus CIGMAT penetration of untreated and treated ultra-soft soil with lime and polymer is shown in Figure 4.95 a through 4.95 c. The numerical modeling showed good agreement with the experimental data for untreated and treated ultra-soft soils. The R^2 and RMSE for untreated ultra-soft soil were 0.95, 5.9 mm, 0.97 and 6.7 mm for 7 gm and 28 gm CIGMAT penetrometer respectively. The R^2 and RMSE for treated ultra-soft soil with lime were 0.63, 2.1 mm, 0.87 and 1.5 mm for 7 gm and 28 gm CIGMAT penetrometer respectively, while the R^2 and RMSE for treated ultra-soft soil with polymer were 0.96, 1.9 mm, 0.95 and 2.9 mm for 7 gm and 28 gm CIGMAT penetrometer respectively.

Contours of stress distribution

The contours of shear stress distribution of two different ultra-soft soil strength (untreated, $S_u=0.17$ kPa and treated, $S_u= 6.8$ kPa) with the CIGMAT penetrometer is shown in Figure 4.96 a and 4.96 b. For CIGMAT penetrometer weight of 7 gm, the effective stress zone varied from 64% S_u to 32% S_u at depth of 4.2 d and 6 d for untreated ultra-soft soil ($S_u=0.17$ kPa, Figure 4.96 a-1) and from 16% S_u to 8% S_u at depth of 3 d

and 5 d for treated ultra-soft soil ($S_u=6.8$ kPa, Figure 4.96 a-2) respectively. For CIGMAT penetrometer weight of 28 gm, the effective stress zone varied from 90% S_u to 45% S_u at depth of 6.2 d and 8 d for untreated ultra-soft soil ($S_u=0.17$ kPa, Figure 4.96 b-1) and from 28% S_u to 14% S_u at depth of 5 d and 7 d for treated ultra-soft soil ($S_u=6.8$ kPa, Figure 4.96 b-2) respectively. As the CIGMAT penetrometer increased by four times, the maximum effective stress zone and effective depth increased by 41% and 48% for untreated ultra-soft soil ($S_u=0.17$ kPa) and by 75% and 67% for treated ultra-soft soil ($S_u=6.8$ kPa) respectively.

4.8.pH measurement

In this study, pH measurement for both untreated and treated bentonite ultra-soft soil have been studied. pH probe was immersed in the untreated and treated ultra-soft soil and reading was taken after 5 minutes where all the tests have been done at room temperature.

(i) Untreated ultra-soft soil

The relationship between the measured pH and bentonite content of ultra-soft soil is shown in Figure 4.97. Initially, the tap water without bentonite had 7.5 pH value and with the addition of bentonite to the tap water; the pH increased to almost 7.7 with a slight fluctuation with the increase of bentonite content in the mix. It indicates that the bentonite increased the alkaline content slightly in the mix with almost no effect with higher content.

(ii) Lime treatment

The relationship between the measured pH and lime content of treated ultra-soft soil is shown in Figure 4.97. The lime had the capability to increase the pH of the treated ultra-soft soil. As the lime content increased from 0% to 10%, the pH increased by 24%.

(iii) Polymer treatment

The relationship between the measured pH and polymer content of treated ultra-soft soil is shown in Figure 4.97. The lime had the potential to increase the pH of the treated ultra-soft soil. As the polymer content increased from 0% to 10%, the pH increased by 37%.

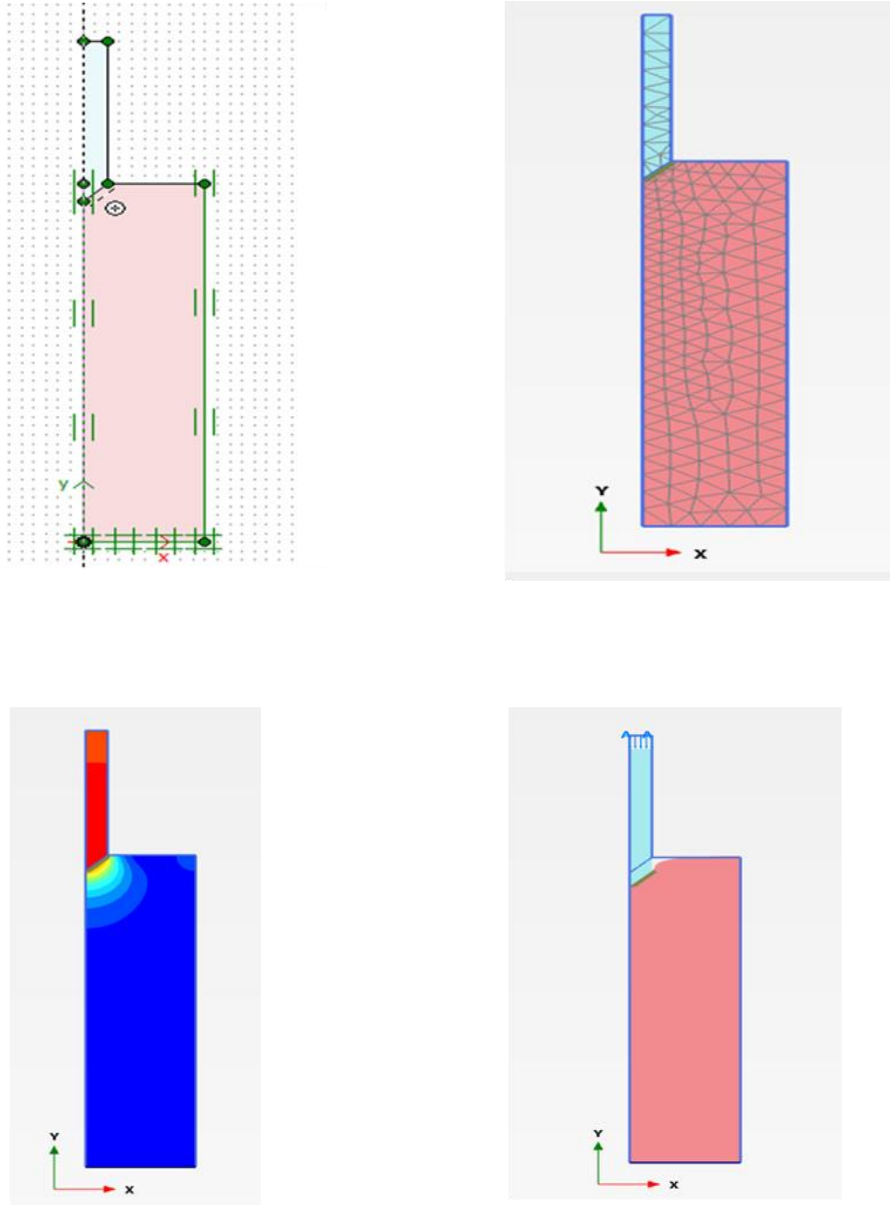


Figure 4.93 Finite element description of CIGMAT penetrometer.

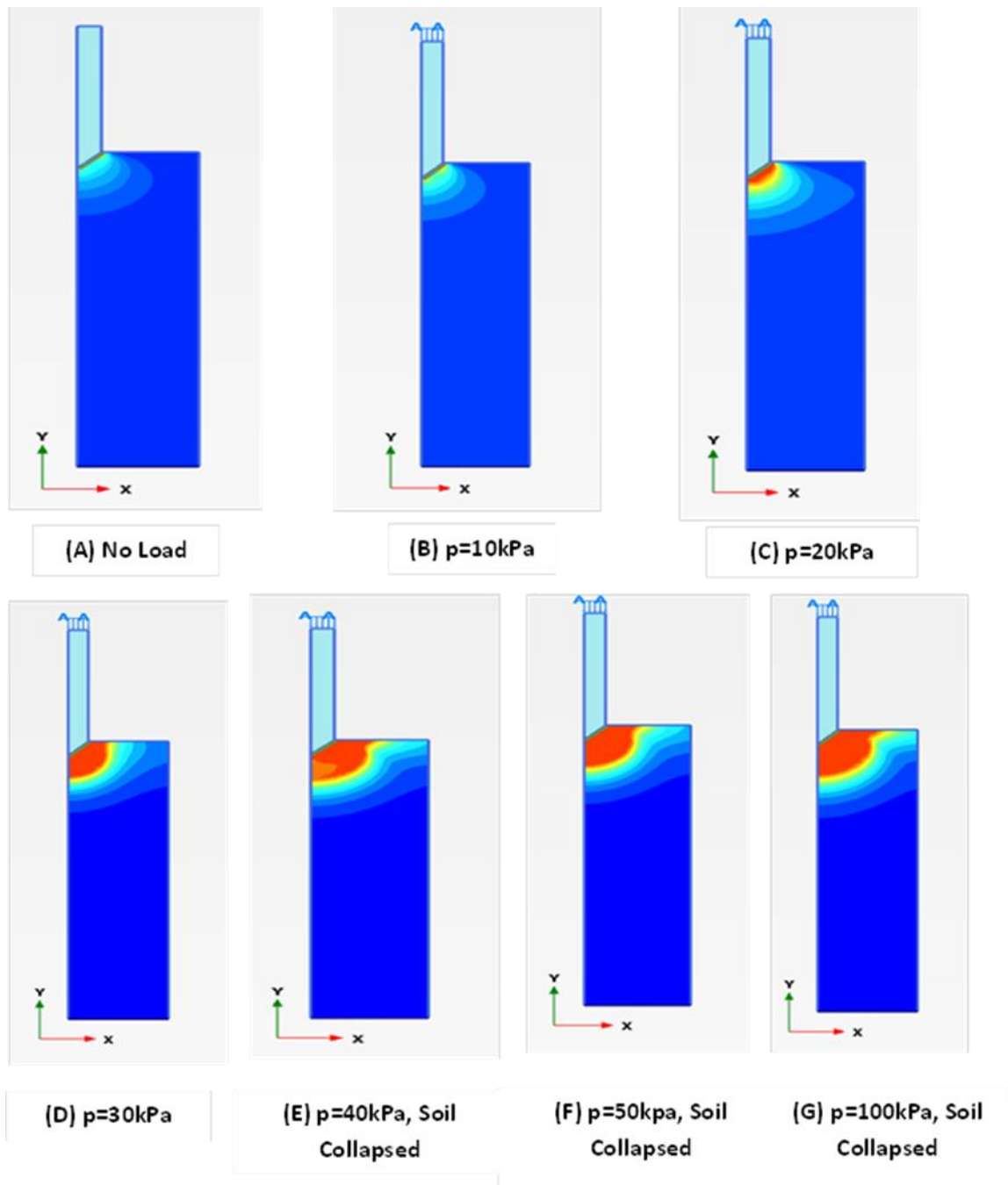


Figure 4.94 Generated stressed zone underneath the CIGMAT miniature penetrometer with different loading conditions.

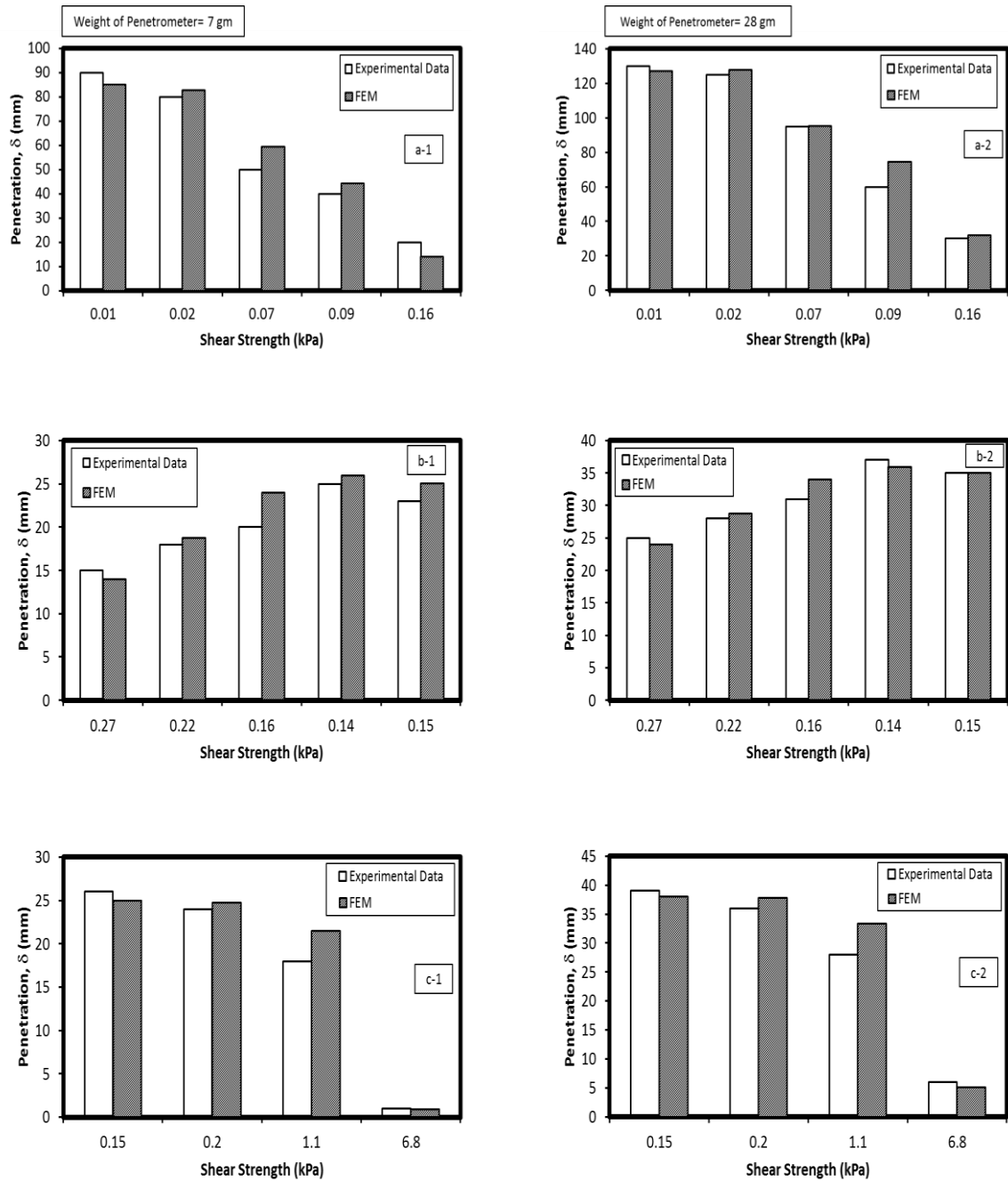


Figure 4.95 Comparing the predicted (FEM) and experimental CIGMAT miniature penetrometer penetration (a) untreated ultra-soft soil (b) 10% bentonite ultra-soft soil treated with lime, and (c) 10% bentonite ultra-soft soil treated with polymer.

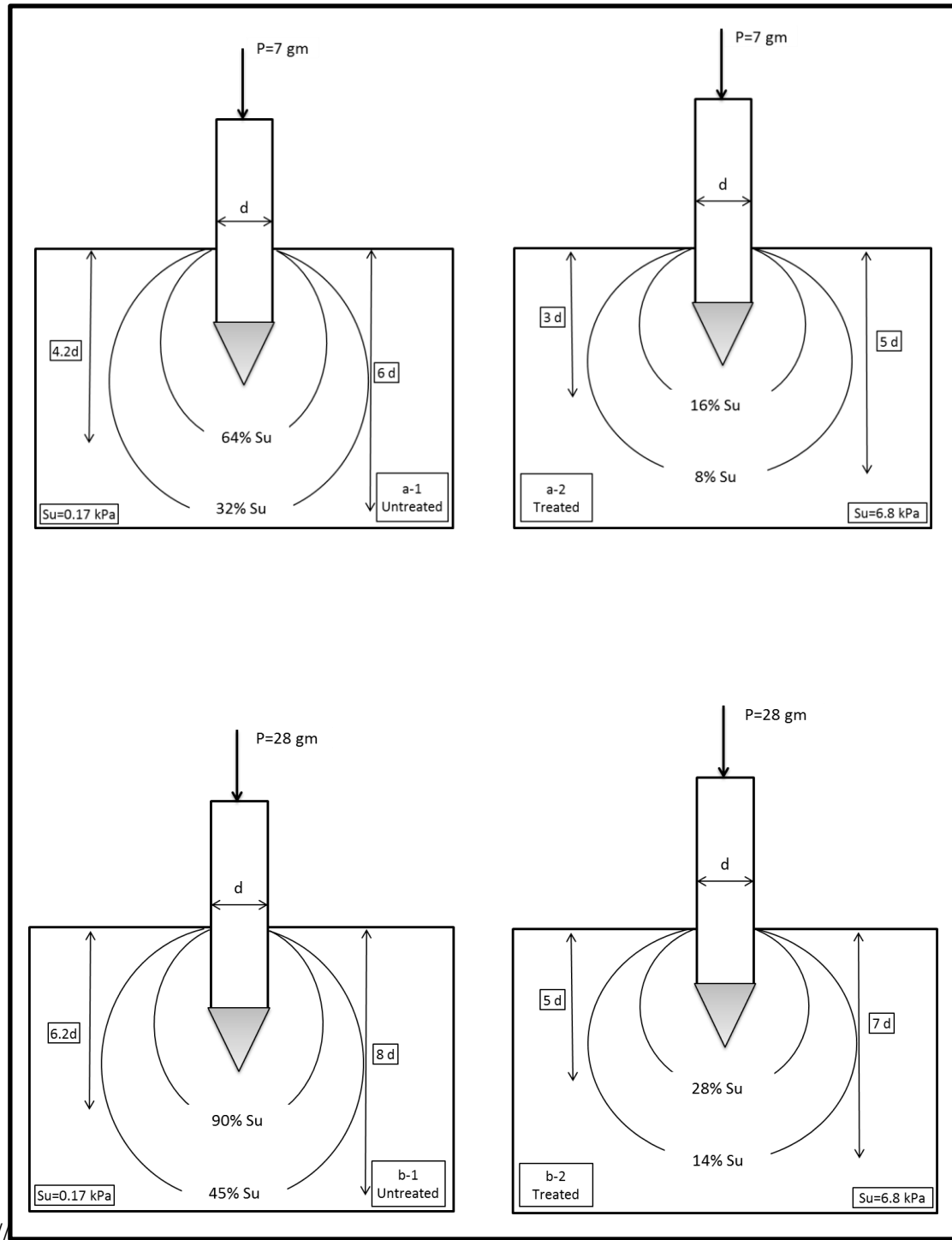


Figure 4.96 Maximum shear stress contours for untreated and treated ultra-soft soils with different weights (P) of CIGMAT penetrometer (a) P = 7 gm, and (b) P = 28 gm.

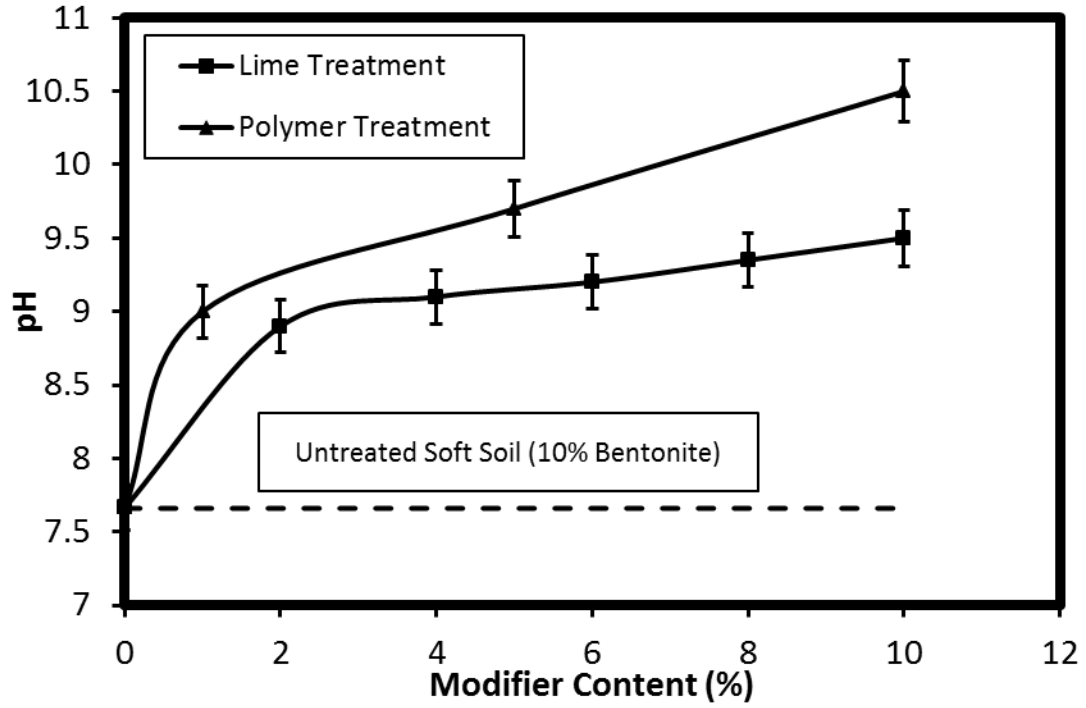


Figure 4.97 Variation of pH with modifier content (lime and polymer) for 10% bentonite ultra-soft soil.

4.9. Impedance spectroscopy characterization

The electrical impedance for untreated and treated ultra-soft clayey soil was measured using LCR-device for different frequencies (from 20 Hz to 300 kHz). Such type of material characterization is clearly identified the material behavior at high frequency where the material can behave as capacitor or resistor (Vipulanandan and Prashanth 2013).

(i) Modeling the impedance spectroscopy behavior

The equivalent electrical circuit of the ultra-soft soil behavior has shown in Figure 4.98 (Vipulanandan and Prashanth, 2013). The bulk capacitance of the material (C_b) has neglected and the total impedance of the equivalent circuit can be evaluated

$$Z = R_b + \frac{2R_c}{1 + \omega^2 R_c^2 C_c^2} - j \frac{2\omega R_c^2 C_c}{1 + \omega^2 R_c^2 C_c^2} \quad (4.11)$$

where Z is the impedance, R_b is the drilling mud (bulk) resistance, R_c is the contact resistance, ω is the angular frequency of the applied alternative current (AC) signal and C_c is the contact capacitance.

The proposed model (4.11) has shown a very good agreement with the experimental data for untreated and treated ultra-soft soil as identified in Figure 4.99, Figure 4.100, and Figure 4.101. A summary of the details of the proposed model (Eq. 4.11) for untreated and treated ultra-soft soil parameters are summarized in Table 4.25.

(ii) Untreated ultra-soft soil

The variation of impedance spectroscopy versus frequency of untreated ultra-soft soil for different bentonite content has been shown in Figure 4.99. For all the ultra-soft soil compositions, as the frequency increased the electrical impedance decreased due to minimizing the effect of the contact resistance. The electrical impedance-frequency relationship has shown that the material is pure resistor at high frequency due to negligible contact impact of the wires (Vipulanandan and Prashanth 2013). Hence, the highest frequency (300 kHz) has been used in this study for measuring the bulk resistance for the ultra-soft soil of different bentonite content. Decreasing the bentonite content has increased the impedance of the ultra-soft soil due to the increase in the conductivity of the medium with higher bentonite content. The drop in the electrical impedance has shown a significant value from 2.67 k.Ωm to 1.71 k.Ωm and 1.6 k.Ωm to 0.607 k.Ωm at 0.02 kHz and 300 kHz respectively.

(iii) Lime treatment

The variation of impedance spectroscopy versus frequency of treated ultra-soft soil with lime for different lime content has been shown in Figure 4.100. Decreasing the lime content has increased the impedance of the treated ultra-soft soil due to the increase in the conductivity of the medium with higher lime content. The drop in the electrical impedance has shown a considerable value from 0.707 k.Ω to 0.414 k.Ω and 0.227 k.Ω to 0.156 k.Ω at 0.02 kHz and 300 kHz respectively.

(iv) Polymer treatment

The variation of impedance spectroscopy versus frequency of treated ultra-soft soil with polymer for different polymer content has been shown in Figure 4.101. Decreasing the polymer content has increased the impedance of the treated ultra-soft soil due to the increase in the conductivity of the medium with higher polymer content. The drop in the electrical impedance has shown a substantial value from 0.773 k.Ω to 0.394 k.Ω and 0.428 k.Ω to 0.176 k.Ω at 0.02 kHz and 300 kHz respectively.

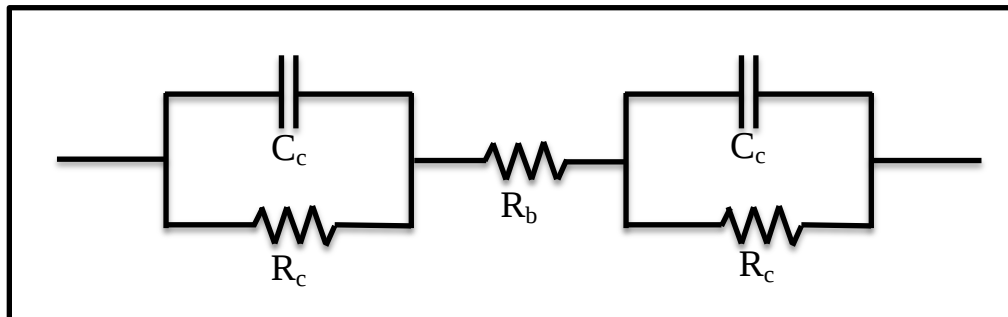


Figure 4.98 The equivalent electrical circuit for the ultra-soft soil condition (Vipulanandan and Prashanth, 2013).

Table 4.25 Model parameters (Eq. 4.11) of untreated and treated ultra-soft soil

Untreated ultra-soft soil	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
2% bentonite	1.6	0.6	5.5	0.98	0.057
4% bentonite	1.35	0.7	6	0.99	0.059
6% bentonite	1	0.8	4	0.99	0.052
8% bentonite	0.8	0.8	8	0.99	0.056
10% bentonite	0.6	0.6	6	0.99	0.075
Treated ultra-soft soil (10% bentonite+lime)	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
2% lime	0.23	0.25	6	0.99	0.014
4% lime	0.2	0.2	8	0.99	0.014
6% lime	0.15	0.20	16	0.99	0.007
8% lime	0.15	0.2	16	0.99	0.022
10% lime	0.16	0.15	20	0.99	0.009
Treated ultra-soft soil (10% bentonite+polymer)	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
1% polymer	0.45	1	18	0.99	0.022
5% polymer	0.30	1	24	0.99	0.016
10% polymer	0.20	0.1	15	0.99	0.015

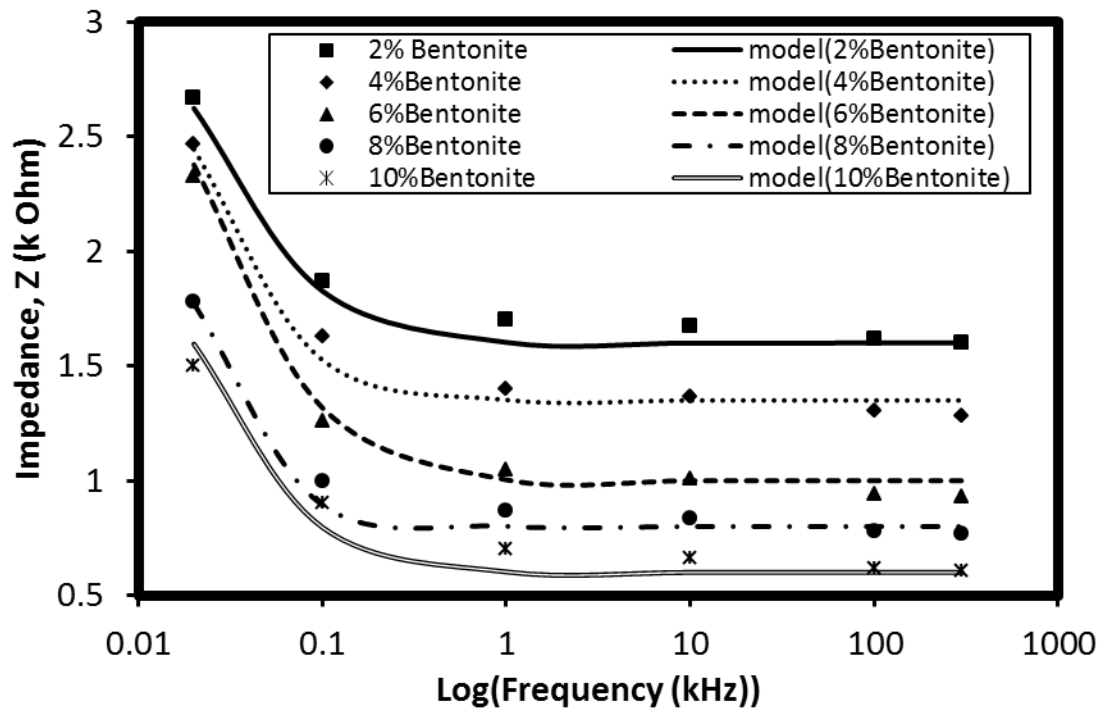


Figure 4.99 Predicted and measured impedance frequency relationship of untreated ultra-soft soil.

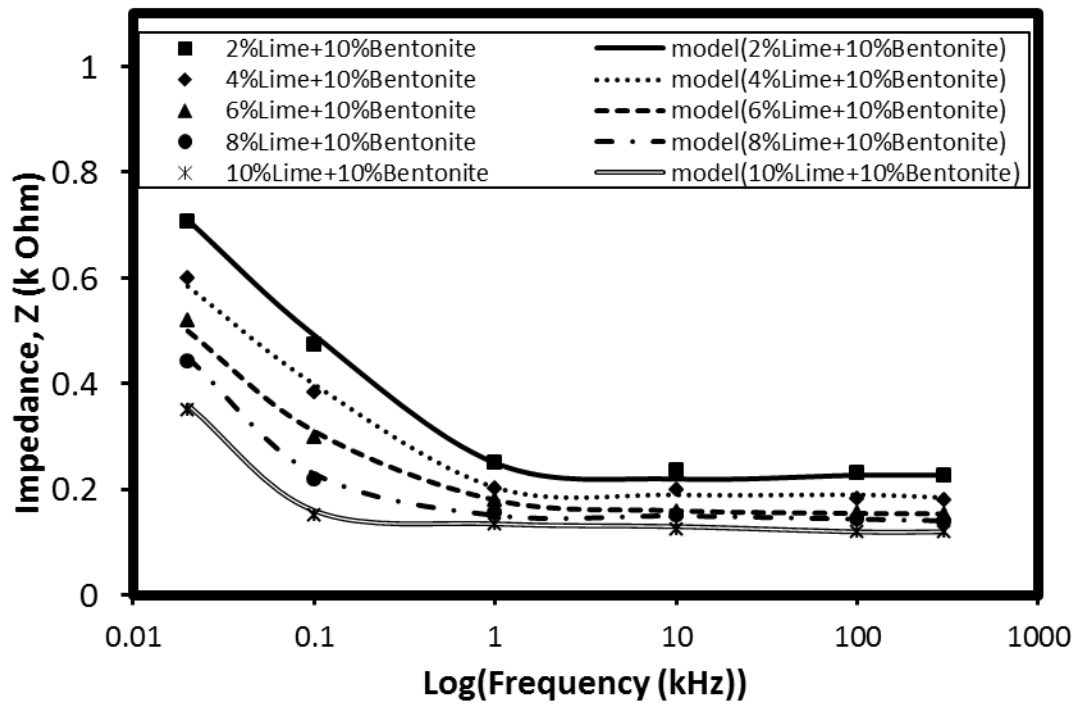


Figure 4.100 Predicted and measured impedance frequency relationship of 10% bentonite ultra-soft soil treated with lime.

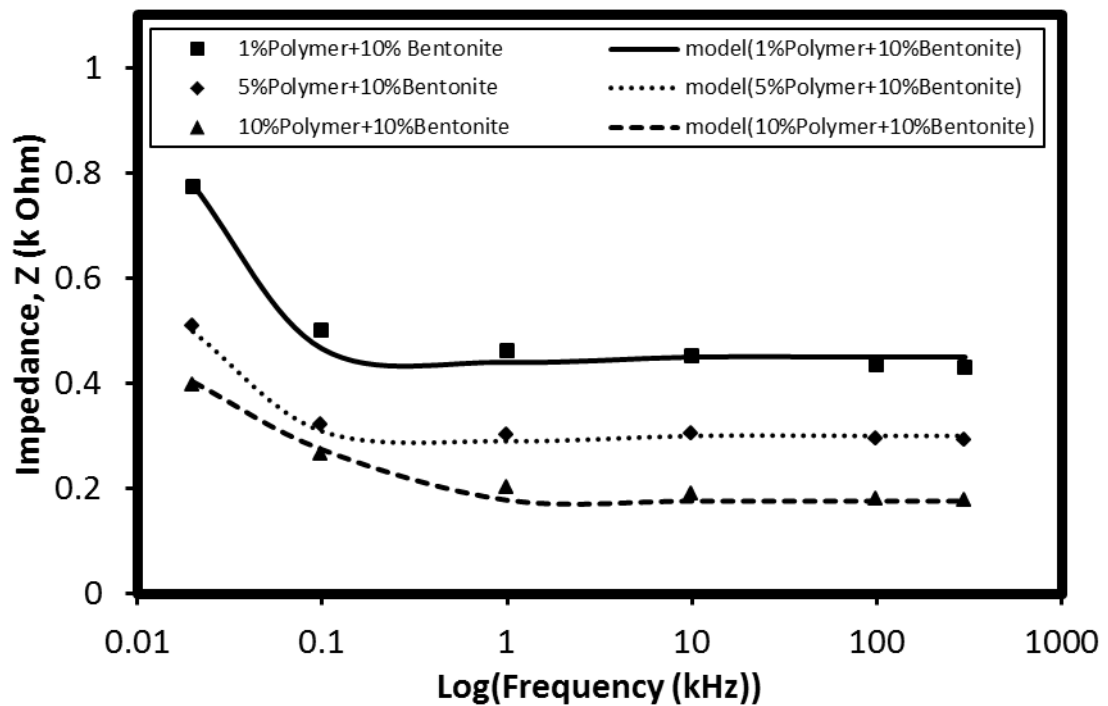


Figure 4.101 Predicted and measured impedance frequency relationship of 10% bentonite ultra-soft soil treated with polymer.

The variation of impedance spectroscopy versus frequency of kaolinite soft soil has been shown in Figure 4.102. Decreasing the kaolinite content has increased the impedance of the ultra-soft soil due to the increase in the conductivity of the medium with higher kaolinite content. The drop in the electrical impedance has shown a significant value from 19.56 k.Ohm to 10.68 k.Ohm and 2.97 k.Ohm to 0.71 k.Ohm at 0.02 kHz and 300 kHz respectively. The proposed model (4.11) has shown a very good agreement with the experimental data for untreated kaolinite ultra-soft soil as identified in Figure 4.102. A summary of the details of the proposed model (Eq. 4.11) for untreated kaolinite ultra-soft soil parameters are summarized in Table 4.26.

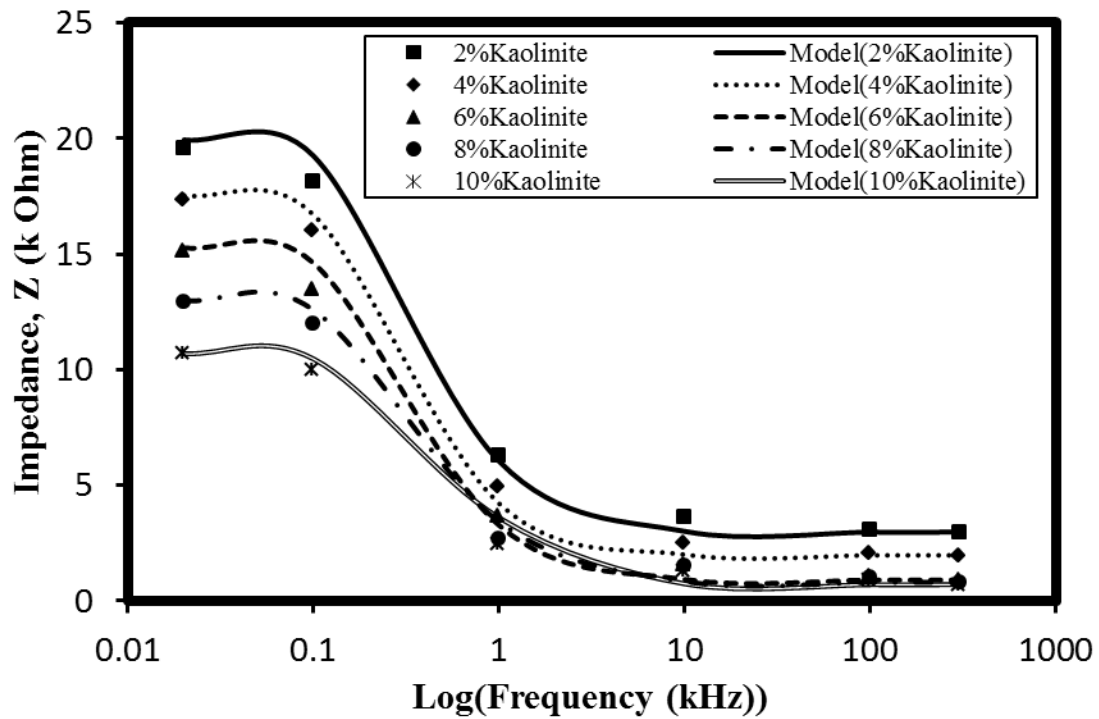


Figure 4.102 Predicted and measured impedance frequency relationship of 10% kaolinite ultra-soft soil.

Table 4.26 Model parameters (Eq. 4.11) of kaolinite ultra-soft soil.

Untreated ultra-soft soil	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
2% kaolinite	2.97	8.5	0.04	0.99	0.55
4% kaolinite	1.97	7.8	0.05	0.99	0.48
6% kaolinite	0.9	7.2	0.05	0.99	0.56
8% kaolinite	0.8	6.1	0.05	0.99	0.48
10% kaolinite	0.7	5	0.05	0.99	0.55

The variation of impedance spectroscopy versus frequency of 10% bentonite soft soil treated with cement has been shown in Figure 4.103. Decreasing the cement content has increased the impedance of the treated ultra-soft soil due to the increase in the conductivity of the medium with higher cement content. The drop in the electrical impedance has shown a significant value from 1.71 k.Ohm to 1.077 k.Ohm and 0.607 k.Ohm to 0.38 k.Ohm at 0.02 kHz and 300 kHz respectively. The proposed model (4.11) has shown a very good agreement with the experimental data for 10% bentonite treated ultra-soft soil with cement as identified in Figure 4.103. A summary of the details of the proposed model (Eq. 4.11) for 10% bentonite treated ultra-soft soil parameters are summarized in Table 4.27.

Table 4.27 Model parameters (Eq. 4.11) of treated ultra-soft soil with cement

Treated ultra-soft soil	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
10% bentonite	0.61	0.58	2.3	0.99	0.05
10% bentonite+ 1% cement	0.54	0.51	2.3	0.99	0.057
10% bentonite+ 5% cement	0.5	0.46	2.3	0.99	0.062
10% bentonite+ 10% cement	0.38	0.35	1.8	0.99	0.056

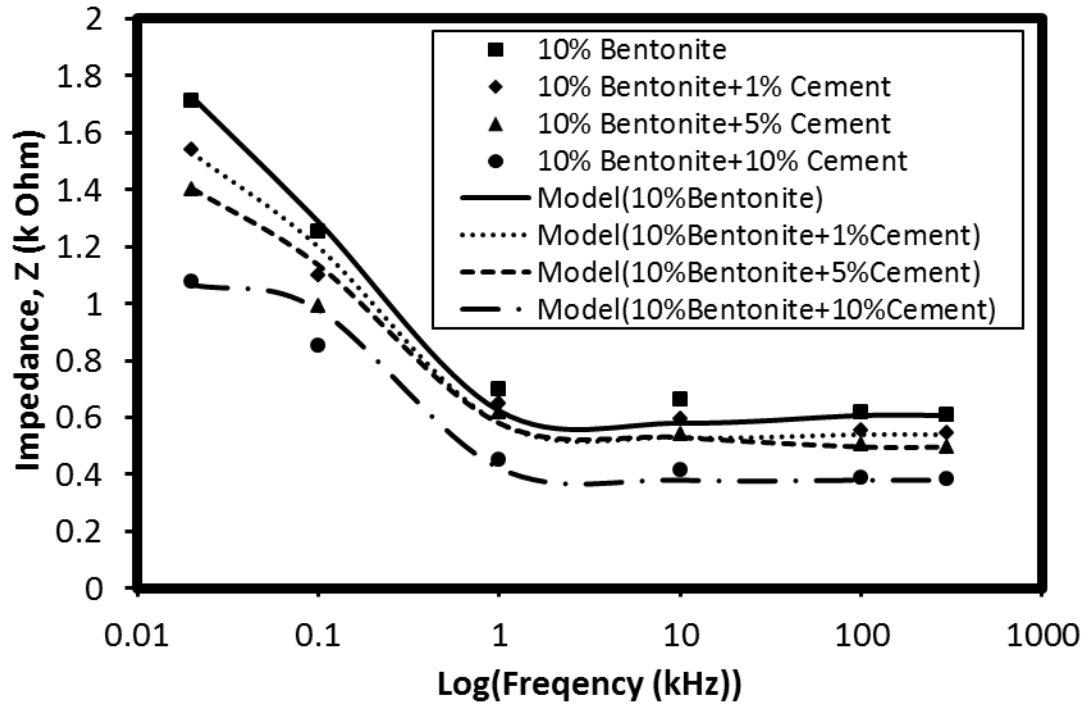


Figure 4.103 Predicted and measured impedance frequency relationship of 10% bentonite ultra-soft soil treated with cement.

The variation of impedance spectroscopy versus frequency of 10% bentonite soft soil treated with sand has been shown in Figure 4.104. Increasing the sand content has increased the impedance of the treated ultra-soft soil due to the decrease in the conductivity of the medium with higher sand content. As the sand content increased from 0% to 1% in the 10% bentonite treated soft soil, the electrical impedance decreased from 1.71 k.Ohm to 0.829 k.Ohm and 0.421 k.Ohm to 0.38 k.Ohm at 0.02 kHz and 300 kHz respectively. The proposed model (4.11) has shown a very good agreement with the experimental data for 10% bentonite treated ultra-soft soil with sand as identified in Figure 4.104. A summary of the details of the proposed model (Eq. 4.11) for 10% bentonite treated ultra-soft soil parameters are summarized in Table 4.28.

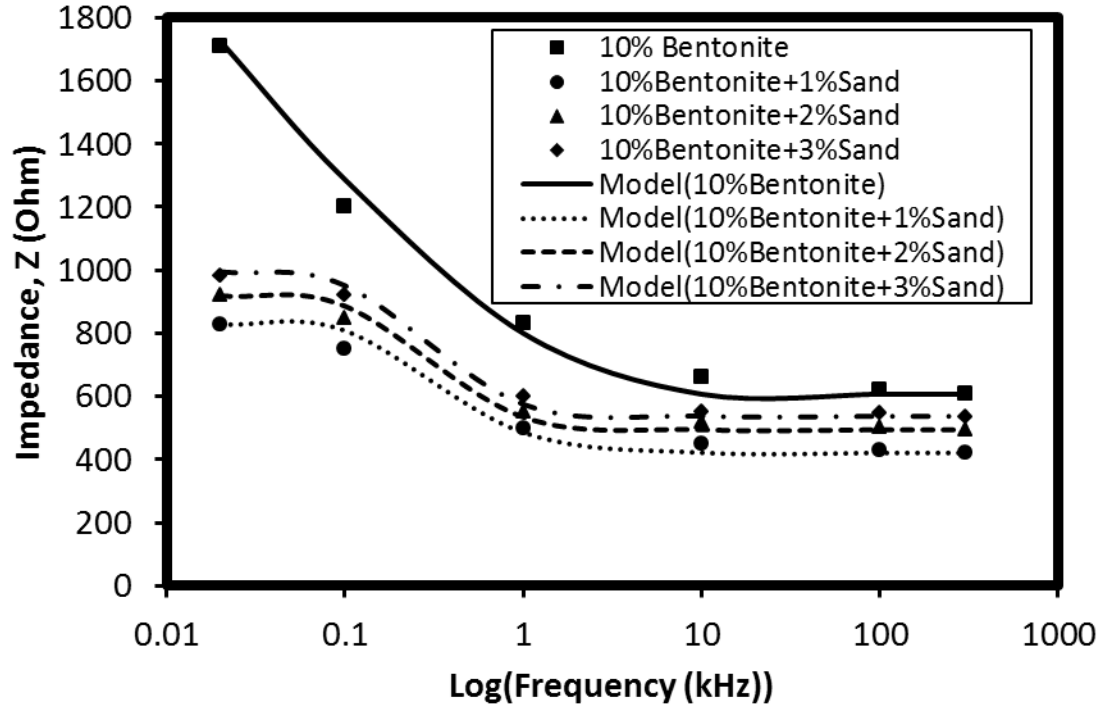


Figure 4.104 Predicted and measured impedance frequency relationship of 10% bentonite ultra-soft soil treated with sand.

Table 4.28 Model parameters (Eq. 4.11) of treated ultra-soft soil with sand

Treated ultra-soft soil	R_b (k. Ohm)	R_c (k. Ohm)	C_c (pF)	R^2	RMSE (k. Ohm)
10% bentonite	0.61	0.58	2.3	0.99	0.095
10% bentonite+ 1% Sand	0.54	0.23	2.3	0.99	0.077
10% bentonite+ 2% Sand	0.49	0.215	2.3	0.99	0.074
10% bentonite+ 3% Sand	0.42	0.204	1.8	0.99	0.070

The variation of impedance spectroscopy versus frequency of 10% bentonite soft soil treated with kaolinite has been shown in Figure 4.105. Increasing the kaolinite content has decreased the impedance of the treated ultra-soft soil due to the increase in the conductivity of the medium with higher kaolinite content. As the kaolinite content increased from 2% to 10% in the 10% bentonite treated soft soil, the electrical impedance decreased from 1.1 k. Ohm to 0.7 k. Ohm and 0.675 k. Ohm to 0.45 k. Ohm at 0.02 kHz and

300 kHz respectively. The proposed model (4.11) has shown a very good agreement with the experimental data for 10% bentonite treated ultra-soft soil with kaolinite as identified in Figure 4.105. A summary of the details of the proposed model (Eq. 4.11) for 10% bentonite treated ultra-soft soil parameters are summarized in Table 4.29.

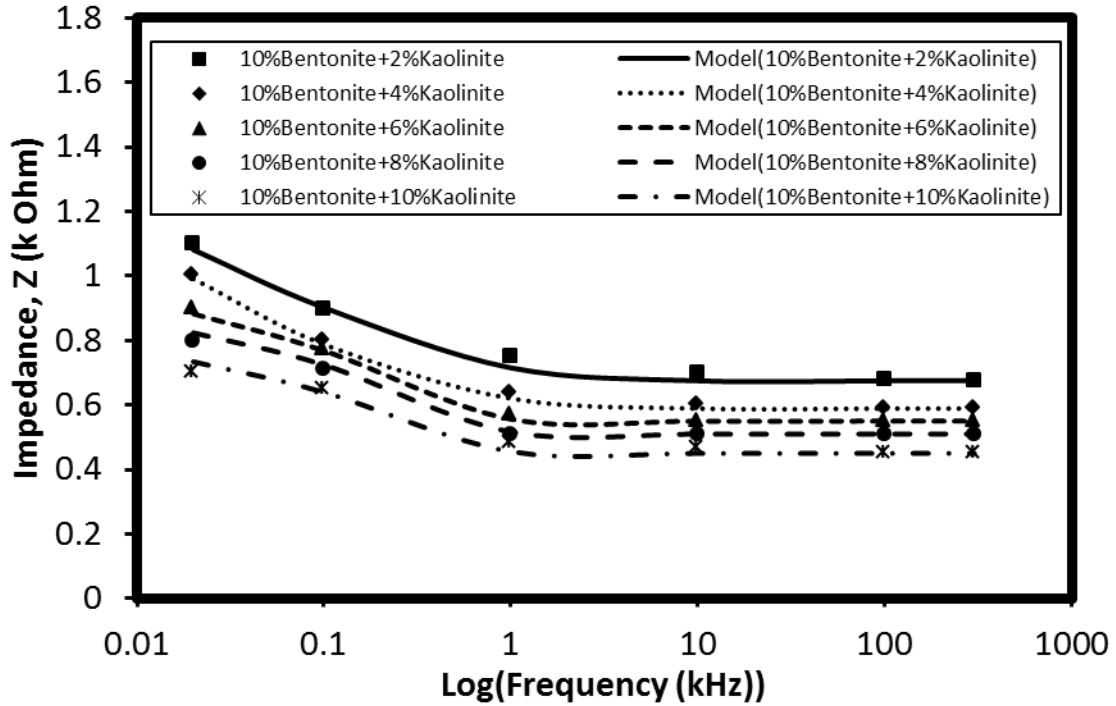


Figure 4.105 Predicted and measured impedance frequency relationship of 10% bentonite ultra-soft soil treated with kaolinite.

Table 4.29 Model parameters (Eq. 4.11) of treated ultra-soft soil with kaolinite

Treated ultra-soft soil	R_b (k.Ohm)	R_c (k.Ohm)	C_c (pF)	R^2	RMSE (k.Ohm)
10% bentonite+ 2% kaolinite	0.68	0.21	7	0.98	0.019
10% bentonite+ 4% kaolinite	0.59	0.21	8	0.99	0.01
10% bentonite+ 6% kaolinite	0.55	0.17	7	0.99	0.009
10% bentonite+ 8% kaolinite	0.51	0.16	7	0.99	0.012
10% bentonite+ 10% kaolinite	0.45	0.145	8	0.94	0.072

4.10. Stress-strain analysis of soft soil

4.10.1. Experimental tests

The modified vane shear device at a very slow rate of 0.1 mm/min was used to test 2% to 10% bentonite soft soil. The experimental test has been done at room temperature to quantify the stress-strain relationship for the soft soil as shown in Figure 4.106. It was indicated that as the bentonite content increased from 2% to 10%, the peak stress and ultimate strain increased by 800% and 150% respectively.

4.10.2. Hyperbolic model

The following mathematical model was used to predict the stress-strain relationship for both 2% and 10% bentonite ultra-soft soil

$$\text{Shear Stress} = \frac{\text{Strain (\%)}}{A_3 + B_3 * \text{Strain (\%)}} \quad (4.12)$$

where A_3 (1/ kPa) and B_3 (1/ kPa) are model parameters. Table 4.30 summarized the hyperbolic model parameters (Eq. 4.12) for both 2% and 10% bentonite soft soil. The hyperbolic model prediction with experimental results for both 2% and 10% bentonite soft soil is shown in Figure 4.106. The hyperbolic model predication showed a very good agreement with the experimental results. The R^2 and RMSE were 0.98, 0.0012 (kPa), 0.99, and 0.0048 (kPa) for both 2% and 10% bentonite soft soil respectively.

Table 4.30 Hyperbolic model parameters for Stress-Strain Relationship

Composition	Model Parameters		Accuracy Predication	
	A_3 (1/ kPa)	B_3 (1/ kPa)	R^2	RMSE (kPa)
2% Bentonite	1000	25	0.98	0.0012
10% Bentonite	180	4	0.99	0.0048

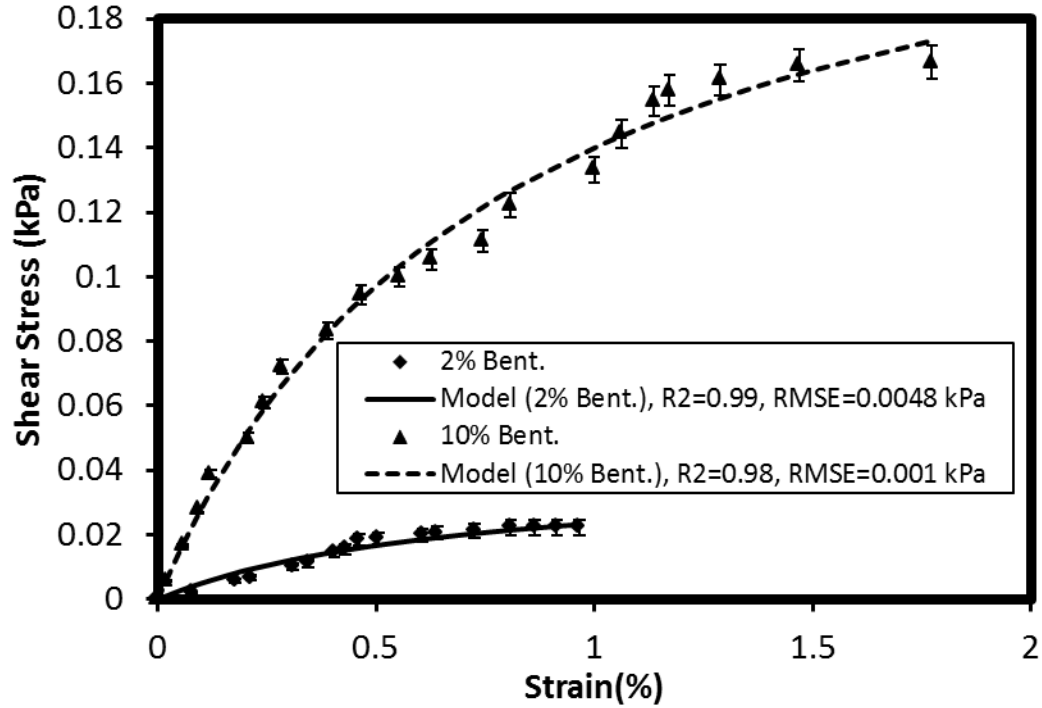


Figure 4.106 The stress-strain relationship of experimental and model prediction of 2% and 10% soft soil.

4.10.3. Statistical analysis of Eq. 4.12

To better assess the evaluation of the prediction of the proposed model (Eq. 4.12), several statistical models such as regression analysis, error estimation analysis, classical statistical analysis, cumulative probability function and overall evaluation of these models is investigated. The statistical models are used to estimate the accuracy prediction for Eq. 4.12 to predict the stress-strain relationship of 2% bentonite to 10% bentonite soft soil (Table 4.31). Based on the R^2 estimation, the model prediction for 10% bentonite content was the highest, while the model prediction for 6% bentonite content was the lowest. Based on the RMSE estimation, the model prediction for 2% bentonite content was the highest, while the model prediction for 8% bentonite content was the lowest. Based on the mean evaluation, the model prediction for 8% bentonite content was the highest, while the model prediction of 4% bentonite content was the lowest. Based on the

cumulative probability function, the model prediction for 2% bentonite content was the highest, while the model prediction for 8% bentonite content was the lowest. Based on the overall evaluation, the model prediction for 2% bentonite content was the highest, while the model prediction for 6% bentonite content was the lowest.

Table 4.31 Statistical assessment for Eq. 4.12

Comp.		Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function			Overall Evaluation	
Bent.(%)	No. of Data	R ²	r ₁ Rank	RMSE (kPa)	r ₂ Rank	Mean of T*	r ₃ Rank	T at P ₅₀	P ₉₀ -P ₅₀ **	r ₄ Rank	r _g gross Rank	E
10	25	0.993	1	0.00482	3	0.9514	3	0.9954	0.0735	2	9	2
8	25	0.955	4	0.01035	5	0.9844	1	0.9597	0.3342	5	15	4
6	25	0.944	5	0.00554	4	1.06	4	1.0055	0.934	3	16	5
4	25	0.956	3	0.00268	2	0.8686	5	1.0112	0.111	4	14	3
2	25	0.976	2	0.00117	1	1.0402	2	0.9956	0.6323	1	6	1

*T is the ratio between the predicted to measured shear strength, and

** P₅₀, P₉₀ are the probability function estimation at 50% and 90% respectively.

4.10.4. Finite element modeling (FEM)

An elasto-plastic Mohr-Coulomb model was chosen to illustrate the non-linear soft soil behavior in this study. When the soft soil is subjected to primary deviatoric loading, cohesive soft soil stiffness decreased and irreversible plastic strains developed. The undrained shear strength of the soft soil is used as limiting states of stresses. The soft soil parameters used in this investigation were similar to Table 4.24.

The analysis was performed using the 2-D and 3-D models for vane shear simulation in soft soil. During the generation of the mesh, 674 elements of 15-noded triangular elements with 1760 nodes having an average element size of 0.45 mm and 1150 elements of 10-noded tetrahedral elements with 2050 nodes having an average element size of 0.54 mm were selected in order to provide a great accuracy in the determination of stresses.

The meshes were chosen to match the corresponding geometries for the modified vane shear device as the experimental model test was performed and were 5 cm wide with depths of 10 cm. The geometry is divided into elements of the basic element type, and compatible plate elements for vane blades. The deformed mesh and displacement distribution for 2D FEM analysis of vane shear simulation are shown in Figure 4.107 and Figure 4.108 respectively. While in Figure 4.109 the deformed mesh of 3D analysis can be shown. It is clearly shown that for both 2D and 3D analyses, the displacement density distribution is high in the places of the tip of the blades in the soft soil. In the laboratory and FEM simulation analyses, the maximum soil deformation started at the tip of the blades and continued up to the failure where maximum soft soil displacement can be clearly identified in such places.

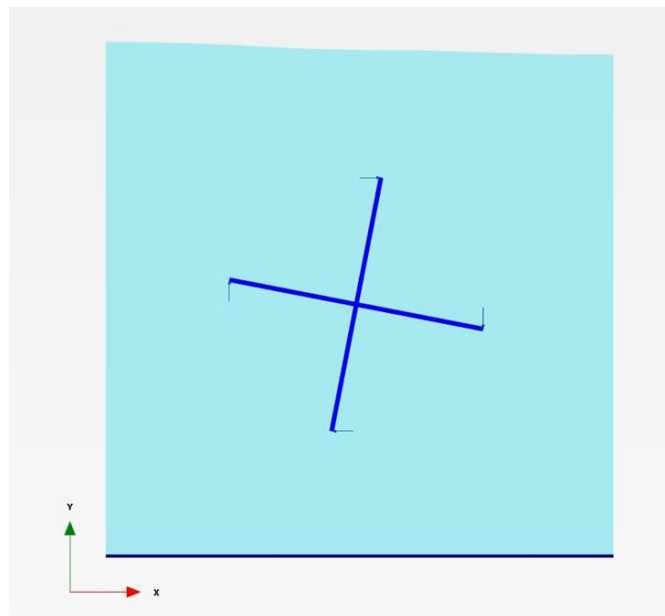


Figure 4.107 Deformed Shape of 2D FEM Analysis.

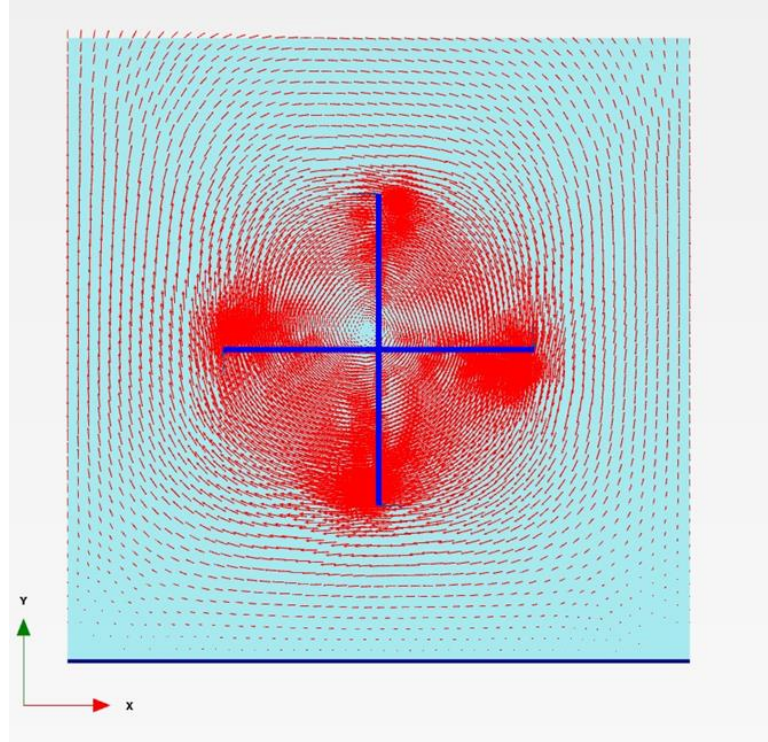


Figure 4.108 Displacement Distribution of 2D FEM Analysis.

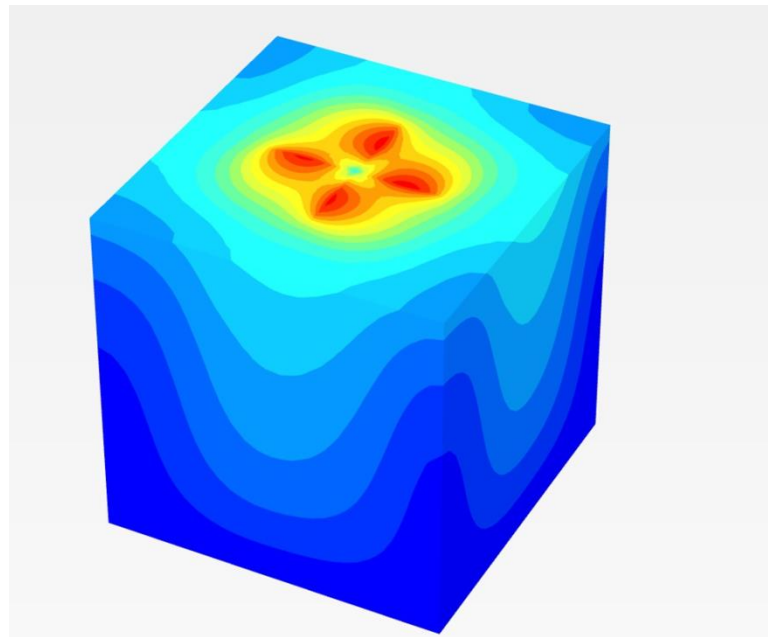


Figure 4.109 Deformed Shape of 3D FEM Analysis.

The FE simulation of the stress-strain relationship for both 2D and 3D analysis of vane shear laboratory experimental tests can be shown Figure 4.110. Both 2D and 3D

analyses predict the experimental data with a very good degree of accuracy with better prediction for 3D analysis. As the FE simulation varied from 2D to 3D analysis, the R^2 increased from 0.93 to 0.99 and RMSE decreased from 0.016 kPa to 0.005 kPa respectively.

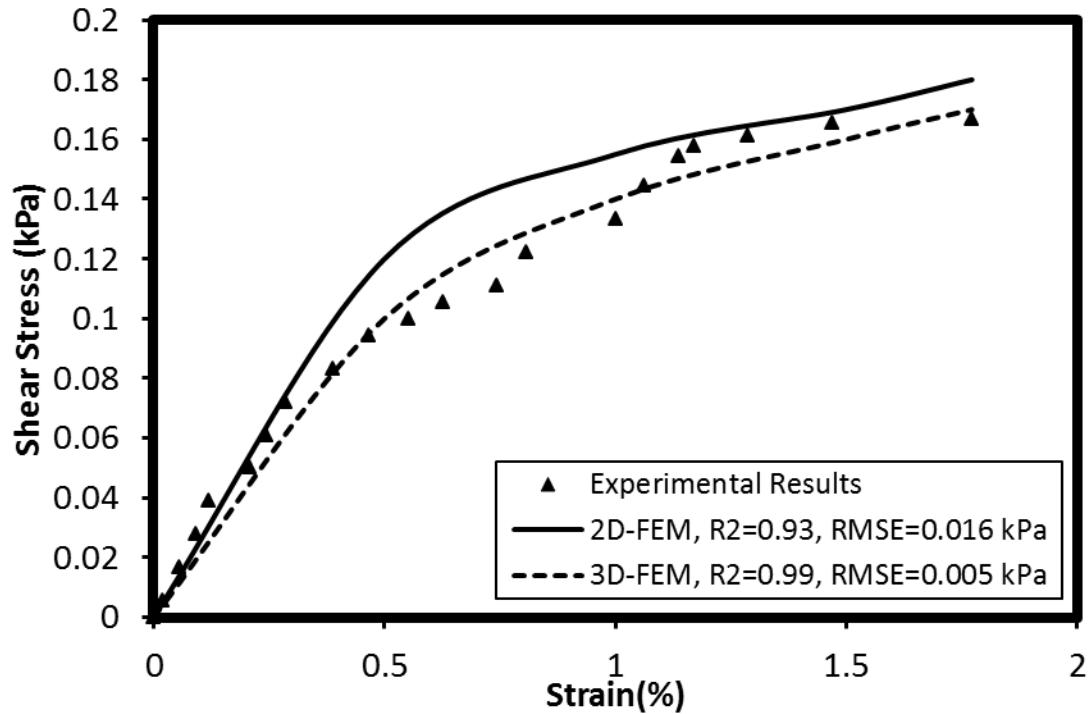


Figure 4.110 2D and 3D FEM Modeling of Soft Soil.

4.11. Non-destructive soft soil characterization

The nondestructive assessment is the best way to characterize the soft soil and cake properties for formation damage application. Hence, experimental and analytical modeling were performed to correlate both pulse velocity and electrical resistivity of soft soil using 2% to 10% bentonite drilling mud. The main objective of such kind of studies is to back calculate the soft cake content using nondestructive tests through both pulse velocity and electrical resistivity methods. The electrical resistivity variation with bentonite content up to 10% is shown in Figure 4.111. The electrical resistivity is

decreased as the bentonite content is increased due to the increase in the medium conductivity with the higher bentonite content. As the bentonite content in the soft soil increased from 2% to 10%, the electrical resistivity decreased by 60%. As it was mentioned in the earlier sections in this chapter, the hyperbolic model is used to predict the electrical resistivity variation with bentonite content with R^2 up to 0.99. The variation of compression wave velocity with bentonite content is shown in Figure 4.112. As the bentonite content in the soft soil increased from 2% to 10%, the compression wave velocity decreased by 15%. A linear model is proposed to predict the velocity versus bentonite content relationship as

$$V_c = C_3 + D_3 * \text{Bent.}(\%), \quad (4.13)$$

where C_3 (m/s) and D_3 (m/s) are model parameters; and V_c is the compression wave velocity. The proposed model (Eq. 4.13) predicted the experimental results with very good agreement with R^2 up to 0.99.

From the two independent measurements (Figure 4.111 and Figure 4.112), a useful correlation between compression wave velocity and electrical resistivity of soft soil can be concluded as

$$V_c = E_3 + F_3 * \rho, \quad (4.14)$$

where E_3 (m/s) and F_3 (m/(s.Ohm.m)) are model parameters.

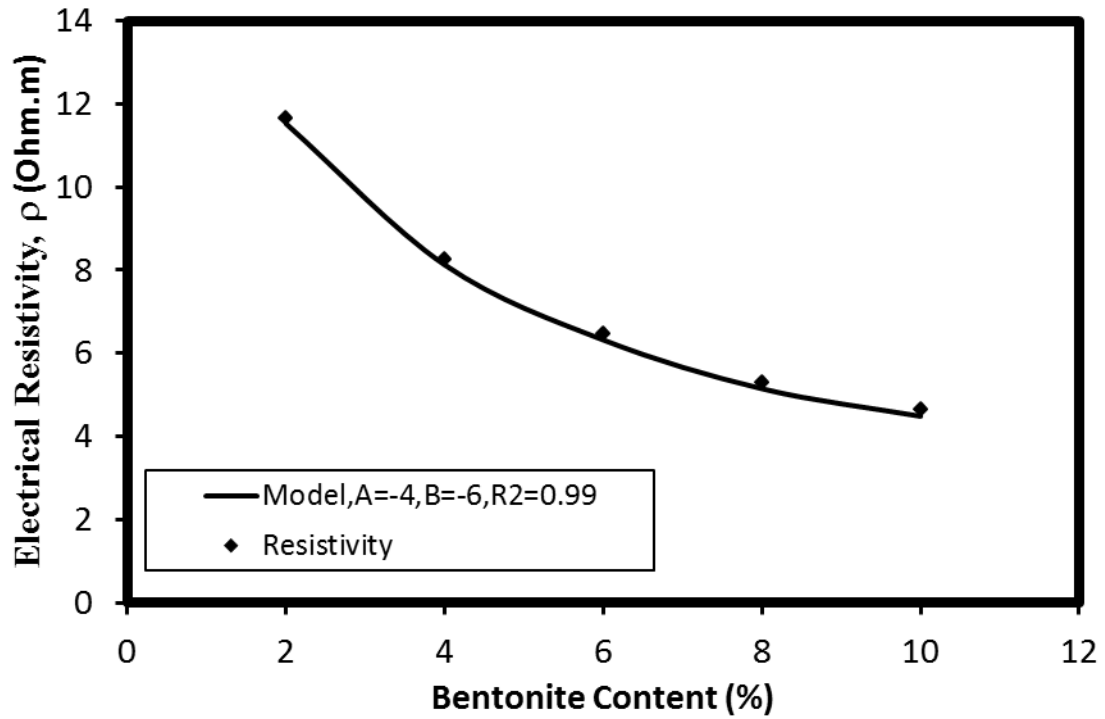


Figure 4.111 Electrical resistivity variation with different bentonite content for soft soil.

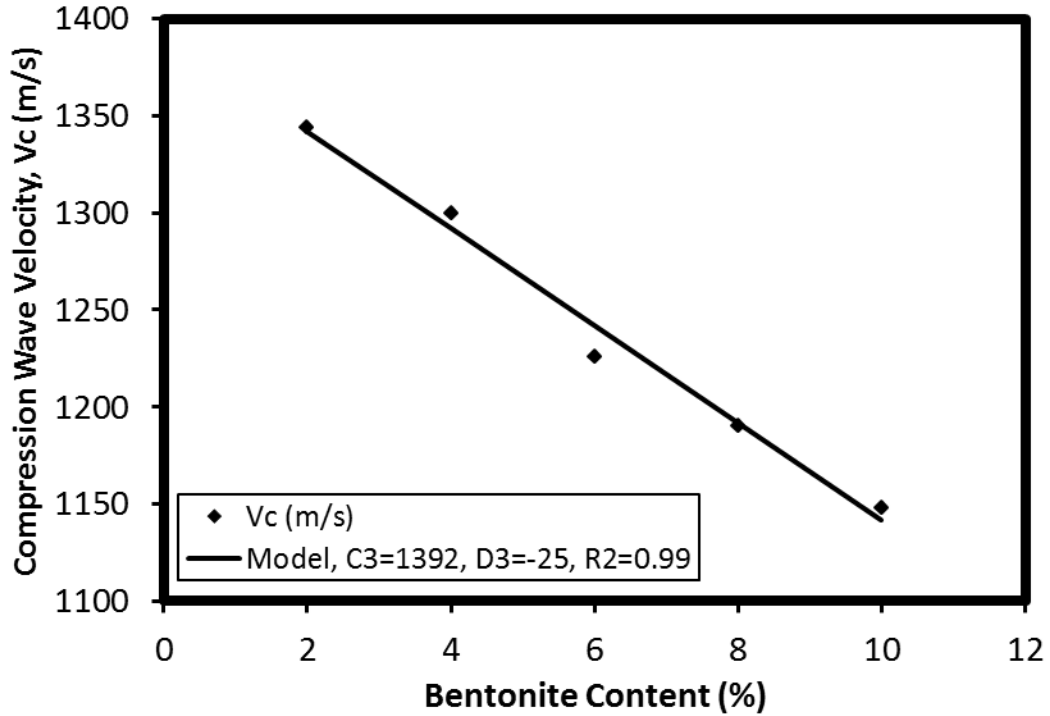


Figure 4.112 Compression wave velocity variation with bentonite content for bentonite soft soil.

Eq. (4.14) is a powerful model since it is correlating two non-destructive measurements (compression pulse velocity and electrical resistivity) together, and then any of them can be used in the field measurement based on the available information. Eq. (4.14) prediction with experimental data can be shown in Figure 4.113. A good agreement between the proposed model (Eq. 4.14) and experimental data can be noticed with R^2 up to 0.92. As the electrical resistivity of the soft soil increased from 4.6 Ohm.m to 11.7 Ohm.m, the compression pulse velocity increased from 1148 m/s to 1344 m/s.

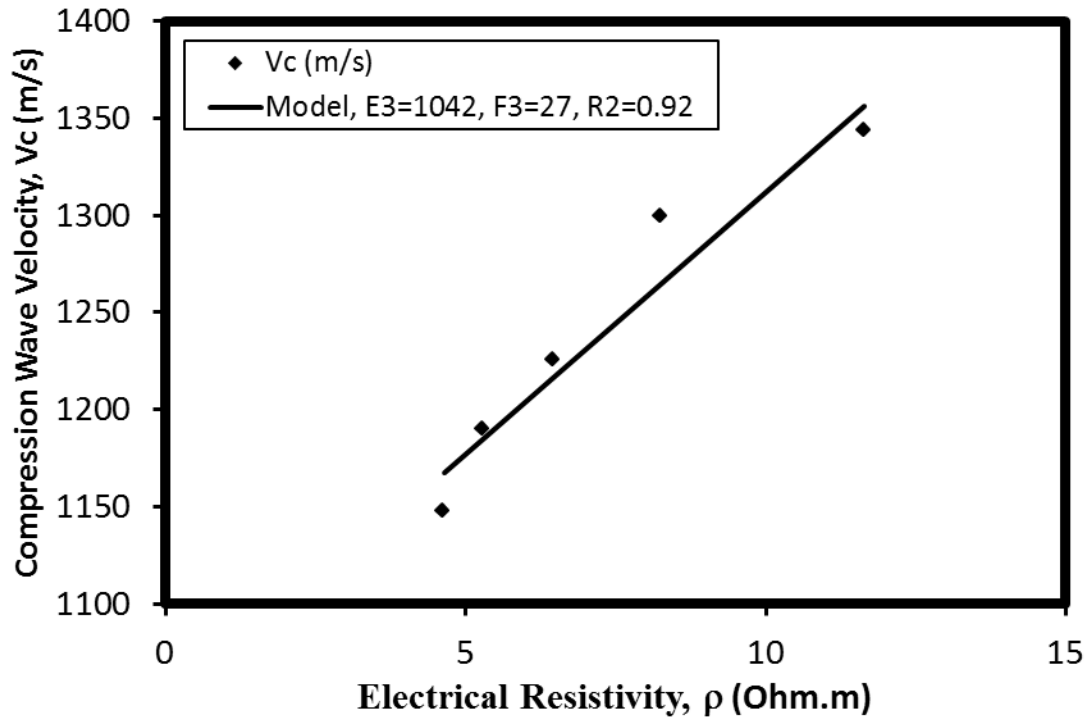


Figure 4.113 Pulse Velocity vs. Electrical Resistivity Relationship.

4.12. Shear strength versus moisture content relationship

Different correlations to predict the undrained shear strength (S_u) of soft soil have been reported in the literature. The undrained shear strength of soil varied from (0.3 to 25) kPa. The shear strength has been correlated to soil properties such as plastic limit (PL), liquid limit (LL), and water content (W/C) (ratio of weight of water to weight of

solid). Based on literature review, over 100 data were collected from different sources for the analyses. New strength relationships were attempted for the very soft soil in terms of moisture content and liquid limit. Therefore, it was very important to re-evaluate some of the correlation equations in the literature and check their effectiveness for predicting the shear strength of soft soil. In addition, new correlations for shear strength in soft soil were introduced combining test results of laboratory miniature vane shear test with high moisture contents and data from the literature. Two relationships are proposed based on the water content and liquid limit of the soft soil:

Model 1: Total of 92 data collected from the literature was used to develop this strength relationship. The strength of the soil varies from (1 to 10) kPa.

$$S_u = -6.0 * \ln(W/C \%) + 15, \quad \text{when "W/C < 300\% \& LL < 500\%."} \quad (4.15)$$

Model 2: Soft soil with varying percentage of bentonite content was used in this study. The clay content varied from (2 to 10) %.

$$S_u = 14.369 * e^{(-0.004 * \frac{W}{C} \%)} + 1/(e^{(\frac{W}{C} \% - LL \%)}), \text{ when "W/C > 300\% \& LL > 500\%,"} \quad (4.16)$$

where S_u is the undrained shear strength of the ultra-soft soil, W/C is the moisture content, and LL is the liquid limit. The variation of the soft soil undrained shear strength with the water content for both $0 \text{ kPa} < S_u < 10 \text{ kPa}$ and $0 \text{ kPa} < S_u < 1 \text{ kPa}$ are shown in Figure 4.114 and Figure 4.115 respectively. The second proposed model (Eq. 4.16) was in a very good agreement with the experimental data having R^2 up to 0.91.

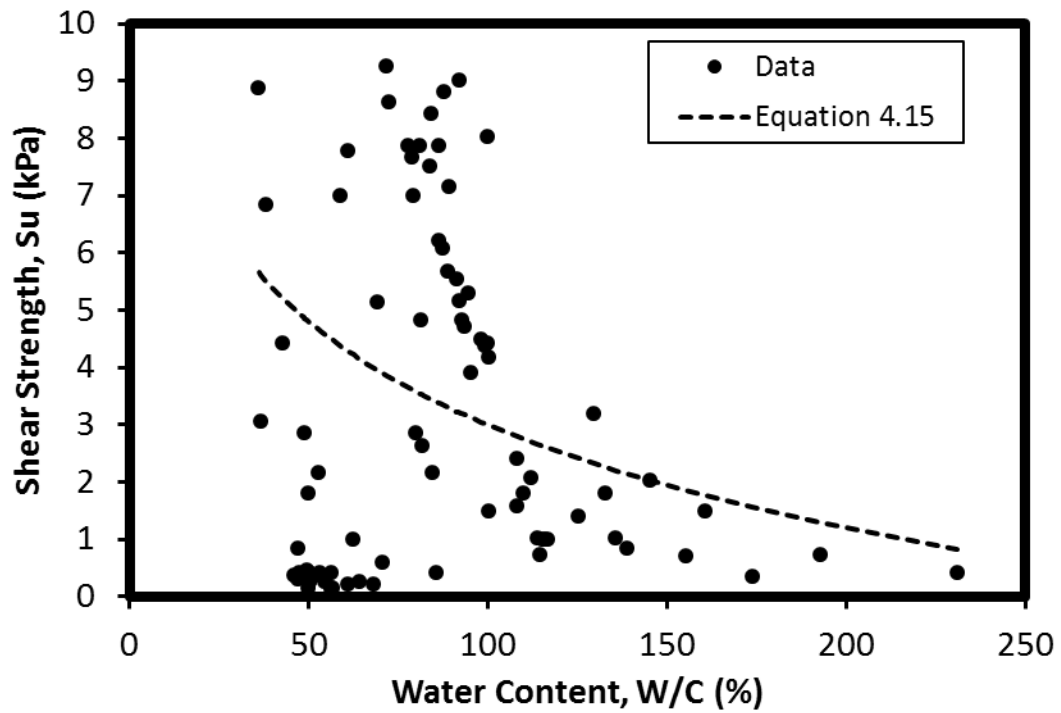


Figure 4.114 Variation of shear strength with water content of soils with ($0 \text{ kPa} < S_u < 10 \text{ kPa}$).

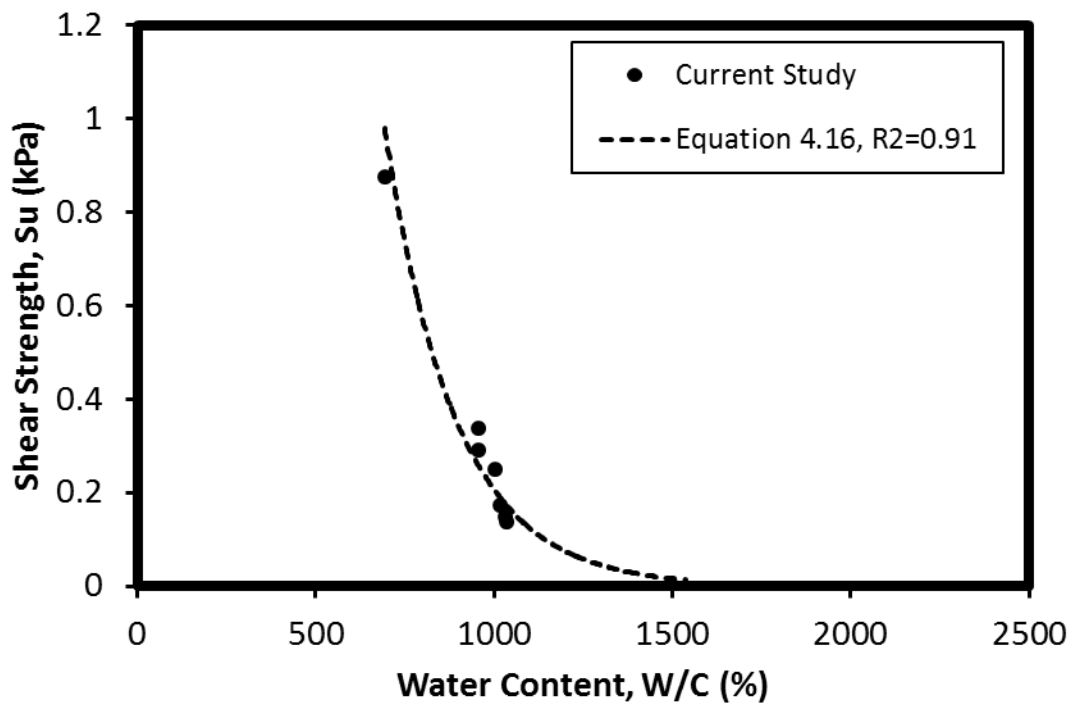


Figure 4.115 Variation of shear strength with water content of soils with ($0 \text{ kPa} < S_u < 1 \text{ kPa}$).

The proposed model 2 (Eq. 4.16) with several other relationships from literature as summarized in Table 2.16 were used to predict the relationship between shear strength and water content for laboratory and reported data for ultra-soft soil with high moisture content as shown in Figure 4.116. It is clearly shown that the previously reported relationships failed to predict the correlation between the shear strength and water content for ultra-soft soil while the provided relationship (Eq. 4.16) predicted the correlation very well supported by laboratory and previous reported data from literature and the coefficient of correlation (R^2) was 0.91.

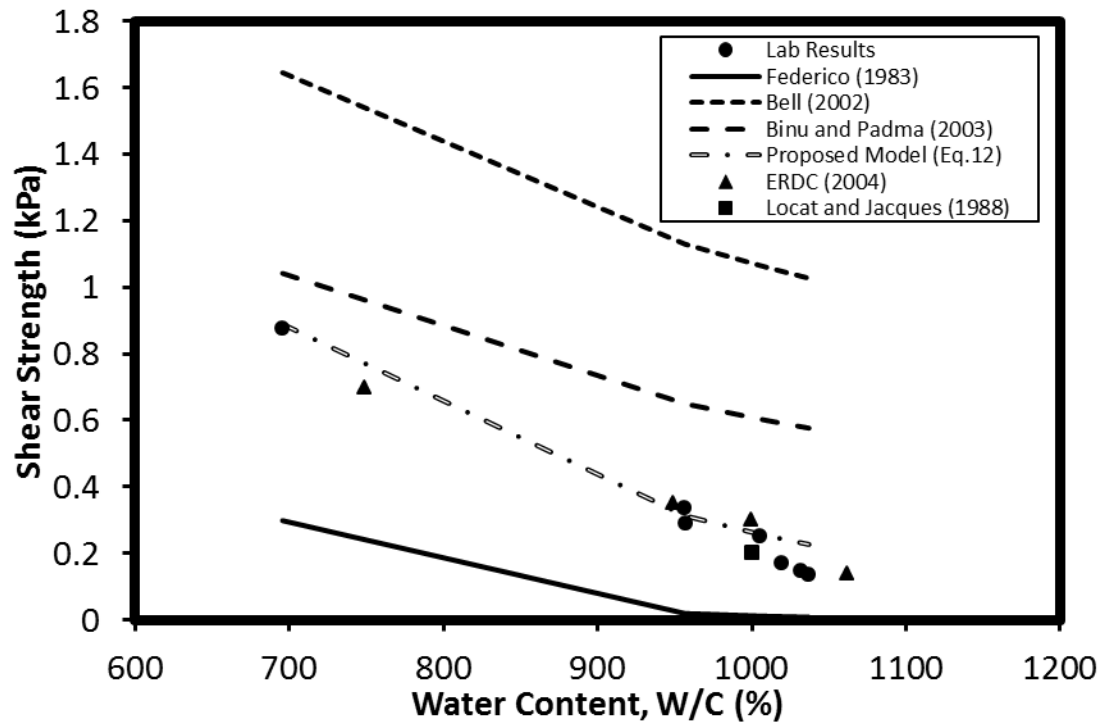


Figure 4.116 Comparison between the proposed relationship (Eq. 4.16) and previous methods to estimate the shear strength-moisture content of ultra-soft soil.

4.13. Anchor tests

4.13.1. Experimental tests

In this study, a displacement controlled device for anchor testing in compression and tension loading under different rates has been shown in Figure 4.117. Three different material types of anchor model made of steel, plastic, and concrete with a diameter of 12.5 cm and a length of 30 cm has been tested in an ultra-soft soil prepared of 10% bentonite content. The test was performed in two different rates of 1 and 10 mm / min for tension and compression loading conditions. The anchor pullout skin friction behavior can be idealized as shown in Figure 4.118. The pullout skin friction can be quantified as:

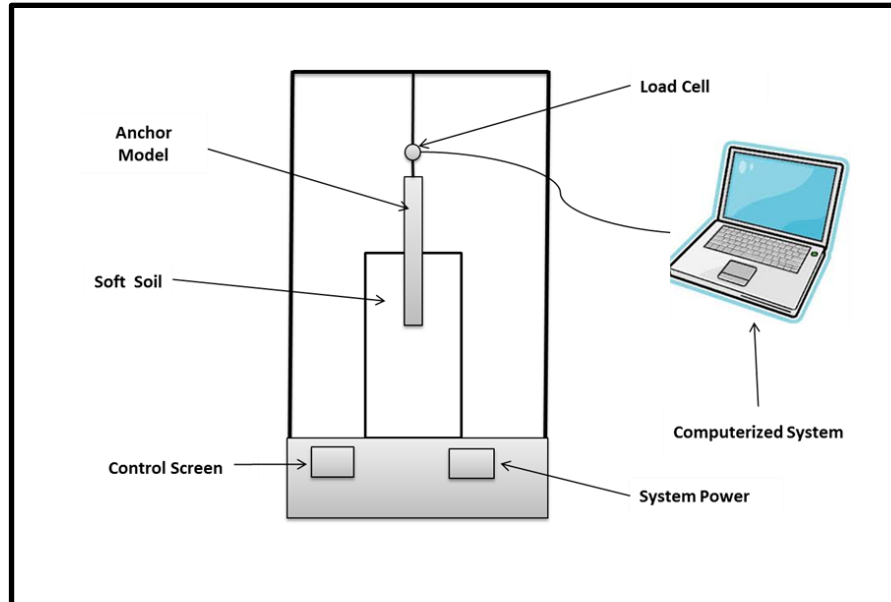


Figure 4.117 Laboratory set up of anchor testing.

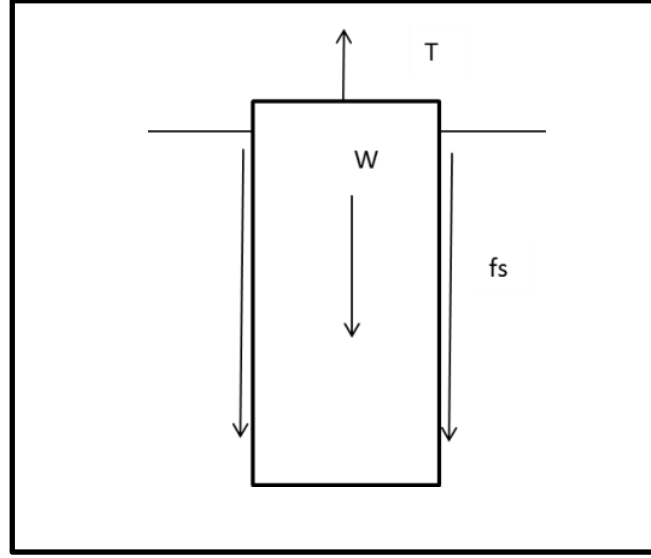


Figure 4.118 Anchor pullout-skin friction idealization.

$$f_s * \pi * D * L + W = T + V\rho g \text{ and} \quad (4.17)$$

$$f_s = \frac{T + V\rho g - W}{\pi * D * L}, \quad (4.18)$$

where f_s is the skin friction, T is the applied load, W is the weight of the anchor, ρ is the density of the soft soil, g is the gravitational acceleration, D is the anchor diameter, L is the submerged length of the anchor.

Anchor pullout skin friction versus relative displacement in 10% bentonite ultra-soft soil for different types of anchors (concrete, steel and plastic) of two different rates (1 and 10 mm/min) have shown in Figure 4.119 and Figure 4.120 respectively. It is clearly denoted that the rate influenced the skin friction-relative displacement relationship showing almost 30% higher values of the skin friction for a lower rate (1 mm/min) which obviously identified the ultra-soft soil depends on the displacement rate. The skin friction

of concrete anchor showed higher values compared to steel and plastic anchors due to the higher surface roughness in concrete anchor compared to the others. As the relative displacement increased from 0 to 0.04, the pullout skin friction increased by 54%, 64%, and 79% for plastic, steel, and concrete anchors at 1 mm/min loading rate condition respectively. As the relative displacement increased from 0 to 0.04, the pullout skin friction increased by 55%, 66%, and 81% for plastic, steel, and concrete anchors at 10 mm/min loading rate condition respectively.

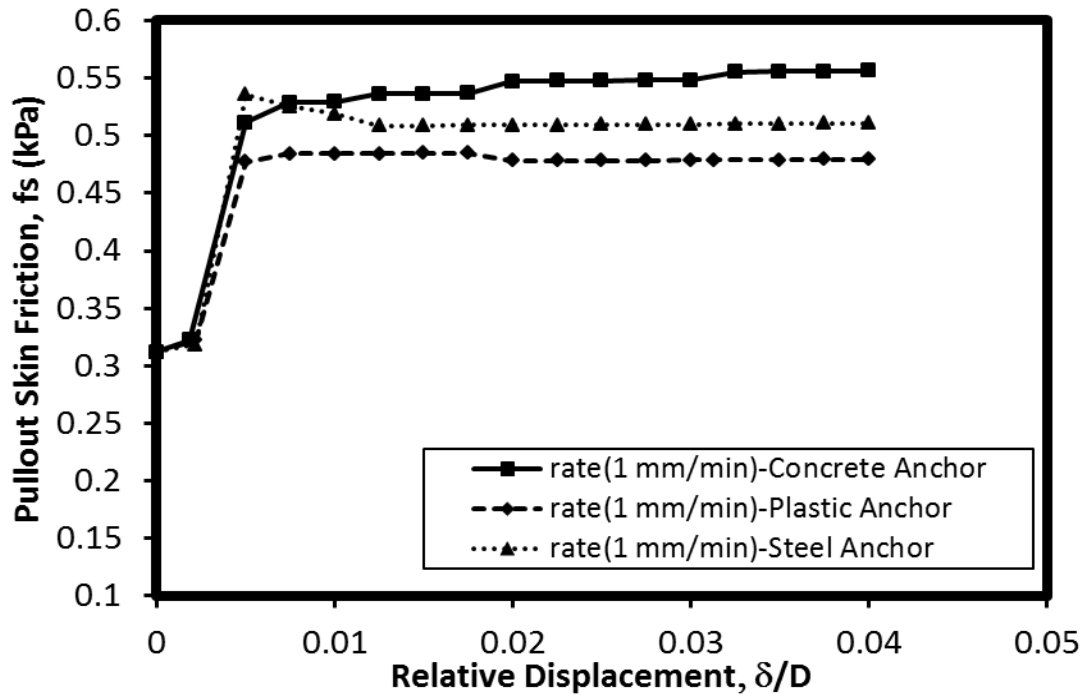


Figure 4.119 Pullout skin friction vs. relative displacement relationship of different types of anchor in (10%) bentonite soft soil under tension loading (rate = 1 mm/min).

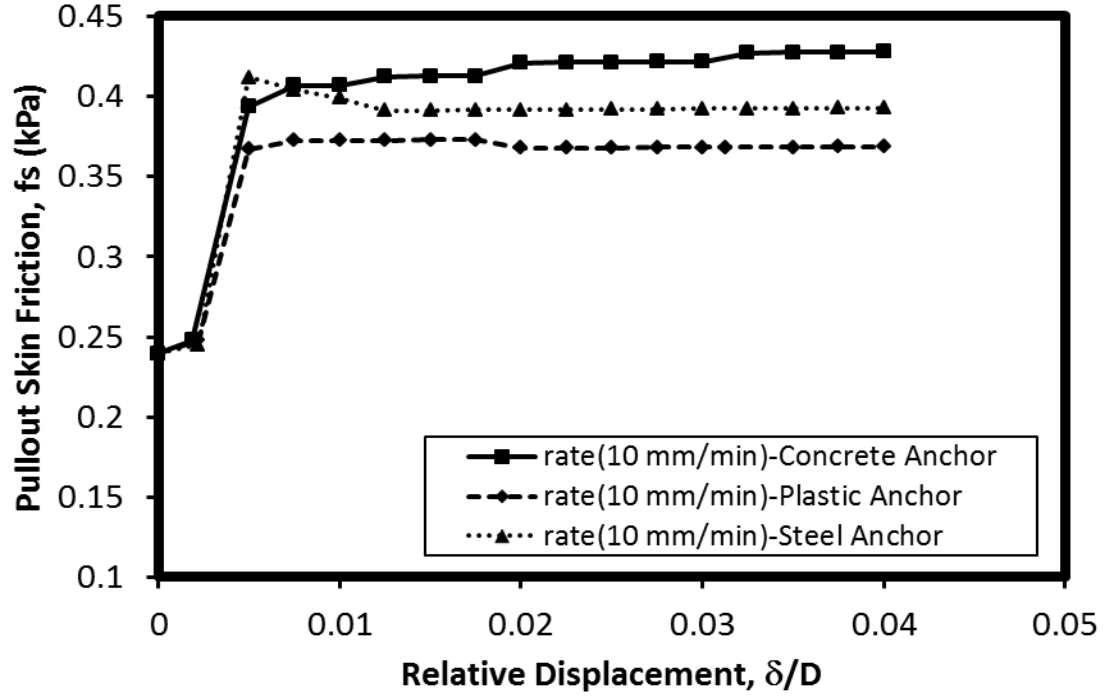


Figure 4.120 Pullout skin friction vs. relative displacement relationship for different types of anchor in (10%) bentonite soft soil under tension loading (rate = 10mm/min).

The pullout skin friction vs. relative displacement has been modeled using the following relationship

$$\text{Pullout Skin Friction}(f_s) = \frac{\delta/D}{A(\dot{\delta}) + B(\dot{\delta}) * \delta/D} \quad (4.19)$$

where $A(\dot{\delta})$ and $B(\dot{\delta})$ are model parameters, δ is the anchor displacement, D is the anchor diameter, $\dot{\delta}$ is the testing displacement rate.

Also, using the α -method, the skin friction can be related to the undrained shear strength of the soft soil (S_u) as follows (Tomlinson, 1957)

$$f_s \left(\frac{\dot{\delta}}{D} \right) = \alpha \left(\frac{\dot{\delta}}{D} \right) S_u \quad (4.20)$$

where $\alpha \left(\frac{\dot{\delta}}{D} \right)$ is the adhesion coefficient (depends on the rate of loading ($\dot{\delta}$)) And S_u is the shear strength of the clay soil. A summary of the pullout skin friction and adhesion factors are summarized in Table 4.32.

Table 4.32 Pullout Skin Friction and Adhesion Coefficient for Different Displacement Rates.

Anchor	$\dot{\delta}=1$ mm/min		$\dot{\delta}=10$ mm/min	
	fs (kPa)	α	fs (kPa)	α
Concrete	0.57	3.34	0.44	2.57
Steel	0.51	3.01	0.39	2.31
Plastic	0.49	2.86	0.38	2.21

Modeling of the pullout skin friction versus relative displacement of different types of anchors (concrete, steel and plastic) for two different rates (1 and 10 mm/min) using Eq. (4.19) have shown in Figure 4.121 and Figure 4.122 respectively. The proposed model has shown a very good agreement for both simulated cases. The R^2 were 0.98, 0.97, and 0.95 for concrete, plastic, and steel anchor at 1 mm/min loading rate condition respectively. The R^2 were 0.97, 0.96, and 0.95 for concrete, plastic, and steel anchor at 10 mm/min loading rate condition respectively.

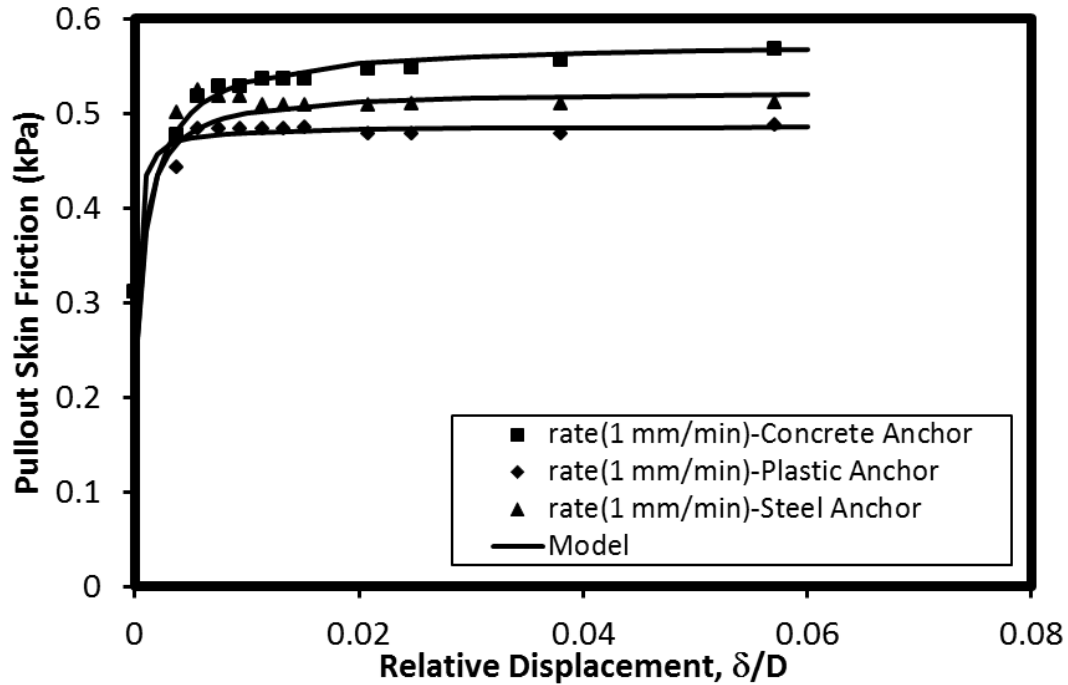


Figure 4.121 Modeling of pullout skin friction vs. relative displacement relationship for different types of anchor in bentonite soft soil (bentonite=10%) under tension loading (rate = 1 mm/min).

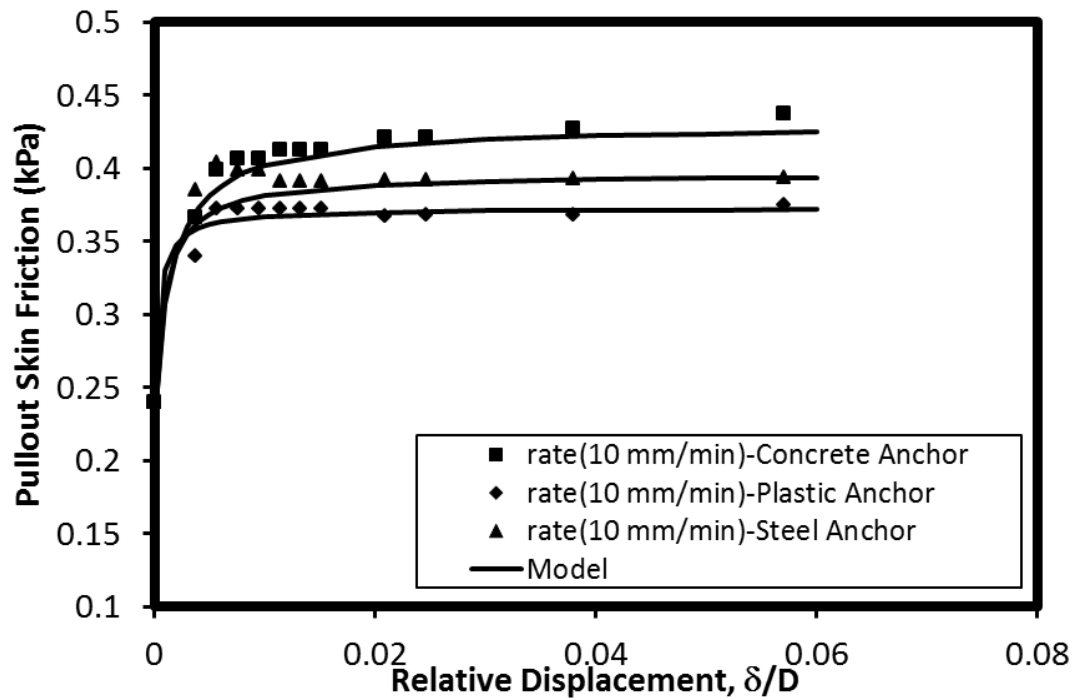


Figure 4.122 Modeling of pullout skin friction vs. relative displacement relationship for different types of anchor in bentonite soft soil (bentonite=10%) under tension loading (rate = 10 mm/min).

Similarly, the variation of the anchor compression friction with the relative displacement in 10% bentonite ultra-soft soil for different types of anchors (concrete, steel and plastic) with two different loading rates (1 and 10 mm/min) have been shown in Figure 4.123 and Figure 4.124 respectively. It is clearly denoted that the rate influenced the skin friction-relative displacement relationship showing 31% higher values of the skin friction for a lower rate (1 mm/min), which obviously identified the ultra-soft soil depends on the displacement rate in compression loading condition as well. The skin friction of concrete anchor showed higher values compared to steel and plastic anchors due to the higher surface roughness in concrete anchor compared to the others. As the relative displacement increased from 0 to 0.057, the compression skin friction increased by 57%, 63%, and 86% for steel, plastic, and concrete anchors at 1 mm/min loading rate condition respectively. As the relative displacement increased from 0 to 0.057, the compression skin friction increased by 57%, 65%, and 84% for steel, plastic, and concrete anchors at 10 mm/min loading rate condition respectively.

Similarly, the variation of the compression skin friction with relative displacement is modeled using Eq. 4.19. A summary of the compression skin friction and adhesion factors are summarized in Table 4.33.

Table 4.33 Compression skin friction and adhesion coefficient for different displacement rates.

Anchor	$\dot{\delta}=1$ mm/min		$\dot{\delta}=10$ mm/min	
	f_s (kPa)	α	f_s (kPa)	α
Concrete	0.75	4.41	0.56	3.31
Steel	0.67	3.92	0.52	3.06
Plastic	0.64	3.74	0.49	2.89

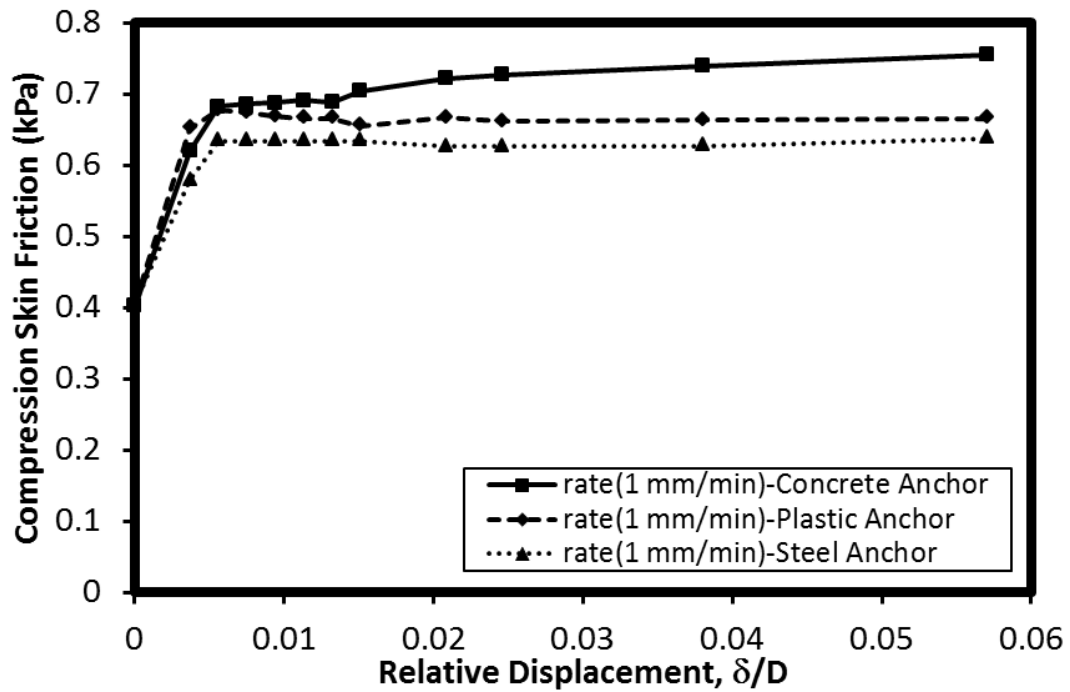


Figure 4.123 Compression skin friction vs. relative displacement relationship of different types of anchor in (10%) bentonite soft soil under tension loading (rate = 1 mm/min).

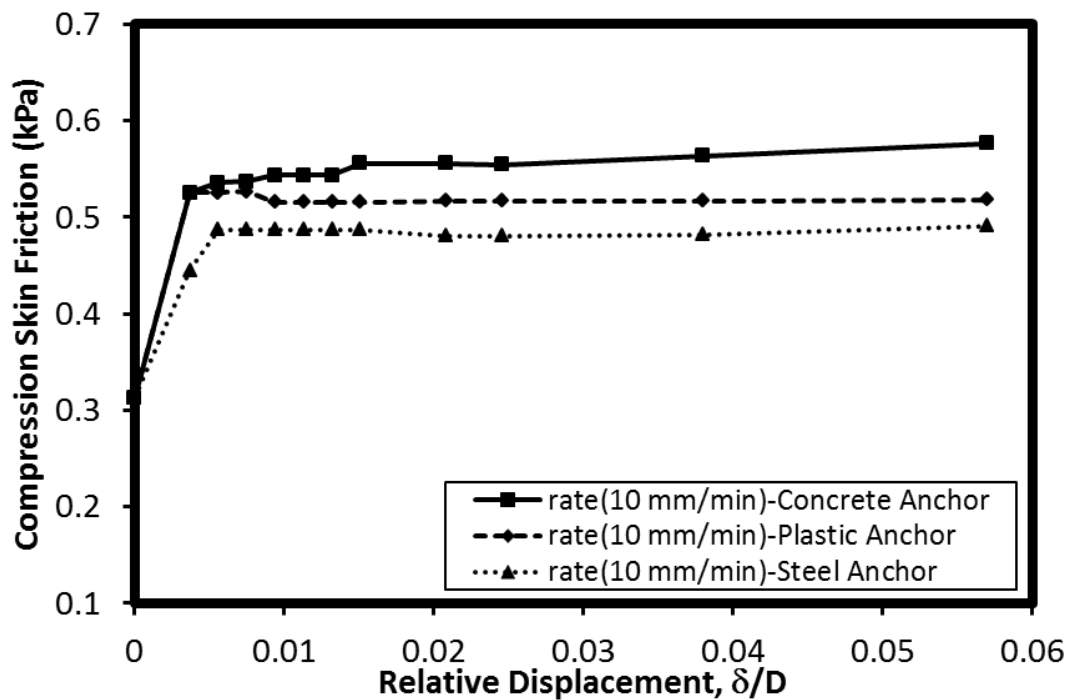


Figure 4.124 Compression friction vs. relative displacement relationship for different types of anchor in (10%) bentonite soft soil under tension loading (rate = 10mm/min).

Modeling of the compression skin friction versus relative displacement of different types of anchors (concrete, steel and plastic) for two different rates (1 and 10 mm/min) using Eq. (4.19) have shown in Figure 4.125 and Figure 4.125 respectively. The proposed model has shown a very good agreement for both simulated cases. The R^2 were 0.94, 0.96, and 0.98 for concrete, plastic, and steel anchor at 1 mm/min loading rate condition respectively. The R^2 were 0.95, 0.96, and 0.97 for concrete, plastic, and steel anchor at 10 mm/min loading rate condition respectively.

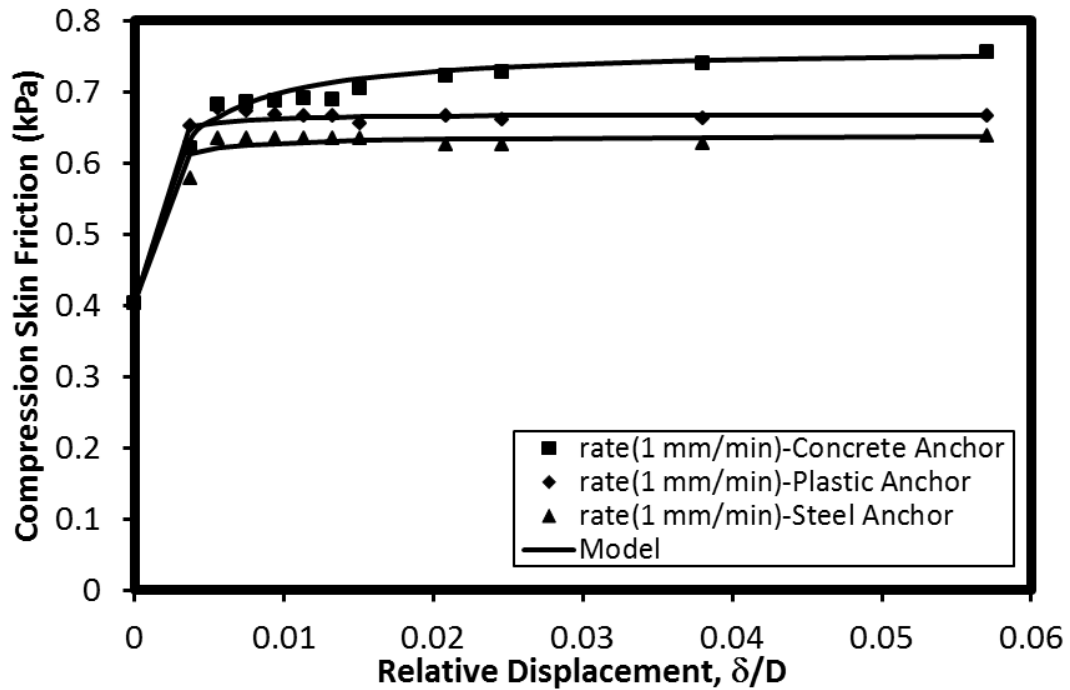


Figure 4.125 Modeling of compression skin friction vs. relative displacement relationship for different types of anchor in bentonite soft soil (bentonite=10%) under tension loading (rate = 1 mm/min).

4.13.2. Finite element modeling (FEM)

An elasto-plastic Mohr-Coulomb model was selected from those available in PLAXIS to describe the non-linear soft soil behavior in this study. When subjected to primary deviatoric loading, cohesive soil shows a decreasing stiffness and simultaneously

irreversible plastic strains develop. Limiting states of stress are described by means of the undrained shear strength of the soft soil. The initial stresses in the case of soft soil were generated using Jaky's formula which gives the at rest earth pressure coefficient $K_0 = 1 - \sin\phi$ where ϕ is the friction angle in terms of effective stress. Values of soil and anchor parameters used in the main investigation are shown in Table 4.34 and Table 4.35 respectively.

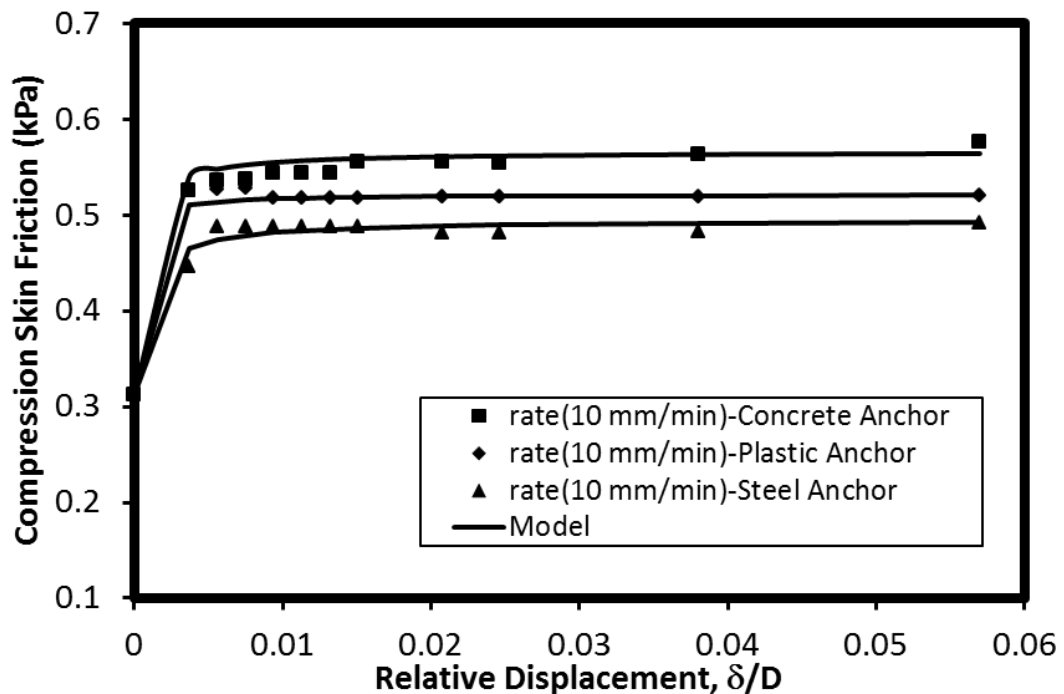


Figure 4.126 Modeling of compression skin friction vs. relative displacement relationship for different types of anchor in bentonite soft soil (bentonite=10%) under tension loading (rate = 10 mm/min).

The analysis was performed using the 3-D model for anchor in ultra-soft soil. During the generation of the mesh, 2644 elements of 10-noded tetrahedral elements with 4440 nodes having an average element size of 1.15 cm were selected in order to provide a great accuracy in the determination of stresses. The meshes were chosen to match the corresponding prototype geometries in the experimental model test and were 12.5 cm

wide with depths of 30 cm. PLAXIS incorporates a fully automatic mesh generation procedure, in which the geometry was divided into elements of the basic element type, and compatible embedded pile structural elements. Finite element mesh for the anchor-soft soil analysis is shown in Figure 4.127.

The cylindrical shape anchor was represented as a rigid embedded pile. In PLAXIS soil/structure interface behavior was modelled using the adhesion coefficient, α in Eq. (4.20). For the boundary condition, bottom was fixed and the vertical sides were friction free. Loading was applied in very small increments of the total applied load.

Table 4.34 Values of soil parameters used in FEM analysis

Parameter	Soft Soil
Unit Weight, γ (kN/m ³)	11
Elastic Modulus (kN/m ²)	100
Undrained Shear Strength (kN/m ²)	0.17
Poisson's Ratio μ	0.45
At Rest Earth Pressure Coefficient K_o	1
α	0.6

Table 4.35 Values of steel anchor parameters used in FEM analysis

Parameter	Anchor
Unit Weight, γ (kN/m ³)	78
Elastic Modulus (GPa)	200
Diameter (cm)	12.5
Length (cm)	30

The finite element analysis of steel anchor with different pullout skin friction in ultra-soft soil under tension loading has been used to predict the load-displacement relationship for steel anchor as shown in Figure 4.128. A very good agreement between the experimental and finite element analysis can be noticed with an increase in the load-displacement prediction of 30% when an increase of 20% in the pullout skin friction applied. Similarly, the finite element prediction for the load-displacement relationship for steel anchor with ultra-soft soil under compression loading condition is shown in Figure 4.129. As the loading changed from tension to compression, the load capacity for the soft soil increased by 40% for steel anchor in 10% bentonite ultra-soft soil.

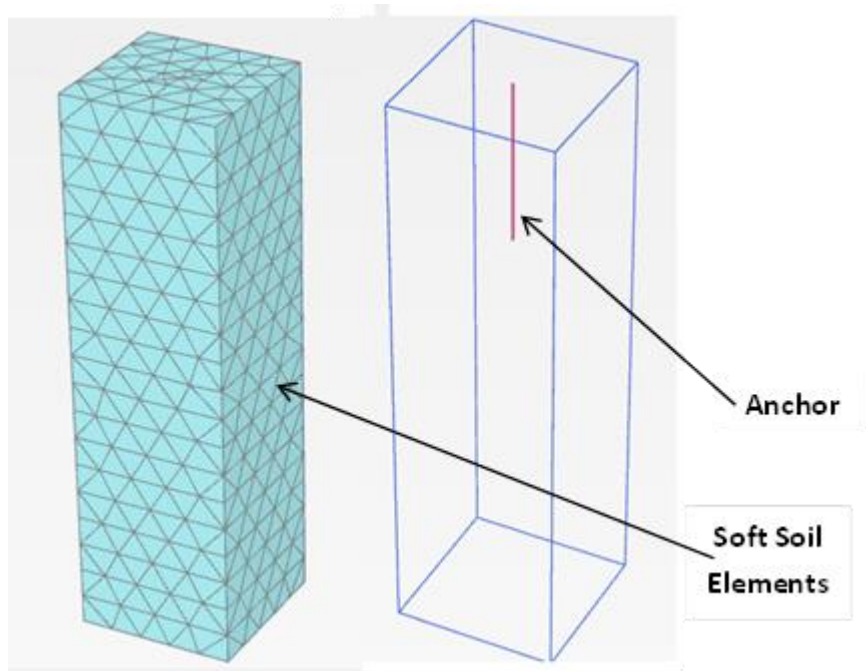


Figure 4.127 3D finite element mesh used for the analysis.

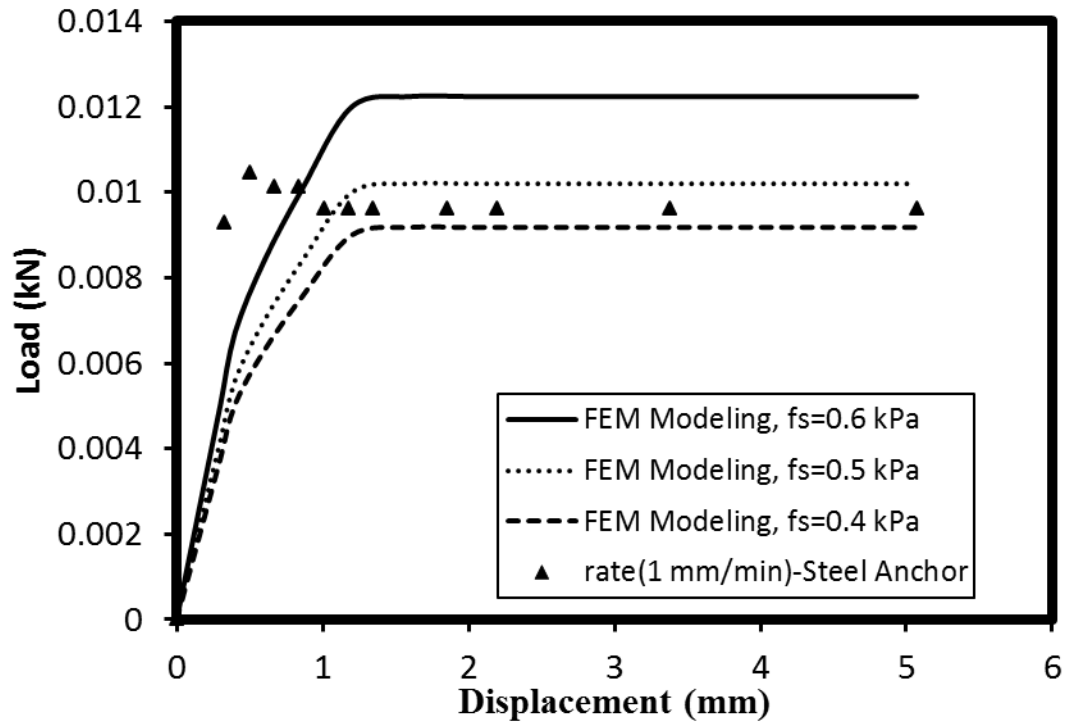


Figure 4.128 Finite element modeling for steel anchor under tension loading.

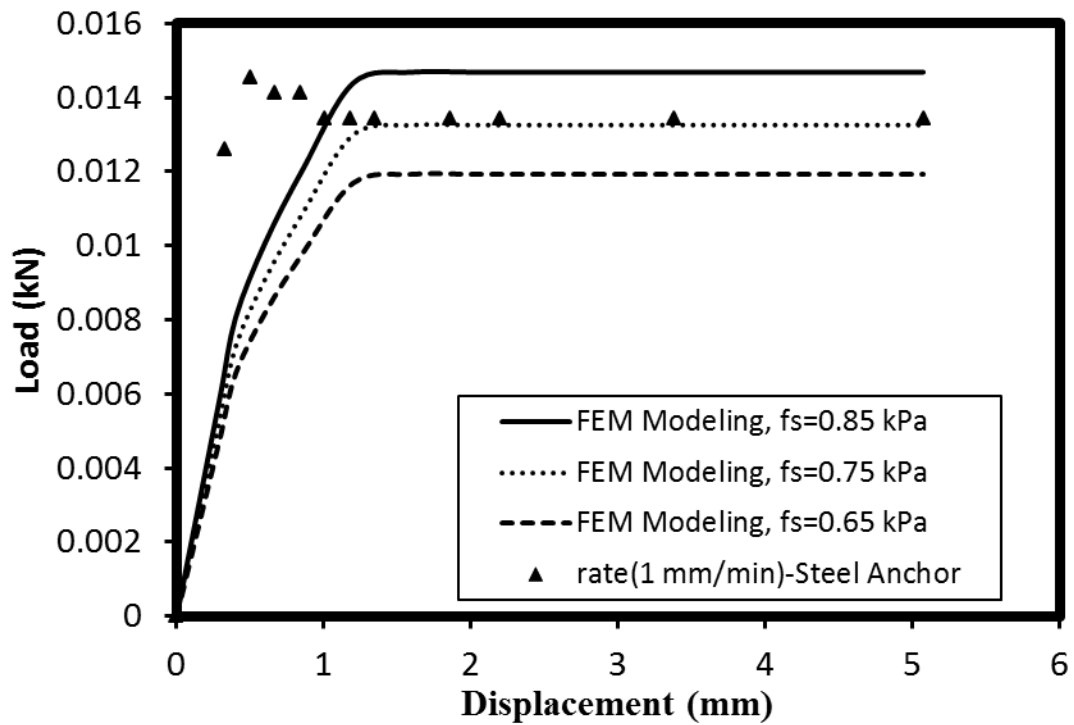


Figure 4.129 Finite element modeling for steel anchor under compression loading.

4.14. Summary

In this chapter, the following points can be summarized:

- 1.The shear strength of the bentonite soft soil increased from 0.01 kPa to 0.17 kPa and the water content decreased from 98% to 90 % as the bentonite content increased from 2% to 10% respectively.
- 2.As bentonite content in the soft soil increased from 2% to 10%, the initial electrical resistivity decreased from 6.29 Ohm.m to 2.56 Ohm.m.
- 3.The soft soil density increased from 1.012 gm/cm³ to 1.066 gm/cm³ as the bentonite content increased from 2% to 10%.
- 4.The drop in the electrical impedance of the 2% and 10% bentonite soft soil has shown a significant value from 2.67 k.Ohm to 1.71 k.Ohm and 1.6k.Ohm to 0.607 k.Ohm at 0.02 kHz and 300 kHz respectively.
- 5.The shear strength of the kaolinite soft soil increased from 0.01 kPa to 0.14 kPa and the water content decreased from 98% to 90 % as the kaolinite content increased from 2% to 10% respectively.
- 6.As kaolinite content in the soft soil increased from 2% to 10%, the initial electrical resistivity decreased from 23.9 Ohm.m to 13 Ohm.m.
- 7.The soft soil density increased from 1.011 gm/cm³ to 1.065 gm/cm³ as the kaolinite content increased from 2% to 10%.
- 8.The drop in the electrical impedance of the 2% and 10% bentonite soft soil has shown a significant value from 19.56 k.Ohm to 10.68 k.Ohm and 2.97 k.Ohm to 0.71 k.Ohm at 0.02 kHz and 300 kHz respectively.

9. It is clearly shown that both shear strength and water content for the 10% bentonite treated soft soil decreased from 0.17 kPa to 0.15 kPa and from 90% to 80 % as the lime content increased from 0% to 10% respectively with an optimum shear strength of 0.27 kPa at 2% lime treatment.
10. As lime content increased from 2% to 10 % in the 10% bentonite treated soft soil , the initial resistivity decreased from 0.65 Ohm.m to 0.38 Ohm.m.
11. The soft soil density increased from 1.024 gm/cm³ to 1.13 gm/cm³ as the lime content increased from 2% to 10% in the 10% bentonite treated soft soil.
12. The drop in the electrical impedance for the 10% bentonite treated soft soil with 2% to 10% lime has shown a significant value from 0.707 k.Ohm to 0.414 k.Ohm and 0.28 k.Ohm to 0.16 k.Ohm at 0.02 kHz and 300 kHz respectively.
13. It is clearly shown that shear strength for the 10% bentonite treated soft soil increased from 0.17 kPa to 6.7 kPa and the water content decreased from 90% to 80% as the polymer content increased from 0% to 10% respectively.
14. As polymer content increased from 0% to 10 % in the 10% bentonite treated soft soil , the initial resistivity decreased from 2.56 Ohm.m to 0.47 Ohm.m.
15. The soft soil density increased from 1.092 gm/cm³ to 1.16 gm/cm³ as the polymer content increased from 0% to 10% in the 10% bentonite treated soft soil.
16. The drop in the electrical impedance for the 10% bentonite treated soft soil with 0% to 10% polymer has shown a significant value from 1.71 k.Ohm to 0.394 k.Ohm and 0.607 k.Ohm to 0.176 k.Ohm at 0.02 kHz and 300 kHz respectively.

17. It is clearly shown that both shear strength and water content for the 10% bentonite treated soft soil decreased from 0.17 kPa to 0.055 kPa and from 90% to 87% as the sand content increased from 0% to 3% respectively.
18. As sand content increased from 0% to 3 % in the 10% bentonite treated soft soil, the initial resistivity increased from 4.5 Ohm.m to 5.55 Ohm.m.
19. The soft soil density increased from 1.073 gm/cm^3 to 1.085 gm/cm^3 as the sand content increased from 0% to 3% in the 10% bentonite treated soft soil.
20. The drop in the electrical impedance for the 10% bentonite treated soft soil with 0% to 3% sand has shown a significant value from 1.71 k.Ohm to 0.829 k.Ohm and 0.607 k.Ohm to 0.421 k.Ohm at 0.02 kHz and 300 kHz respectively.
21. It is clearly shown that the shear strength for the 10% bentonite treated soft soil increased from 0.17 kPa to 0.233 kPa and the water content decreased from 90% to 80% as the cement content increased from 0% to 10% respectively.
22. As cement content increased from 0% to 10 % in the 10% bentonite treated soft soil, the initial resistivity decreased from 2.1 Ohm.m to 0.7 Ohm.m.
23. The soft soil density increased from 1.066 gm/cm^3 to 1.135 gm/cm^3 as the cement content increased from 0% to 10% in the 10% bentonite treated soft soil.
24. The drop in the electrical impedance for the 10% bentonite treated soft soil with 0% to 10% cement has shown a significant value from 1.71 k.Ohm to 1.077 k.Ohm and 0.607 k.Ohm to 0.382 k.Ohm at 0.02 kHz and 300 kHz respectively.
25. It is clearly shown that shear strength for the 10% bentonite treated soft soil increased from 0.15 kPa to 0.26 kPa and the water content decreased from 90% to 80% as the kaolinite content increased from 0% to 10% respectively.

26. As kaolinite content increased from 2% to 10 % in the 10% bentonite treated soft soil, the initial resistivity decreased from 2.15 Ohm.m to 1.05 Ohm.m.
27. The soft soil density increased from 1.078 gm/cm^3 to 1.125 gm/cm^3 as the kaolinite content increased from 2% to 10% in the 10% bentonite treated soft soil .
28. The drop in the electrical impedance for the 10% bentonite treated soft soil with 2% to 10% kaolinite has shown a significant value from 1.05 k.Ohm to 0.8 k.Ohm and 0.64 k.Ohm to 0.534 k.Ohm at 0.02 kHz and 300 kHz respectively.
29. The interface shear is almost 50% of the soft soil shear strength and similar trends can be exhibited for both shear strength and interface shear.
30. As the lime content increased from 0% to 20%, the interface shear decreased from 0.085 kPa to 0.074 kPa with optimum interface shear of 0.094 kPa at 5% lime content while the water content decreased from 90% to 70%.
31. As the polymer content increased from 0% to 10%, the shear strength and interface shear increased from 0.17 kPa to 6.7 kPa and 0.1 kPa to 3.6 kPa respectively.
32. The interface shear strength of treated soft soil with 10% polymer was 49 times greater than the interface shear strength of treated soft soil with 20% lime.
33. Electrical resistivity method is used to mointor the particle settling over the time where hydrometer test is not capable to capture the long-term particle settlement.
34. As the curing time for 10% treated bentonite soft soil with lime is increased from 0 day to 28 days, the shear strength increased by 95% and 1959% for 0% and 20% lime contents respectively.

35. As the curing time for 10% treated bentonite soft soil with polymer is increased from 0 day to 28 days, the shear strength increased by 95% and 313% for 0% and 5% polymer contents respectively.
36. As the bentonite content increased from 2% to 10%, the CIGMAT penetrometer penetration decreased by 85%, 79%, 71% and 69% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.
37. As the lime content in the 10% bentonite treated soft soil is increased from 2% to 10%, the penetrometer penetration increased by 73%, 61%, 57% and 50% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.
38. As the polymer content in the 10% bentonite treated soft soil is increased from 1% to 10%, the penetrometer penetration decreased by 96%, 87%, 84% and 83% for 7 gm, 14 gm, 21 gm and 28 gm of CIGMAT penetrometer weight respectively.
39. As the CIGMAT penetrometer increased by four times, the maximum effective stress zone and effective depth increased by 41% and 48% for untreated ultra-soft soil ($S_u=0.17$ kPa) and by 75% and 67% for treated ultra-soft soil ($S_u=6.8$ kPa) respectively.
40. The lime had the capability to increase the pH of the treated ultra-soft soil. As the lime content increased from 0% to 10%, the pH increased by 24%.
41. The polymer had the potential to increase the pH of the treated ultra-soft soil. As the polymer content increased from 0% to 10%, the pH increased by 37%.
42. As the bentonite content increased from 2% to 10%, the peak stress and ultimate strain increased by 800% and 150% respectively.
43. As the bentonite content in the soft soil increased from 2% to 10%, the compression wave velocity decreased by 15%.

44. As the relative displacement increased from 0 to 0.04, the pullout skin friction increased by 54%, 64%, and 79% for plastic, steel, and concrete anchors in 10% bentonite soft soil at 1 mm/min loading rate condition respectively.

45. As the relative displacement increased from 0 to 0.04, the pullout skin friction increased by 55%, 66%, and 81% for plastic, steel, and concrete anchors in 10% bentonite at 10 mm/min loading rate condition respectively.

Chapter 5 - Filter Cake Characterization and Modeling

5.1. Introduction

In this chapter, the filter cake formation was modeled more precisely using a combination of seepage-consolidation phenomenon. At the start, as the drilling mud is pressurized, the fluid loss is due to the water seepage since no filter cake is formed yet. Then, after a particular time, the cake started to form, then the fluid loss is due to both seepage of the water and consolidation of the already formed filter cake. To model seepage-consolidation phenomenon, both seepage and Terzaghi consolidation models were reviewed. Terzaghi consolidation solution assumed that the coefficient of permeability and consolidation are constants throughout the consolidation process. To model the consolidation more accurately, a new solution for consolidation equation was provided. In this solution, a coupling function of time and elevation was used to express the excess pore pressure function. In the new proposed solution, the permeability and coefficient of consolidation were time dependent functions. The proposed solution was verified against Terzaghi solution and API model for long-term experimental results for both 2%, and 8% bentonite drilling mud under a constant pressure of 100 psi and different temperatures of 25°, 50°, 75°, and 100° C. The verification included, the variations of the fluid loss, permeability, coefficient of consolidation, and excess pore water pressure with the time.

The filtering process of drilling mud forming filter cake has been examined. The modeling of the filtering process has been inspected through currently in use model (API-model). The shortcoming and assumptions of the API-model is addressed and a new kinetic model is proposed. The API-model assumptions has been resolved accurately in

the new kinetic model. Several assumptions of the API-model such as infinity fluid loss at infinity time period, constant filter cake permeability during filter cake formation, constant relative solid content in the filter cake to the mud, constant cake porosity during cake formation, ignoring the effect of both temperature and pressure on the fluid loss and cake formation has been resolved using the kinetic model. Both API and kinetic model's prediction were verified through a series of laboratory experiments results and results from the literature. The laboratory verification included the fluid loss versus time prediction of 2% and 8% bentonite drilling mud under a constant applied pressure of 100 psi and two different tested temperatures of 25°, and 100° C. In addition, the variations of the cake permeability, relative solid contents, cake porosity, and cake thickness over the time were predicted reasonably. Both API and kinetic model's fluid loss versus time predictions were verified for literature results of 5% bentonite drilling mud under 300 psi and 225° C, 4% bentonite with xanthan gum additive under 100 psi and 25° C, easter drilling mud with salt (NaCl) under 100 psi and 25° C, modified cement with nanosilica additive under 100 psi and 25° C, and 4% bentonite with biopolymer additive under 100 psi and 25° C.

5.2.Filter cake seepage-consolidation combination

Suppose you have the filter cake with the external pressure (P) as shown below. Due to the applied pressure on the surface of the cake, the cake is subjected to a combined effect of seepage and consolidation. Seepage-consolidation idealization during filter cake formation can be shown in Figure 5.1.

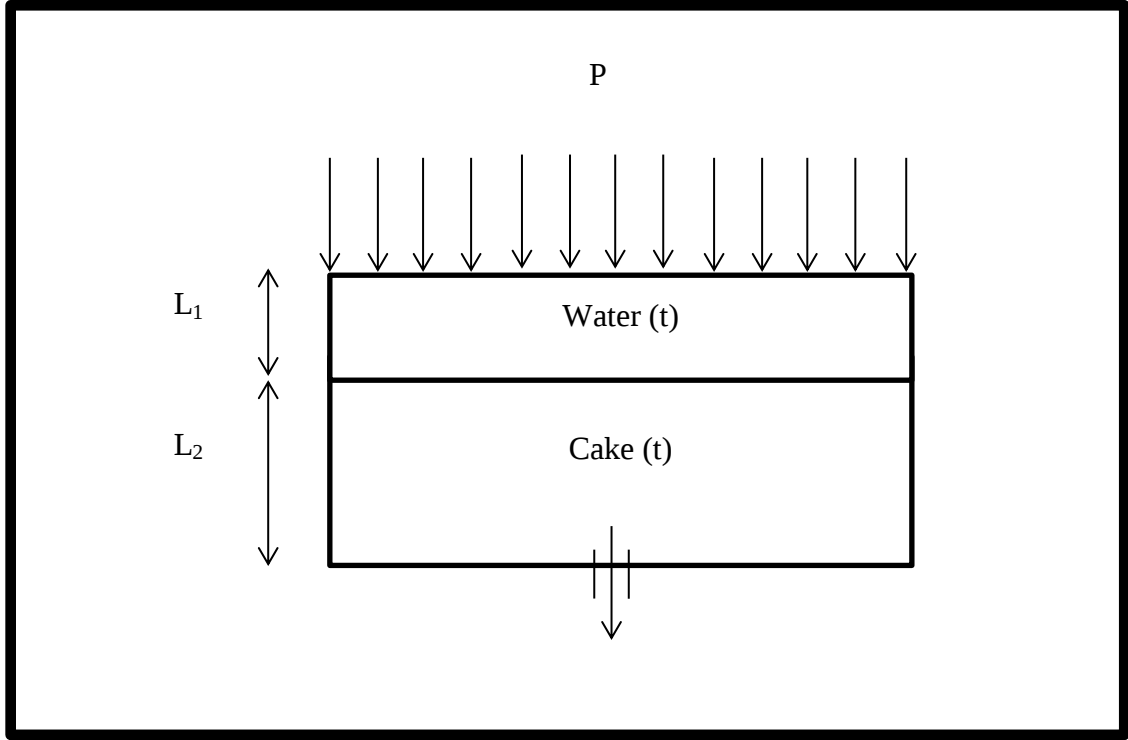


Figure 5.1 Seepage-consolidation idealization during filter cake formation.

To properly model the behavior of the filter cake, the boundary condition of the cake should be defined correctly for precious validation of the results with the experimental observations. The filter cake can be modeled using a 1-D simplification of both seepage and consolidation since the fluid loss can occur experimentally only through a small opening (diameter = 1 mm) in the bottom while the vertical sides are impervious. Hence, the total amount of the fluid loss can be identified as

$$\Delta Q_T(t) = \Delta Q_1(t) + \Delta Q_2(t), \quad (5.1)$$

where $\Delta Q_T(t)$ is the total amount of the fluid loss during the test, $\Delta Q_1(t)$ is the amount of the fluid loss due to seepage while $\Delta Q_2(t)$ is the amount of the fluid loss due to consolidation.

5.2.1.Seepage

The 3-D block has dimensions dx , dy , and dz . Let v_x , v_y , and v_z be the components of the discharge velocity in x , y , and z direction. The rate of flow of water into the elemental block in x , y , and z directions are $v_x dy dz$, $v_y dx dz$, and $v_z dx dy$ respectively. Figure 5.2 idealized the inflow and outflow in the 3-D scheme. The rates of outflow from the block in the x , y , and z directions are as

$$\left(v_x + \frac{\partial v_x}{\partial x}\right) dydz, \quad (5.2)$$

$$\left(v_y + \frac{\partial v_y}{\partial y}\right) dx dz, \text{ and} \quad (5.3)$$

$$\left(v_z + \frac{\partial v_z}{\partial z}\right) dx dy. \quad (5.4)$$

Assuming that water is incompressible and that no volume change in the soil mass occurs. Then, the total rate of inflow should equal the total rate of outflow. Thus,

$$\left[\left(v_x + \frac{\partial v_x}{\partial x}\right) dydz + \left(v_y + \frac{\partial v_y}{\partial y}\right) dx dz + \left(v_z + \frac{\partial v_z}{\partial z}\right) dx dy\right] - [v_x dydz + v_y dx dz + v_z dx dy] = 0, \quad (5.5)$$

or,

$$\left[\left(\frac{\partial v_x}{\partial x}\right) + \left(\frac{\partial v_y}{\partial y}\right) + \left(\frac{\partial v_z}{\partial z}\right)\right] = 0. \quad (5.6)$$

With Darcy's law, the discharge velocities can be expressed as

$$v_x = i_x k_x = k_x \frac{\partial h}{\partial x}, \quad (5.7)$$

$$v_y = i_y k_y = k_y \frac{\partial h}{\partial y}, \text{ and} \quad (5.8)$$

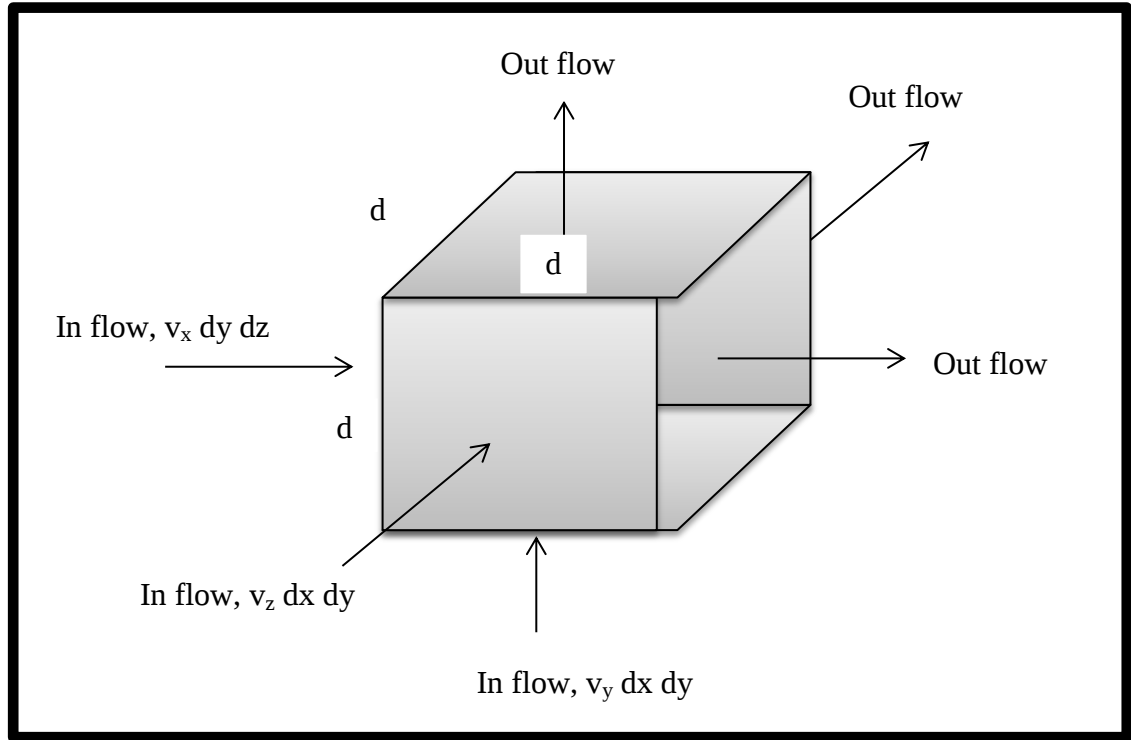


Figure 5.2 Inflow and outflow idealization scheme in 3-D.

$$v_z = i_z k_z = k_z \frac{\partial h}{\partial z}, \quad (5.9)$$

where h is the total head, k_x, k_y , and k_z are the hydraulic conductivity in x , y , and z directions respectively. From Eqs. (5.7, 5.8, and 5.9), then

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0. \quad (5.10)$$

If the soil is isotropic with respect to hydraulic conductivity that $k_x=k_y=k_z$, the Eq. (5.10) can be further simplified to

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0, \quad (5.11)$$

the seepage model for 1-D condition can be represented as

$$\Delta Q_1(t) = k(t) * \frac{\partial h}{\partial z} * A_o, \quad (5.12)$$

from Continuity (1-D): $\frac{\partial^2 h}{\partial z^2} = 0 \Rightarrow \frac{\partial h}{\partial z} = A_1$ and

$$\Delta Q_1(t) = k(t) * A_1 * A_o. \quad (5.13)$$

5.2.2.Consolidation

(i) Terzaghi consolidation equation

The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi (1943) (Figure 5.3). The underlying assumptions in the derivation of the mathematical equations are as

1. The clay layer is homogeneous.
2. The clay layer is saturated.
3. The compression of the soil layer is due to the change in volume only, which, in turn, is due to the squeezing out of water from the void spaces.
4. Darcy's law valid.
5. Deformation of soil occurs only in the direction of the load application.
6. The coefficient of consolidation C_v is constant during the consolidation.

With the above assumptions, let us consider a clay layer of thickness H_1 as shown in Figure 5.3. The layer is located between two highly permeable sand layers. In this case of one-dimensional consolidation, the flow of water into and out of the soil element is in one direction only, i.e., in the z direction. This means that q_x , q_y , dq_x , and dq_y equal to zero, and thus the rate of flow into and out of the soil element can be given by

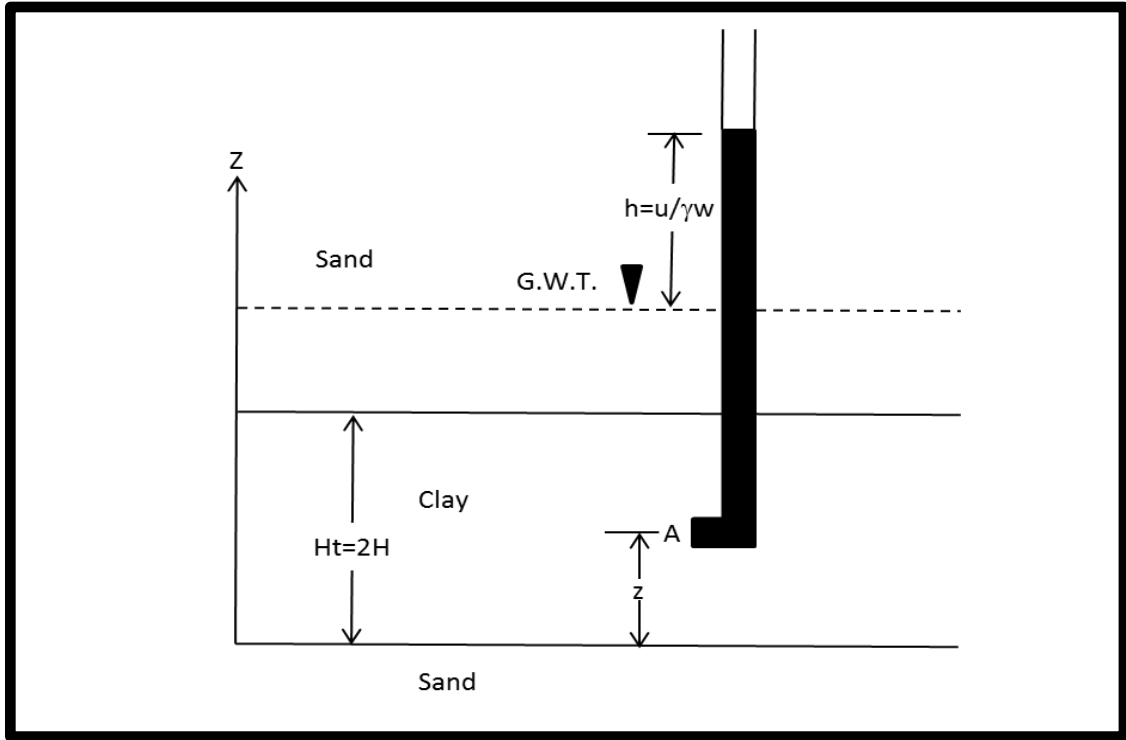


Figure 5.3 Clay layer undergoing consolidation.

$(q_z + dq_z) - q_z = \text{rate of change of volume of soil element}$

$$= \frac{\partial V}{\partial t}, \quad (5.14)$$

where $V = d_x d_y d_z$, (5.15)

we obtain

$$k \frac{\partial^2 h}{\partial z^2} d_x d_y d_z = \frac{\partial V}{\partial t}, \quad (5.16)$$

where k is the coefficient of permeability ($k = k_z$). However,

$$h = \frac{u}{\gamma_w}, \quad (5.17)$$

where γ_w is the unit weight of water. Substitution of Eq. (5.17) and (5.16) and rearranging gives

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{d_x d_y d_z} \frac{\partial V}{\partial t}. \quad (5.18)$$

During consolidation, the rate of change of volume is equal to the rate of change of the void volume. So,

$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t}, \quad (5.19)$$

where V_v is the volume of voids in the soil element. But

$$V_v = eV_s, \quad (5.20)$$

where V_s is the volume of soil solids in the element, which is constant, and e is the void ratio. Therefore, the changes in the volume with respect to time can be represented as

$$\frac{\partial V}{\partial t} = V_s \frac{\partial e}{\partial t} = \frac{V}{1+e} \frac{\partial e}{\partial t} = \frac{d_x d_y d_z}{1+e} \frac{\partial e}{\partial t}. \quad (5.21)$$

Substituting the above relation into Eq. (5.18), we get

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e} \frac{\partial e}{\partial t}. \quad (5.22)$$

The change in void ratio, ∂e , is due to the increase of effective stress; assuming that these are linearly related, then

$$\partial e = -a_v \partial(\Delta\sigma). \quad (5.23)$$

Combining Eqs. (5.22) and (5.23),

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{a_v}{1+e} \frac{\partial u}{\partial t} = m_v \frac{\partial u}{\partial t}, \quad (5.24)$$

$$\text{where } m_v = \text{coefficient of volume compressibility} = \frac{a_v}{1+e}. \quad (5.25)$$

$$\text{Or, } \frac{\partial u}{\partial t} = \frac{k}{\gamma_w m_v} \frac{\partial^2 u}{\partial z^2} = C_v \frac{\partial^2 u}{\partial z^2}, \quad (5.26)$$

$$\text{where } C_v = \text{coefficient of consolidation} = \frac{k}{\gamma_w m_v}. \quad (5.27)$$

Eq. (5.26) is the basic differential equation of Terzaghi's consolidation theory and can be solved with proper boundary conditions. To solve the equation, assume u to be the product of two functions, i.e., the product of a function of z and a function of t , or

$$u = F(z)G(t), \quad (5.28)$$

$$\text{so, } \frac{\partial u}{\partial t} = F(z) \frac{\partial}{\partial t} G(t) = F(z)\dot{G}(t), \quad (5.29)$$

$$\text{and, } \frac{\partial^2 u}{\partial z^2} = \frac{\partial^2}{\partial z^2} F(z)G(t) = \ddot{F}(z)G(t), \quad (5.30)$$

$$F(z)\dot{G}(t) = C_v \ddot{F}(z)G(t) \text{ and} \quad (5.31)$$

$$\frac{\ddot{F}(z)}{F(z)} = \frac{\dot{G}(t)}{C_v G(t)}. \quad (5.32)$$

The left-hand side of Eq. (5.32) is a function of z only and is independent of t ; the right-hand side of the equation is a function of t only and is independent of z . Therefore, they must be equal to a constant ($-B^2$). Hence, the second derivatives of depth is

$$\ddot{F}(z) = -B^2 F(z), \quad (5.33)$$

a solution of Eq.(5.33) can be given by

$$F(z) = A_1 \cos Bz + A_2 \sin Bz, \quad (5.34)$$

where A_1 and A_2 are constants.

Again, the right-hand side of Eq. (5.32) may be written as

$$\dot{G}(t) = -B^2 C_v G(t). \quad (5.35)$$

The solution to Eq. (5.35) is given by

$$G(t) = A_3 \exp(-B^2 C_v t), \quad (5.36)$$

where A_3 is a constant. Combining Eqs. (5.28), (5.34), and (5.36),

$$u = (A_1 \cos Bz + A_2 \sin Bz) A_3 \exp(-B^2 C_v t) \text{ and} \quad (5.37)$$

$$u = (A_4 \cos Bz + A_5 \sin Bz) \exp(-B^2 C_v t), \quad (5.38)$$

$$\text{where } A_4 = A_1 A_3, A_5 = A_2 A_3. \quad (5.39)$$

The constants in Eq. (5.38) can be evaluated from the boundary conditions, which are as

1. At time, $t = 0$, $u = u_i$ (initial excess pore water pressure at any depth).
2. $u = 0$ at $z = 0$
3. $u = 0$ at $z = H_t = 2H$

Note that H is the length of the longest drainage path. In this case, which is a two-way drainage condition (top and bottom of the clay layer), H is equal to half the total thickness of the clay layer, H_t . The second boundary condition dictates that $A_4 = 0$, and from the third boundary condition we get

$$A_5 \sin 2BH = 0 \quad \text{or} \quad 2BH = n\pi, \quad (5.40)$$

where n is an integer. From the above, a general solution of the Eq. (5.38) can be in giving the form

$$u = \sum_{n=1}^{n=\infty} A_n \sin \frac{n\pi z}{2H} \exp\left(-\frac{n^2\pi^2 T_v}{4}\right), \quad (5.41)$$

where T_v is the nondimensional time factor and is equal to $C_v t/H^2$.

To satisfy the first boundary condition, we must have the coefficients of A_n such that

$$u = \sum_{n=1}^{n=\infty} A_n \sin \frac{n\pi z}{2H}. \quad (5.42)$$

Eq. (5.42) is a Fourier sine series, and A_n can be given by

$$A_n = \frac{1}{H} \int_0^{2H} u_t \sin \frac{n\pi z}{2H} dz. \quad (5.43)$$

Combining Eqs. (5.41) and (5.43)

$$u = \sum_{n=1}^{n=\infty} \left(\frac{1}{H} \int_0^{2H} u_t \sin \frac{n\pi z}{2H} dz \right) \sin \frac{n\pi z}{2H} \exp\left(-\frac{n^2\pi^2 T_v}{4}\right), \quad (5.44)$$

so far we have not made any assumptions regarding the variation of u_t with the depth of the clay layer. Several possible types of variation for u_t are considered below. Constant u_t with depth. If u_t is constant with depth – i.e., if $u_t = u_o$

$$A_n = \frac{1}{H} \int_0^{2H} u_o \sin \frac{n\pi z}{2H} dz = \frac{2u_o}{n\pi} (1 - \cos n\pi) \text{ and} \quad (5.45)$$

$$u = \sum_{n=1}^{n=\infty} \frac{2u_o}{n\pi} (1 - \cos n\pi) \sin \frac{n\pi z}{2H} \exp\left(-\frac{n^2\pi^2 T_v}{4}\right). \quad (5.46)$$

Note that the term $(1 - \cos n\pi)$ in the above equation is zero for cases when n is even; therefore, u is also zero. For the nonzero terms, it is convenient to substitute $(n = 2m + 1)$ where m is an integer. So Eq. (5.46) will now can be represented as

$$u = \sum_{m=0}^{\infty} \frac{2u_0}{(2m+1)\pi} (1 - \cos(2m+1)\pi) \sin \frac{(2m+1)\pi z}{2H} \exp\left(-\frac{(2m+1)^2 \pi^2 T_v}{4}\right), \quad (5.47)$$

$$\text{or, } u = \sum_{m=0}^{\infty} \frac{2u_0}{M} \sin \frac{Mz}{H} \exp(-M^2 T_v) \quad \text{and} \quad (5.48)$$

$$\text{where, } M = \frac{(2m+1)\pi}{2},$$

$$\Delta Q_2 = \frac{2k_0 A_0 u_0}{H \gamma_w} \sum_{m=0}^{\infty} \cos \frac{Mz}{H} \exp(-M^2 T_v). \quad (5.49)$$

Substitute Eqs. (5.13 and 5.49) in Eq.(5.1), then

$$\Delta Q_T(t) = k(t) * A_0 * A_1 + \left[\frac{2k_0 A_0 u_0}{H \gamma_w} \sum_{m=0}^{\infty} \cos \frac{Mz}{H} \exp(-M^2 T_v) \right]. \quad (5.50)$$

(ii) New method (coupling solution)

In this method, a coupling function of time (t) and elevation (z) can be used to express the excess pore pressure function as

$$u_e(z, t) = \frac{N * \gamma_w * z^{(n+1)}}{k_0 * A_0 * (n+1) * (A+Bt)}. \quad (5.51)$$

From Eq. (5.51), the first and second derivatives of excess pore water pressure with respect to time and depth can be represented respectively as

$$\frac{\partial u}{\partial t} = \frac{-N * \gamma_w * z^{(n+1)} * B}{k_0 * A_0 * (n+1) * (A+Bt)^2} \quad \text{and} \quad (5.52)$$

$$\frac{\partial^2 u}{\partial z^2} = \frac{n * N * \gamma_w * z^{(n-1)}}{k_0 * A_0 * (A+Bt)}. \quad (5.53)$$

Substitute Eqs. (5.52) and (5.53) in Eq. (5.26), then

$$\frac{-N*\gamma_w*z^{(n+1)}*B}{k_o*A_o*(n+1)*(A+Bt)^2} = C_v*\frac{n*N*\gamma_w*z^{(n-1)}}{k_o*A_o*(A+Bt)}. \quad (5.54)$$

To satisfy both sides of Eq. (5.54), then

$$C_v = \frac{-B*z^{(n+1)}}{(n+1)*(A+Bt)*n*z^{(n-1)}}. \quad (5.55)$$

Let $n = -0.5$, then

$$C_v = \frac{4*B*z^2}{(A+Bt)}, \quad (5.56)$$

$$u_e(z, t) = \frac{2*N*\gamma_w*z^{0.5}}{k_o*A_o*(A+Bt)}, \quad (5.57)$$

$$h_e = \frac{u_e(z, t)}{\gamma_w} = \frac{2*N*z^{0.5}}{k_o*A_o*(A+Bt)}, \quad (5.58)$$

$$\frac{\partial h_e}{\partial z} = \frac{N}{k_o*A_o*z^{0.5}*(A+Bt)}, \quad (5.59)$$

$$k(t) = \frac{k_o*A*z^{0.5}}{(A+Bt)}, \quad (5.60)$$

$$\Rightarrow \Delta Q_2(t) = \frac{dV_f}{dt} = \frac{k_o*A*z^{0.5}}{(A+Bt)} * \frac{N}{k_o*A_o*z^{0.5}*(A+Bt)} * A_o, \text{ and} \quad (5.61)$$

$$V_f = \frac{N*t}{(A+Bt)}. \quad (5.62)$$

Substitute Eqs. (5.13 and 5.61) in Eq.(5.1), then

$$\Delta Q_T(t) = k(t) * A_o * \left(A_1 + \frac{N}{k_o*A_o*z^{0.5}*(A+Bt)} \right), \quad (5.63)$$

where $k(t)$: permeability (decreased with the time),

A and B: arbitrary constants,

A_1 : constant at each time step (difference in the head over the distance),

A_0 : cross-sectional area,

Δu : initial excess pore water pressure, and

C_v : coefficient of consolidation.

5.2.3. Model validation

The variation of fluid loss, coefficient of permeability, coefficient of consolidation, and pore water pressure with the time using API-model, Terzaghi, and new method for 2% and 8% bentonite drilling muds under different temperatures have been investigated.

(i) 2% Bentonite drilling mud

The variations of fluid loss, coefficient of permeability, coefficient of consolidation, and pore water pressure with the time under a constant pressure of 100 psi and various temperatures 25°, 50°, 75°, and 100° C for 2% bentonite drilling mud can be shown in Figure 5.4, Figure 5.5, Figure 5.6, and Figure 5.7 respectively. In Figure 5.4 a, all three models predicted the fluid loss very well with higher accuracy for new method ($R^2 = 0.99$) compared to API model and Terzaghi method. In Figure 5.4 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 126000 times over 420 minutes in new method. Similarly, in Figure 5.4 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 126000 times over 420

minutes in new method. In Figure 5.4 d, the pore water pressure decreased by 24% and 26% over 420 minutes in Terzaghi and new method respectively.

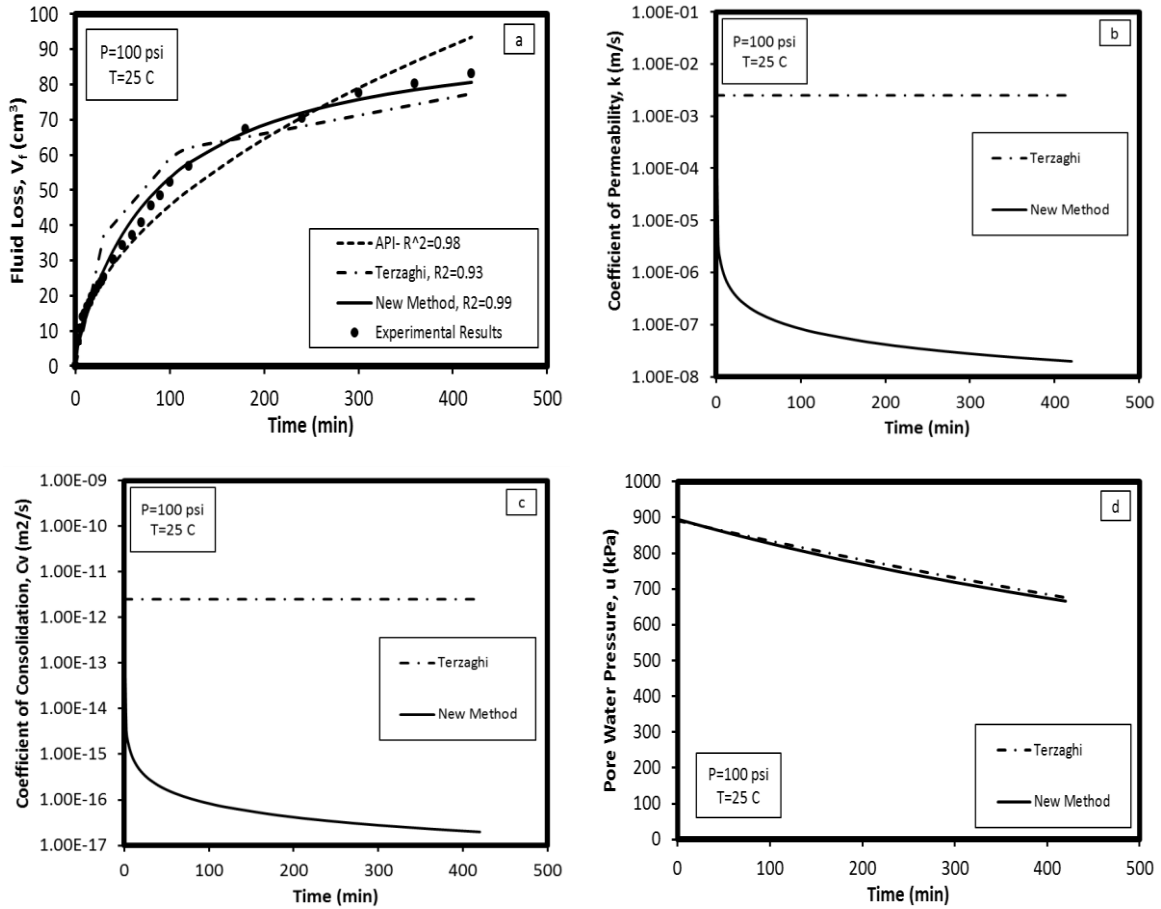


Figure 5.4 Long-term model predictions of current study on 2% bentonite drilling mud at 25°C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

In Figure 5.5 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.5 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 109847 times over 420 minutes in new method. Similarly, in Figure 5.5 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation

decreased by 109847 times over 420 minutes in new method. In Figure 5.5 d, the pore water pressure decreased by 24% and 25% over 420 minutes in Terzaghi and new method respectively.

In Figure 5.6 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.6 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 100801 times over 420 minutes in new method. Similarly, in Figure 5.6 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 100801 times over 420 minutes in new method. In Figure 5.6 d, the pore water pressure decreased by 24% and 25% over 420 minutes in Terzaghi and new method respectively.

In Figure 5.7 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.7 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 100801 times over 420 minutes in new method. Similarly, in Figure 5.7 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 100801 times over 420 minutes in new method. In Figure 5.7 d, the pore water pressure decreased by 24% and 25% over 420 minutes in Terzaghi and new method respectively. All model predictions for 2% bentonite drilling mud at different tested temperatures 25° C, 50° C, 75° C, and 100° C are summarized in Table 5.1. It is clearly shown that the new method is the most accurate model compared to API and

Terzaghi models where new method had the highest R^2 and lowest RMSE compared to others.

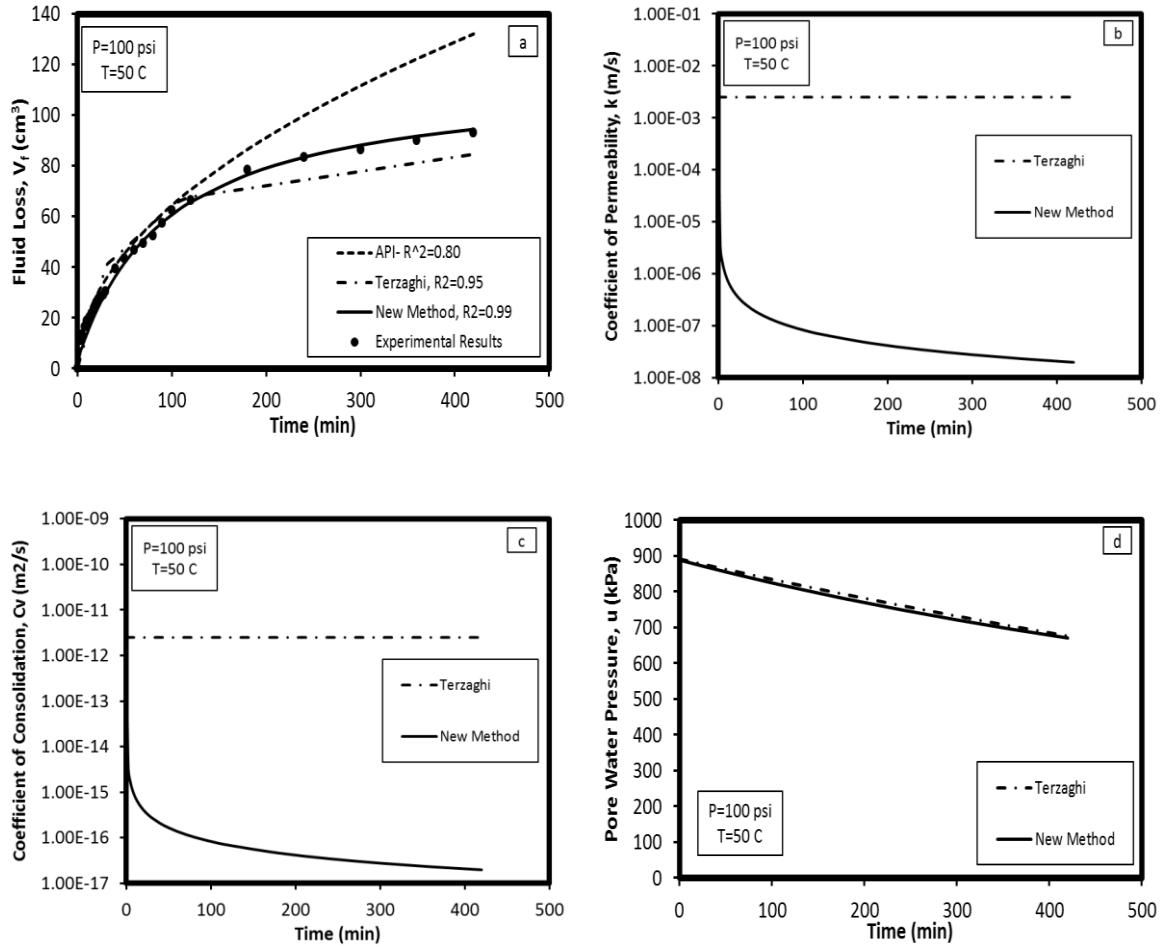


Figure 5.5 Long-term model predictions of current study on 2% bentonite drilling mud at 50°C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

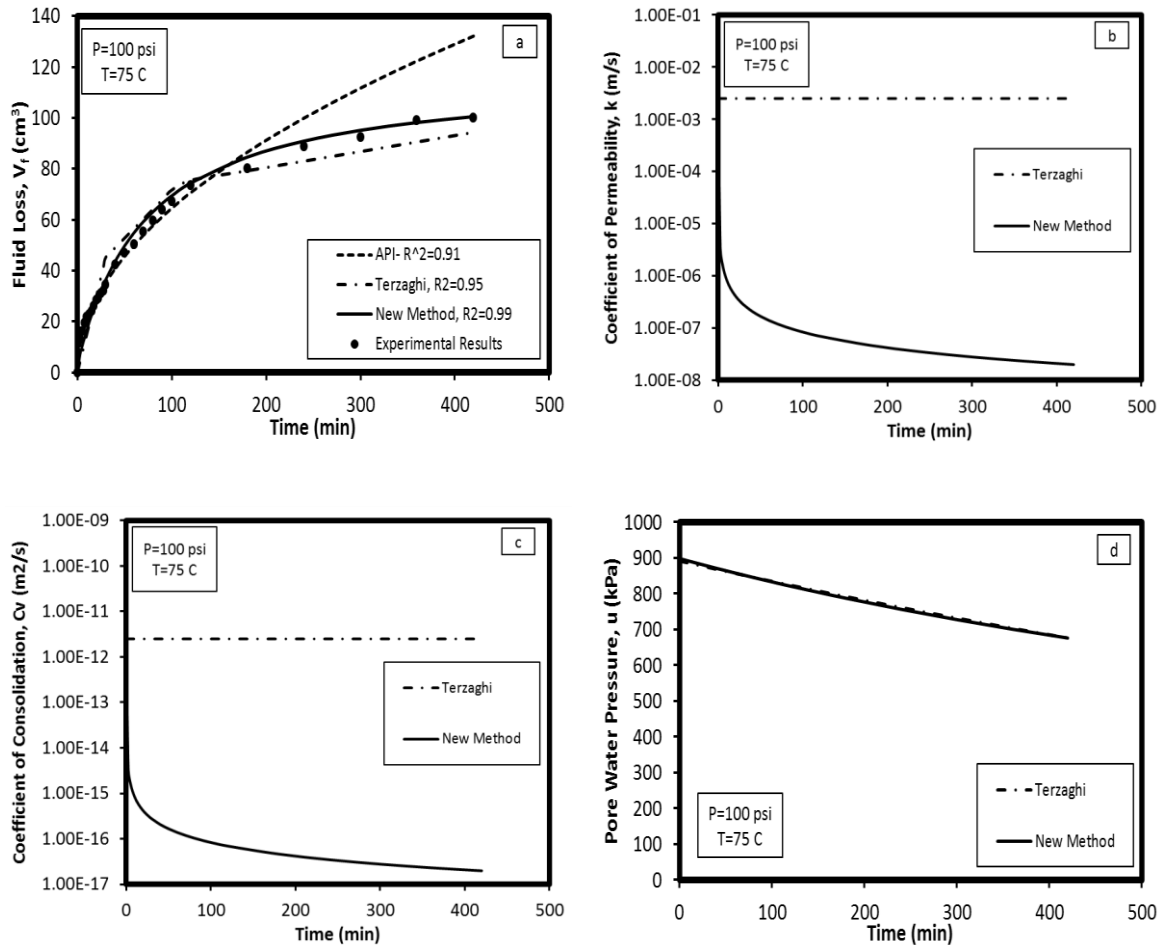


Figure 5.6 Long-term model predictions of current study on 2% bentonite drilling mud at 75° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

Table 5.1 Fluid loss prediction for 2% bentonite drilling mud using three different models.

Drilling Mud (%)	Temp. C	API Model		Terzaghi Model		New method	
		R^2	RMSE (cm ³)	R^2	RMSE (cm ³)	R^2	RMSE (cm ³)
2	25	0.98	3.636	0.93	6.397	0.99	2.216
2	50	0.80	12.003	0.95	5.819	0.99	2.777
2	75	0.91	9.163	0.95	6.035	0.99	3.155
2	100	0.93	8.991	0.94	8.091	0.99	3.764

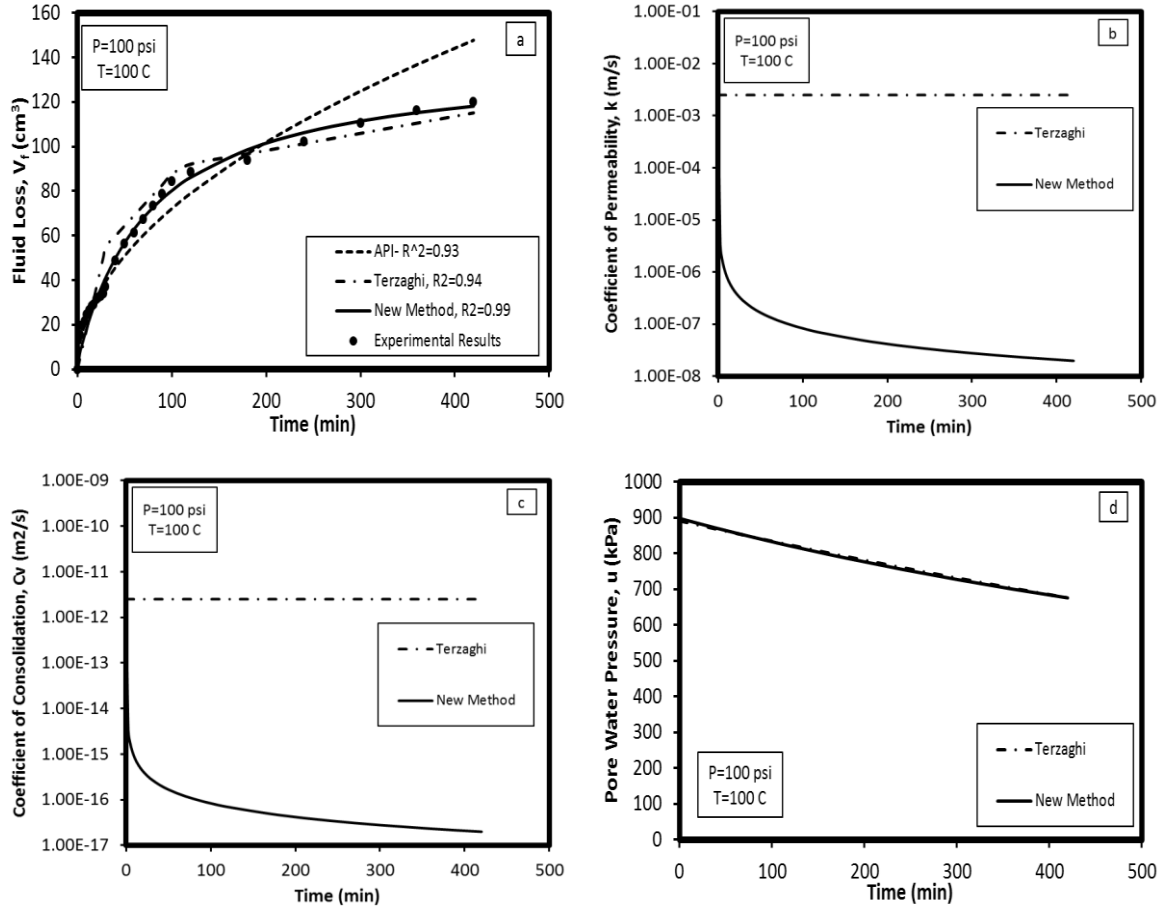


Figure 5.7 Long-term model predictions of current study on 2% bentonite drilling mud at 100° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

(ii) 8% Bentonite drilling mud

The variations of fluid loss, coefficient of permeability, coefficient of consolidation, and pore water pressure with the time under a constant pressure of 100 psi and various temperatures 25°, 50°, 75°, and 100° C for 8% bentonite drilling mud can be shown in Figure 5.8, Figure 5.9, Figure 5.10, and Figure 5.11 respectively. In Figure 5.8 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.8 b, the coefficient of permeability was constant over the time in Terzaghi method while the

coefficient of permeability decreased by 365396 times over 420 minutes in new method. Similarly, in Figure 5.8 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 365396 times over 420 minutes in both new method. In Figure 5.8 d, the pore water pressure decreased by 24% and 23% over 420 minutes in Terzaghi and new method respectively.

In Figure 5.9 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.9 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 255523 times over 420 minutes in new method. Similarly, in Figure 5.9 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 255523 times over 420 minutes in new method respectively. In Figure 5.9 d, the pore water pressure decreased by 24% and 22% over 420 minutes in Terzaghi and new method respectively.

In Figure 5.10 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.10 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 235740 times over 420 minutes in new method. Similarly, in Figure 5.10 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 235740 times over 420 minutes in new method. In Figure 5.10 d, the pore water pressure decreased by 24% and 22% over 420 minutes in Terzaghi and new method respectively.

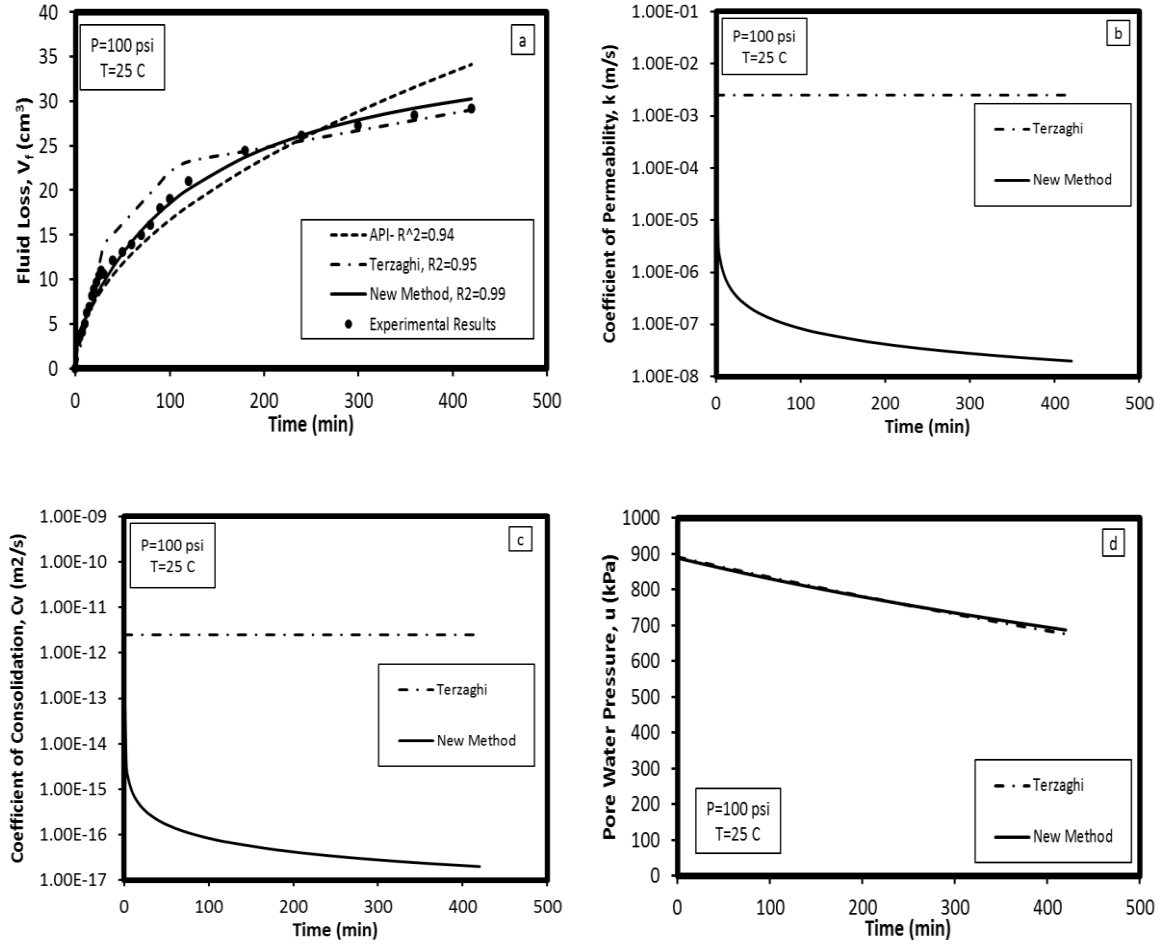


Figure 5.8 Long-term model predictions of current study on 8% bentonite drilling mud at 25° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

In Figure 5.11 a, new method predicted the fluid loss very well with a coefficient of correlation (R^2) in 0.99 where Terzaghi predication was better than the API - model. In Figure 5.11 b, the coefficient of permeability was constant over the time in Terzaghi method while the coefficient of permeability decreased by 202999 times over 420 minutes in new method. Similarly, in Figure 5.11 c, the coefficient of consolidation was constant over the time in Terzaghi method while the coefficient of consolidation decreased by 202999 times over 420 minutes in new method. In Figure 5.11 d, the pore

water pressure decreased by 24% and 23% over 420 minutes in Terzaghi and new method respectively. All model predictions for 8% bentonite drilling mud at different tested temperatures 25° C, 50° C, 75° C, and 100° C are summarized in Table 5.2. It is clearly shown that the new method is the most accurate model compared to API and Terzaghi models where new method had the highest R^2 and lowest RMSE compared to others.

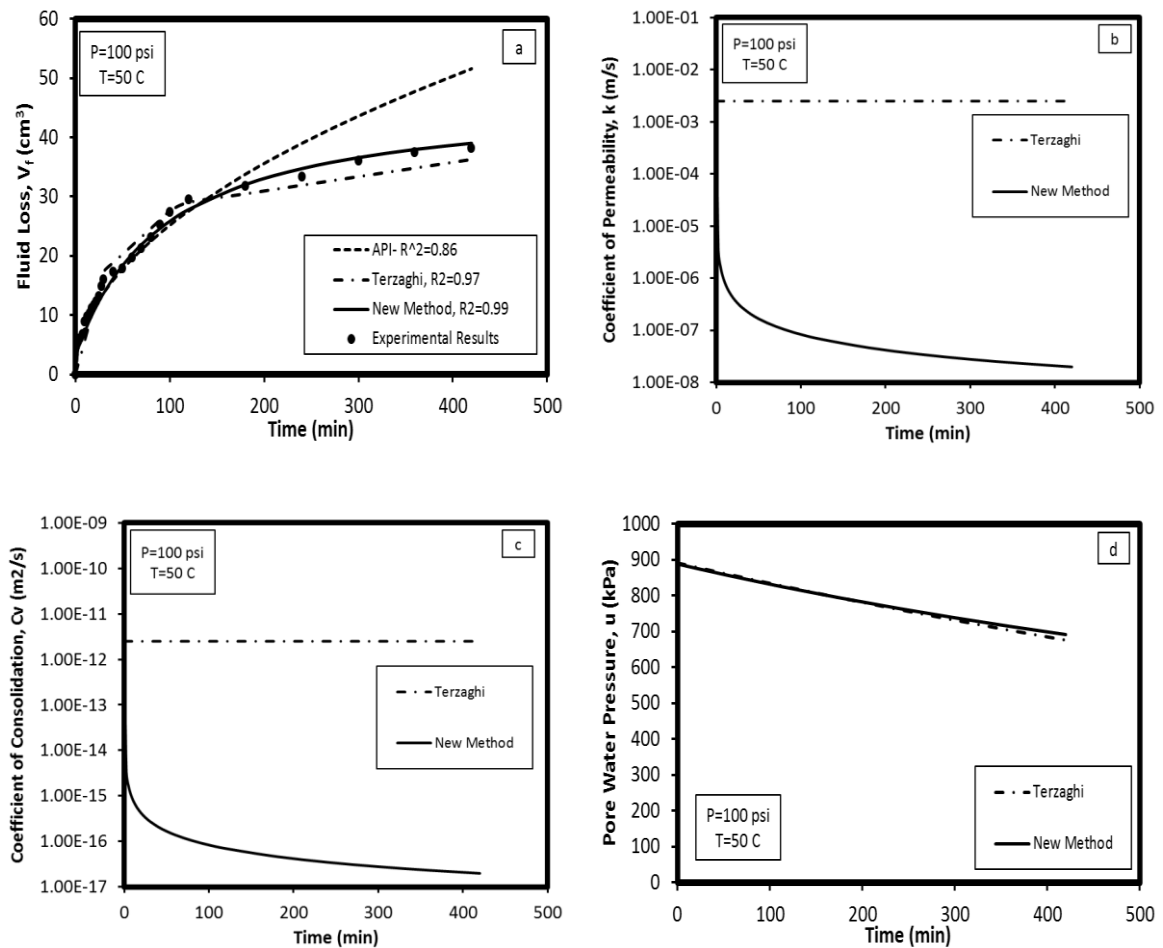


Figure 5.9 Long-term model predictions of current study on 8% bentonite drilling mud at 50° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

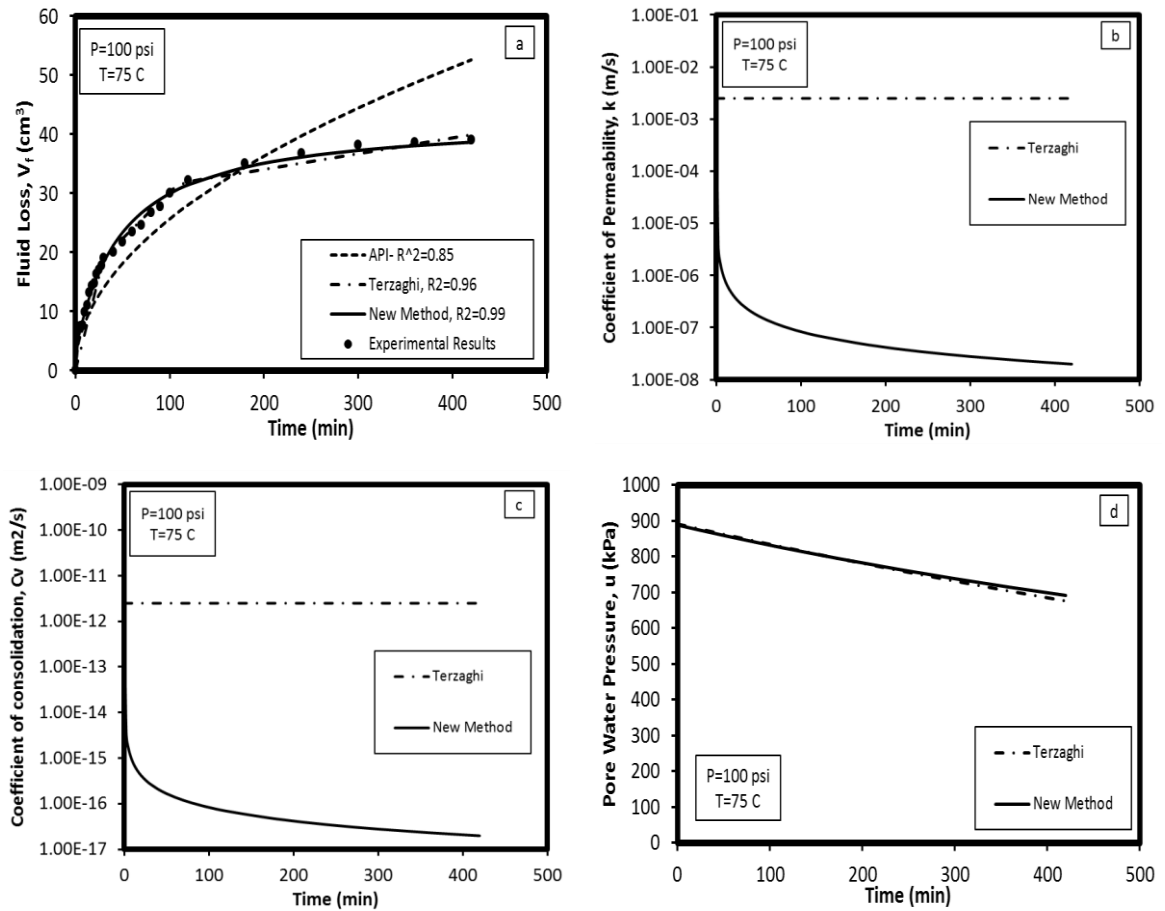


Figure 5.10 Long-term model predictions of current study on 8% bentonite drilling mud at 75° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

Table 5.2 Fluid loss prediction for 8% bentonite drilling mud using three different models.

Drilling Mud (%)	Temp. C	API Model		Terzaghi Model		New method	
		R^2	RMSE (cm ³)	R^2	RMSE (cm ³)	R^2	RMSE (cm ³)
8	25	0.94	1.820	0.95	2.005	0.99	0.828
8	50	0.86	3.958	0.97	1.935	0.99	1.096
8	75	0.85	4.717	0.96	2.139	0.99	0.932
8	100	0.65	7.266	0.93	3.201	0.99	0.803

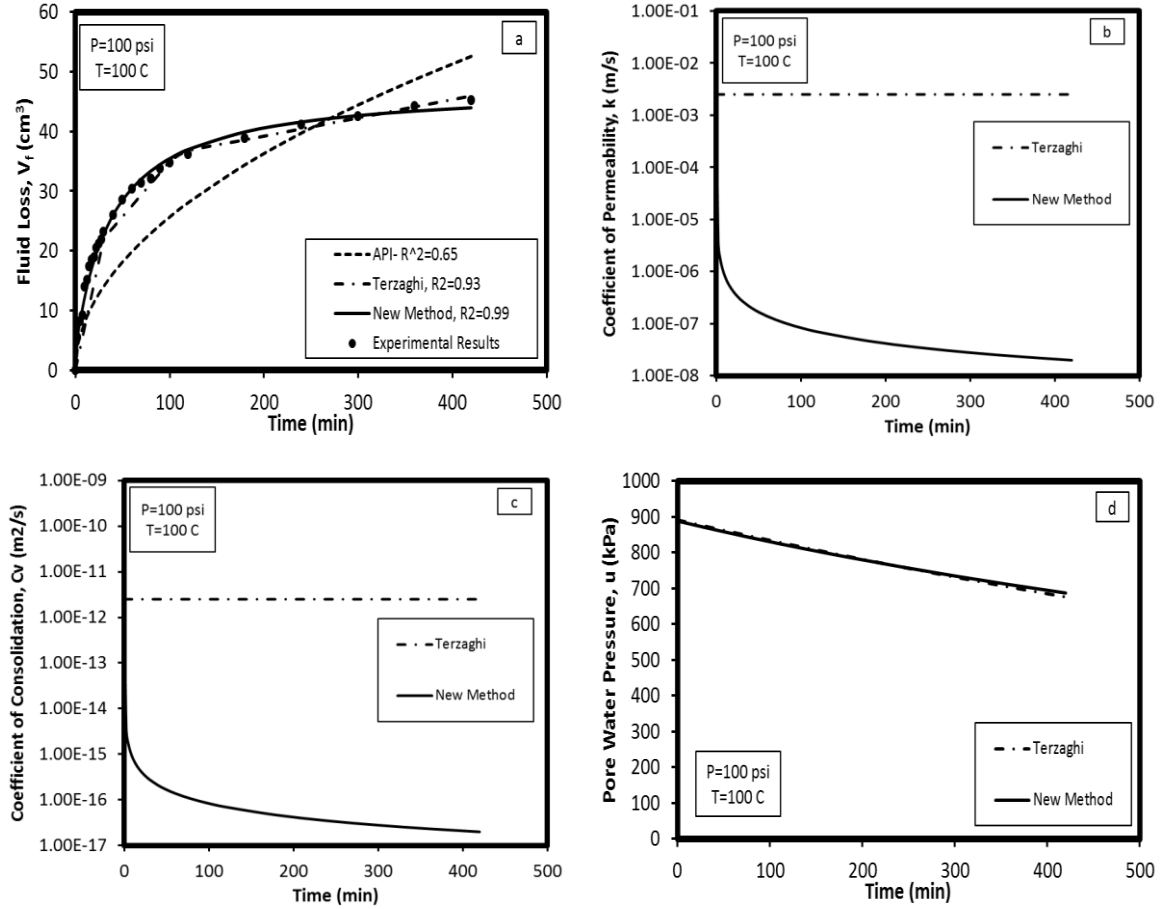


Figure 5.11 Long-term model predictions of current study on 8% bentonite drilling mud at 100° C (a) fluid loss versus time, (b) variation of permeability with time, (c) variation of coefficient of consolidation with time, and (d) variation of pore water pressure with time.

5.3.Filtrating process

Filtration process may occur under static and/or dynamic conditions. In static filtration process, the particles are continuously deposited to form thicker filter cakes until the space available is full of the filter cake. Schematic filter cake formation process is shown in Figure 5.12.

During the filtration process, number changes can occur within the filter cake and affect the fluid loss. The modeling of this phenomenon can be idealized as shown in Figure 5.13.

5.4. Modeling of filtering process

5.4.1. General model

The flow of fluid through the filter cake is governed by the Darcy law (Grace 1953).

Hence, the incremental fluid loss (dV_f) during a time period dt is

$$\frac{dV_f}{dt} = \frac{k(t)A_o\Delta p}{\mu(T)h_{mc}(t)} > 0, \quad (5.64)$$

where $k(t)$ is the permeability of the filter cake, A_o is the filter area, Δp is the applied pressure, $\mu(T)$ is the fluid viscosity, $h_{mc}(t)$ is thickness of the filter cake.

At any time during the filtration process, the volume content of solids in the mud is f_{sm} in the volume of drilling mud that was filtrated is V_m . The volume content of solids deposited in the filter cake is f_{sc} and equating the relevant volume of solid the relationship is as follows:

$$f_{sm}dV_m = f_{sc}(t)dh_{mc}A_o, \quad (5.65)$$

$$f_{sm}(dh_{mc}A + dV_f) = f_{sc}(t)dh_{mc}A_o, \text{ and} \quad (5.66)$$

$$dh_{mc} = \frac{f_{sm}dV_f}{A_o(f_{sc}(t)-f_{sm})}. \text{ Hence } h_{mc} = \frac{V_f}{A_o\left(\frac{f_{sc}}{f_{sm}}(t)-1\right)}. \quad (5.67)$$

Substitute Eq. (5.67) in Eq. (5.64);

$$\int_0^{V_f} V_f dV_f = \int_0^t \frac{k(t)A_o\Delta p}{\mu(T)} A_o \left(\frac{f_{sc}}{f_{sm}}(t) - 1 \right) dt. \quad (5.68)$$

5.4.2. Model 1: Static model (API model)

This model was developed based on the following idealization of the conditions

1. The percentage of volume solid content in the cake (f_{sc}) is a constant.

2. The permeability of the cake (k) is constant.
3. Viscosity is a constant at the testing temperature.

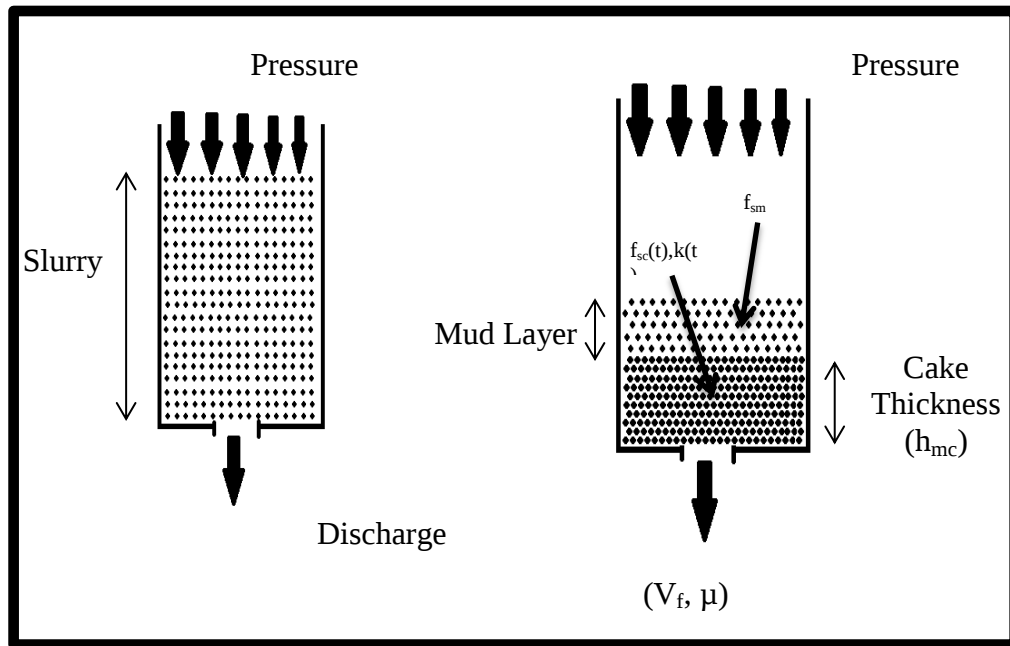


Figure 5.12 Schematic of fluid loss test and filter cake formation.

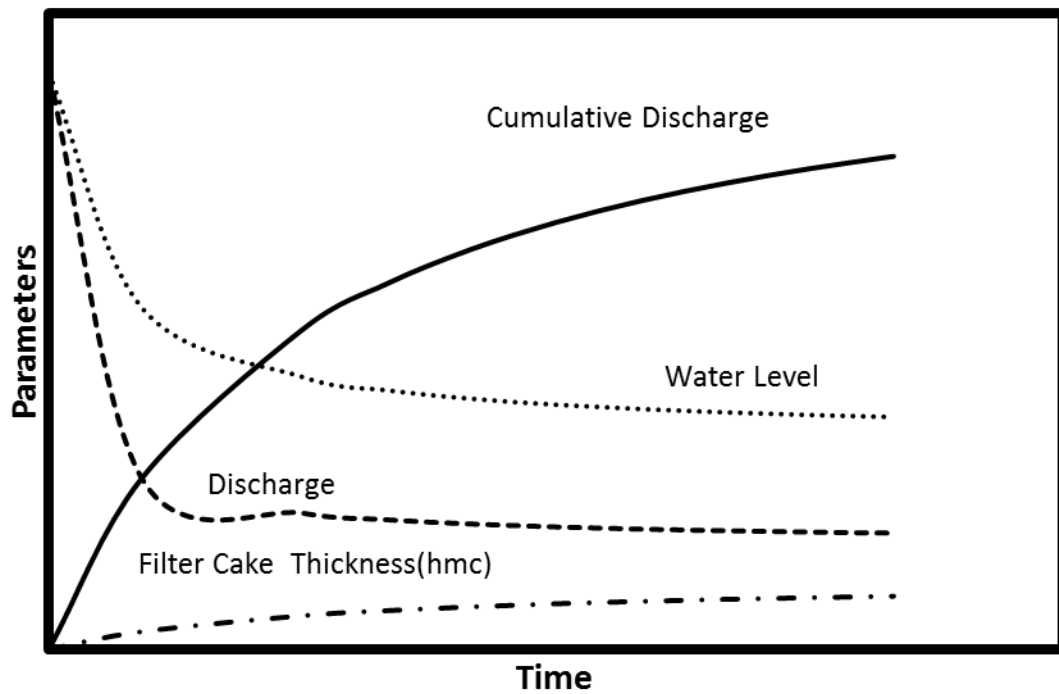


Figure 5.13 Idealization of Filter Cake Formation.

Applying these conditions to Eq. (5.68) and performing the integration with the initial conditions will lead to the following relationship

$$V_f - V_0 = \sqrt{2k\Delta p \left(\frac{f_{sc}}{f_{sm}} - 1 \right)} A_o \frac{\sqrt{t}}{\sqrt{\mu}}. \quad (5.69)$$

Equation (5.69) can be re-written as follows

$$V_f - V_0 = M * \sqrt{t}, \quad (5.70)$$

where

$$M = \sqrt{\frac{2k\Delta p \left(\frac{f_{sc}}{f_{sm}} - 1 \right)}{\mu}} A_o. \quad (5.71)$$

V_f = total volume of fluid loss (cm^3), V_0 = initial volume of fluid loss (spurt) (cm^3), k = permeability of the filter cake (darcy), Δp = applied pressure (atm), f_{sc} = volume fraction of solid in cake, f_{sm} = volume fraction of solids in mud, V_m = volume of solids in mud, A_o = filter area (cm^2), t = time (min), μ = filtering fluid viscosity (cP), h_{mc} = the thickness of the filter (mud) cake (cm).

5.4.3. Model 2: New kinetic (hyperbolic) model

(i) Model Development

Fluid Loss Model

Inspection of the filtration phenomena shows that the rate of infiltration is dependent on the permeability of the filter cake and the ratio of the solid content in the cake to the solid content in the mud and the time. Hence, it is represented as

$$\frac{dV}{dt} = f\left(k(t), \frac{f_{sc}}{f_{sm}}(t)\right), \quad (5.72)$$

with the following assumptions of cake properties variation:

1- The permeability is represented as

$$k(t) = \frac{2A \cdot k_o}{(A+Bt)^{3-p}}. \quad (5.73)$$

2- The ratio of the solid content in the cake to the solid content in the mud as a volume fraction is a function of time

$$\left(\frac{f_{sc}(t)}{f_{sm}} - 1 \right) = \frac{\alpha_o t}{(A+Bt)^p} > 1.0, \quad (5.74)$$

3- The final form of the filtration versus time is taking the form of hyperbolic function after substituting Eqs. (5.73) and (5.74) in equation (5.68) then performing the integration of the equation after applying the initial conditions

$$V_f - V_o = N * \frac{t}{A+Bt}, \quad (5.75)$$

where A: Fluid loss non-dimensional parameter represents the initial rate of fluid loss which depends on pressure and temperature, B: Fluid loss parameter influencing the ultimate fluid loss (1/min), α_o : Parameter related to the solid content in the filter cake (1/min), k_o : Parameter related to the filter cake permeability (darcy), p: Parameter related to the filter cake thickness, A_o : filter area (cm²), V_o : Initial spate of fluid loss (cm³), and

$$N = \sqrt{\frac{2 \cdot k_o \cdot \alpha_o \cdot \Delta p}{\mu(T)}} * A_o. \quad (5.76)$$

Hence, fluid loss parameter M in Model 1 (Eq. (5.70)) is similar to the fluid loss parameter N (α_o , k_o) in Model 2 (Eq. (5.75)).

Filter Cake Model

Porosity of the filter cake can be represented as

$$n = \frac{V_{void}}{V_{total}} = \frac{V - V_s}{V} = 1 - f_{sc} = 1 - f_{sm} (1 + \alpha_o t / (A + Bt)^p). \quad (5.77)$$

Furthermore, Eqn. (5.77) satisfies the following conditions

$$1- \frac{dV}{dt} > 0 \text{ and} \quad (5.78)$$

$$2- \frac{d^2V}{dt^2} < 0. \quad (5.79)$$

3- V_f has a limiting value ($V_o + N/B$). No more fluid loss could happen due to the condition of the consolidated filter cake and testing parameters.

4- Both A and B are functions of pressure and temperature.

None of the above properties can be checked using the API model. However, all the properties can be determined for the proposed new kinetic model. Also, the new model can be used for short and long term fluid loss-time relationship.

(ii) Optimization of kinetic model parameters

For the kinetic (hyperbolic) model, there are five parameters that must be optimized. These five parameters are A, B, α_o , k_o and p. Hence, at least five independent relationships are needed to determine the parameters. Based on the fluid loss experiment, it is possible to obtain normalized parameters $A/ N(\alpha_o, k_o)$ and $B/ N(\alpha_o, k_o)$. Also, the initial condition of permeability and solid content will be related to Model 1 (API Model) to determine the parameters by assuming parameter $p = 1$.

The initial permeability of the filter cake is k_{ini} at time = 0 (equal to the permeability k in Model 1) is represented as

$$k_{ini} = \frac{2*k_o}{A^{2-p}}. \quad (5.80)$$

Hence, parameters k_o and A can be related to the k_{int} using Eq. (5.80) as shown in Figure 5.14 (assuming parameter $p = 1$).

In order to optimize the parameter α_o and parameter B, Eq. (5.74) was used at time = ∞ and $p = 1$

$$\left(\frac{f_{sc}}{f_{sm}} - 1\right)_{t=\infty} = \frac{\alpha_o}{B}, \quad (5.81)$$

where $(f_{sc}/f_{sm}-1)_{t=\infty}$ was assumed to be equal $(f_{sc}/f_{sm}-1)_{API}$ (Model 1) and hence the relationship was modified as

$$\left(\frac{f_{sc}}{f_{sm}} - 1\right)_{API} = \frac{\alpha_o}{B}. \quad (5.82)$$

Hence, parameters α_o and B can be related to the $(f_{sc}/f_{sm}-1)_{API}$ using Eq. (5.82) as shown in Figure 5.15. A comparison of model parameters between API and new kinetic models are summarized in Table 5.3.

(iii) Variation of kinetic parameters with temperature and pressure

Based on the data from the literature and the experimental results from the current study, nonlinear power relationship (Demircan et al. 2011) for the kinematic model parameters (A, B, and α_o) with bentonite content, temperature and pressure were represented as

$$\text{Parameter (A, B, } \alpha_o) = L * (\text{Bent } (\%))^v * (T(C))^q * (P(\text{psi}))^m, \quad (5.83)$$

where: L, v, q, and m are model parameters.

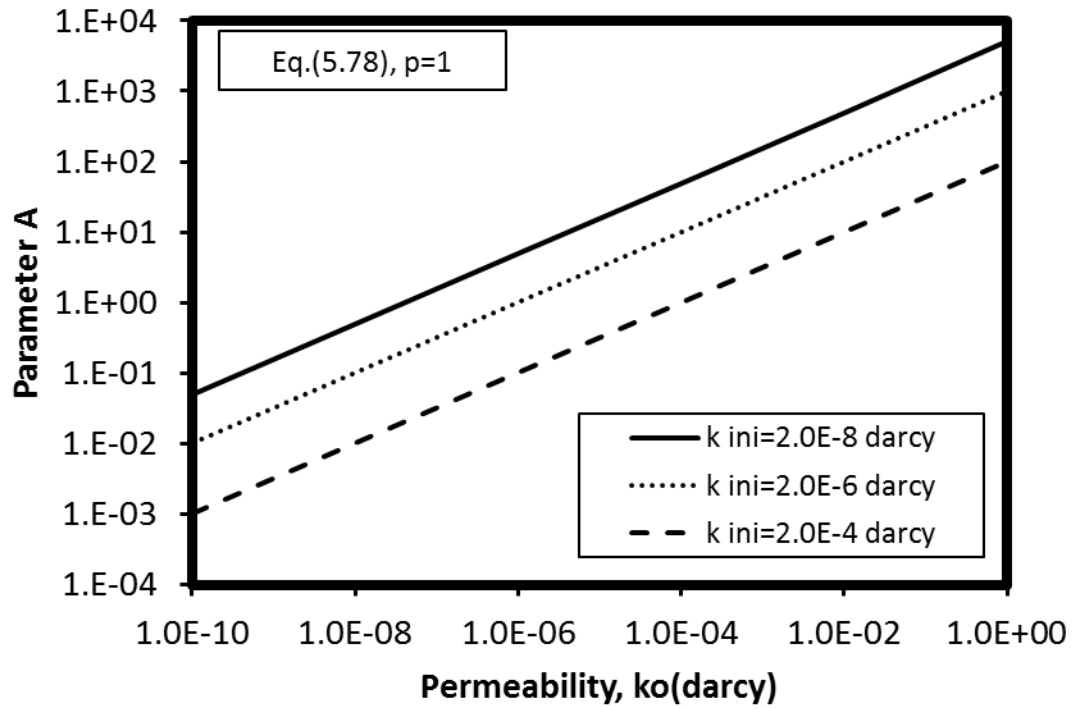


Figure 5.14 Variation of parameter A and parameter k_o with the initial permeability.

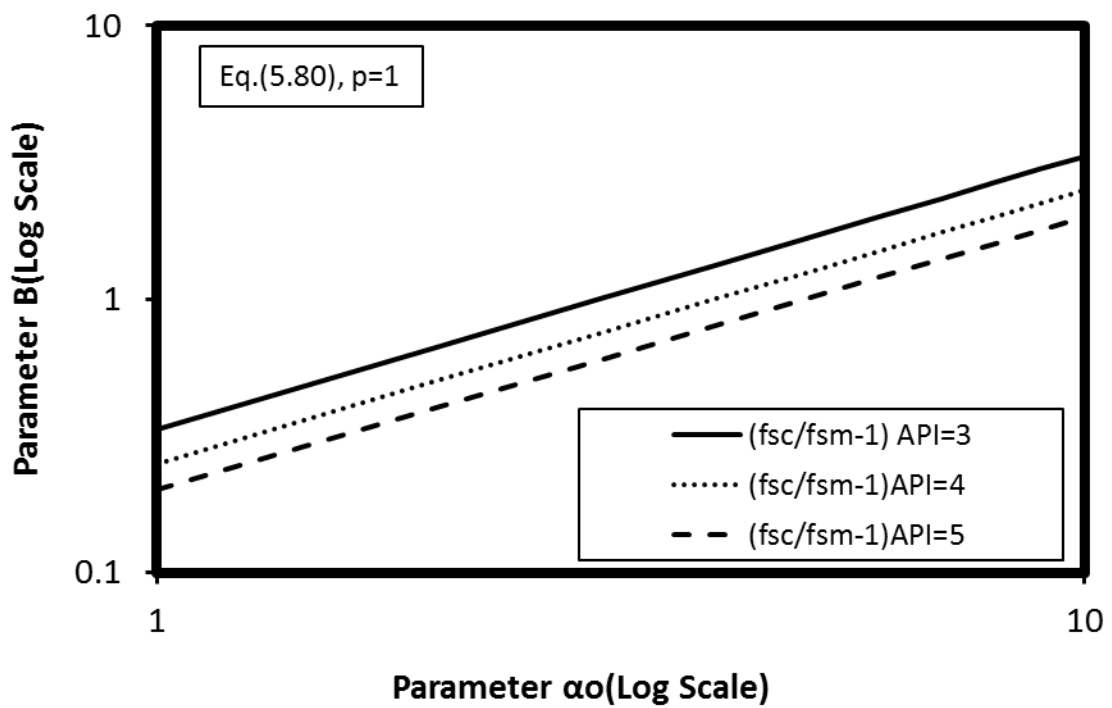


Figure 5.15 Parameter B with parameter α to maintain the API $(f_{sc}/f_{sm}-1)$ value.

Table 5.3 Summary of model parameters

Parameter	API-model	New kinetic model	Remarks
Fluid loss model			
Fluid loss (V_f)	$V_o + M\sqrt{t}$	Hyperbolic model $V_o + Nt/(A + Bt)$	1. New model limits the maximum fluid loss. 2. Model parameters A and B in the new model are temperature and pressure dependents.
Filter cake model			
Porosity ($1-f_{sc}$)	Constant	Vary with time $1 - f_{sm} \left(1 + \frac{\alpha t}{(A + Bt)^p} \right)$	1. Porosity in the new model is time, temperature, and pressure dependents. 2. New model limits the minimum cake porosity at the end of the fluid loss.
Permeability (k)	Constant	Vary with time $\frac{2A * k_o}{(A + Bt)^{3-p}}$	1. Permeability in the new model is time, temperature, and pressure dependents. 2. New model limits the minimum cake permeability at the end of the fluid loss.
Relative solid content $\left(\frac{f_{sc}}{f_{sm}} - 1 \right)$	Constant $\frac{M^2}{A_o^2} * \frac{\mu}{2k * \Delta p}$	Vary with time $\frac{\alpha_o t}{(A + Bt)^p}$	1. Relative solid content in the new model is time, temperature, and pressure dependents. 2. New model limits the maximum relative solid content at the end of the fluid loss.
Cake thickness (h_{mc})	Vary with time $\frac{V_o + M\sqrt{t}}{A_o \left(\frac{f_{sc}}{f_m} - 1 \right)}$	Vary with time $\frac{V_o + Nt/(A + Bt)}{\alpha_o t/(A + Bt)^p}$	Both models predict the increase in cake thickness with time.

From the nonlinear regression analysis of the data (total of 19), the relationships were developed as

$$A = 6.3 * (\text{Bent } (\%))^{-0.685} * (T(C))^{-0.108} * (P(\text{psi}))^{0.036}, \quad (R^2=0.88), \quad (5.84)$$

$$B\left(\frac{1}{\text{min}}\right) = 0.148 * (\text{Bent } (\%))^{-0.213} * (T(C))^{-0.297} * (P(\text{psi}))^{0.302}, \quad (R^2=0.91), \text{ and} \quad (5.85)$$

$$\alpha_o\left(\frac{1}{\text{min}}\right) = 0.554 * (\text{Bent } (\%))^{-0.221} * (T(C))^{-0.259} * (P(\text{psi}))^{0.332}, \quad (R^2=0.90), \quad (5.86)$$

where R^2 is the coefficient of determination for the relationship. In Figure 5.16, Figure 5.17, and Figure 5.18 the nonlinear relationships for parameters A, B and α_o (Eqs. 5.84 to 5.86) are compared with the experimental results at 100° C.

5.5.Results and analysis

In order to verify the prediction of the new Kinetic Hyperbolic model with the API model, experimental results were divided into two categories: Category 1 -short term results (fluid loss time ($t \leq 30$ min)) and Category 2- long term results ($t > 30$ min). The porosity of the filter cake can be represented as

$$n = \frac{w}{w + \frac{1}{G_s}}, \quad (5.87)$$

where w is the moisture content of the filter cake and G_s is the specific gravity of the solid particles. Also, a popular permeability-porosity relationship was used for comparison. Khatib (1994) studied the effect of applied pressure, solids type, and oil presence on the porosity and permeability of thin cakes using compression-permeability cell. The solids investigated were iron sulfide, iron hydroxide, calcium sulfate, calcium carbonate and produced silt and clay. Based on the study, a correlation between permeability and porosity of silt/clay filter cake was developed and the relationship is as

$$K_c = 0.491 * (1 - n)^{-1.97}, \quad (5.88)$$

where K_c is the permeability (darcy) of the mud cake.

5.5.1.Short-term tests ($t \leq 30$ min)

Several kinetic relationships including the fluid loss versus time, the variation in the cake porosity, variation in the cake permeability with the time, variation in the solid contents in the cake to the mud ratio with the time, and the variation of the cake thickness with time were analyzed for each experimental test results comparing the API with the proposed new kinetic Hyperbolic Model.

(i) Case 1

A drilling mud of 2% bentonite content was prepared and tested in high pressure and high temperature cell under 100 psi and 100° C for 30 minutes. As shown in Figure 5.19, the fluid loss after 7.5 min and 30 min for 2% bentonite under 100 psi and 100° C were 21.4 cm³, and 37.1 cm³ respectively.

API-Model

Fluid Loss(V_t)

API model predicted the fluid loss very well with R^2 and parameter M of 0.99 and 4.56 cm³/min^{0.5} respectively as shown in Figure 5.19(a). The spurt fluid loss was 10 cm³. The predicted fluid loss for API model at 30 min was 35 cm³ (94% of experimental value). API model predicted the maximum fluid loss to be infinity.

Porosity (n)

Based on the moisture content (w) of the filter cake at the end of the test, the porosity using Eq. (5.87) was determined to be 94% and the solid content by volume (f_{sc}) in the cake was 6% (Eq. 5.77). Since the filter cake porosity in the API model is assumed to be a constant with time, the variation is shown in Figure 5.19(b).

Permeability

The API model does not predict the changes in the cake permeability (k) with time. The used viscosity of the water at 100° C was 0.28 cp. Based on filter cake solid content (f_{sc}), viscosity (μ), parameter M and using Eq. (5.71), the API cake permeability was determined and it was constant with a value of 0.011 Darcy as shown in Figure 5.19(c).

Relative Solid Content (f_{sc}/f_{sm})

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the change in the solid content in filter cake (f_{sc}) to the solid content in the mud (f_{sm}) with time. At the end of the test, the ratio of the solid content in the cake to the solid content in the mud was the same as the API model prediction as shown in Figure 5.19(d).

Cake Thickness (h_{mc})

Using Eq. (5.67), the cake thickness (h_{mc}) was predicted to be 10 mm after 30 minutes as shown in Figure 5.19 (e).

New Kinetic Model

Fluid Loss (V_f)

New kinetic model predicted the fluid loss very well with R^2 of 0.99. For the best model prediction, the parameters N , A , and B were $4.56 \text{ cm}^3/\text{min}$, 2 , and $0.11/\text{min}$ respectively as shown in Figure 5.19 (a). Using the nonlinear model (Eqs. 5.84 and 5.85), A and B were 1.8 and $0.1/\text{min}$ respectively. The predicted fluid loss for new kinetic model at 30 min was 35.8 cm^3 (96.5% of experimental value). The maximum predicted fluid loss by the new kinetic model ($V_o + N/B$) was 51.5 cm^3 , which was equivalent to 14% of the total volume of the drilling mud.

Porosity (n)

Based on the solid content in cake (f_{sc}) at the end of the test, the parameter α_o is determined using Eq. (5.74). Then, the variation of porosity with time predicted using Eq. (5.77) as shown in Figure 5.19 (b). After 30 minutes, the predicted porosity using Eq. (5.77) was 94%. This also agreed with the porosity obtained from experimental test.

Permeability (k)

The variation of permeability with the time in the kinetic model (Eq. (5.73)) was predicted using $k_o = 0.02 \text{ Darcy}$, $A = 2$, $B = 0.11/\text{min}$ and $p = 1$ as shown in Figure 5.19 (c). Parameter k_o was obtained using Eq. (5.76) with $\alpha_o = 0.87/\text{min}$, $\mu = 0.28 \text{ cp}$ and $N = 4.56 \text{ cm}^3/\text{min}$. The final cake permeability was 0.0015 Darcy 7 times smaller than the permeability predicted by the API model. Using the porosity with the Khatib (1994) permeability relationship (Eq. (5.88)), it over predicted the cake permeability.

Relative Solid Content (f_{sc}/f_{sm})

The variation of the solid content in the cake (f_{sc}) to the mud (f_{sm}) with the time was determined using Eq. (5.74) with $\alpha_o=0.87/\text{min}$ as shown in Figure 5.19 (d). Using the nonlinear model (Eq. 5.86), α_o was 0.75/min. The ratio of solid content in the cake to the solid content in the mud at the end of the test was 100% of the kinetic model prediction.

Cake Thickness (h_{mc})

The cake thickness variation with the time was estimated reasonably well by the new kinetic model (assuming the parameters $A=2$, $B=0.11/\text{min}$, and $p=1$) using Eq. (5.67) as shown in Figure 5.19 (e). The final cake thickness predicted by the new kinetic model was 61% of the cake thickness predicted by API model.

(ii) Case 2

A drilling mud with 8% bentonite content was prepared and tested under 100 psi pressure at 100° C for 30 minutes. As shown in Figure 5.20, the fluid loss after 7.5 min and 30 min for the 8% bentonite under 100 psi and 100° C were 9.2 cm³, and 23.1 cm³ respectively.

API-Model

Fluid Loss (V_f)

API model predicted the fluid loss very well with R^2 and parameter M were 0.96 and 3.53 cm³/min^{0.5} respectively as shown in Figure 5.20 (a). The spurt fluid loss was 3 cm³.

The predicted fluid loss using the API model after 30 min was 22.4 cm³ (97% of experimental value). API model predicted the maximum fluid loss to be infinity.

Porosity (n)

Based on the moisture content (w) of the filter cake at the end of the test, the porosity using Eq. (5.87) was determined to be 86% and the solid content by volume (f_{sc}) in the cake was 14% (Eq. 5.77). Since the filter cake porosity in the API model is assumed to be a constant with time, the variation is shown in Figure 5.20 (b).

Permeability (k)

The API model does not predict the changes in the cake permeability (k) with time. The used viscosity of the water at 100° C was 0.28 cp. Based on filter cake solid content (f_{sc}), viscosity (μ), parameter M and using Eq. (5.71), the API cake permeability was determined and it was constant with a value of 0.005 Darcy as shown in Figure 5.20 (c).

Relative Solid Content (f_{sc}/f_{sm})

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the change in the solid content in filter cake (f_{sc}) to the solid content in the mud (f_{sm}) with time. At the end of the test, the ratio of the solid content in the cake to the solid content in the mud was the same as the API model prediction as shown in Figure 5.20 (c).

Cake Thickness (h_{mc})

Using Eq. (5.67), the cake thickness (h_{mc}) was predicted to be 6 mm after 30 minutes as shown in Figure 5.20 (e).

New Kinetic Model

Fluid Loss (V_f)

New kinetic model predicted the fluid loss very well with R^2 of 0.96. For the best model prediction, the parameters N , A , and B were $0.82\text{cm}^3/\text{min}$, 0.45 and $0.027/\text{min}$ respectively as shown in Figure 5.20 (a). Using the nonlinear model (Eqs. 5.84 and 5.85), A and B were 0.42 and $0.023/\text{min}$ respectively. The predicted fluid loss for new kinetic model at 30 min was 22.6 cm^3 (98% of experimental value). The maximum predicted fluid loss by the new kinetic model (V_o+N/B) was 33.4 cm^3 , which was equivalent to 9% of the total volume of the drilling mud.

Porosity (n)

Based on the solid content in cake (f_{sc}) at the end of the test, the parameter α_o is determined using Eq. (5.74). Then, the variation of porosity with time predicted using Eq. (5.77) as shown in Figure 5.20 (b). After 30 minutes, the predicted porosity using Eq. (5.77) was 86%. This also agreed with the porosity obtained from experimental test.

Permeability (k)

The variation of permeability with the time in the kinetic model (Eq. (5.73)) was predicted using $k_o=0.0065\text{ Darcy}$, $A=0.45$, $B=0.027/\text{min}$ and $p=1$ as shown in Figure 5.20 (c). Parameter k_o was obtained using Eq. (5.76) with $\alpha_o=0.15/\text{min}$, $\mu=28\text{ cp}$ and $N=0.82\text{ cm}^3/\text{min}$. The final cake permeability was 0.0013 Darcy 4 times smaller than the permeability predicted by the API model. Using the porosity with the Khatib (1994) permeability relationship (Eq. (5.88)), it under predicted the cake permeability.

Relative Solid Content (f_{sc}/f_{sm})

The variation of the solid content in the cake (f_{sc}) to the mud (f_{sm}) with the time was determined using Eq. (5.74) with $\alpha_o=0.15/\text{min}$ as shown in Figure 5.20 (d). Using the nonlinear model (Eq. 5.76), α_o was 0.2/min. The ratio of solid content in the cake to the solid content in the mud at the end of the test was 103% of the kinetic model prediction.

Cake Thickness (h_{mc})

The cake thickness variation with the time was estimated reasonably well by the new kinetic model (assuming the parameters $A=0.45$, $B=0.027/\text{min}$, and $p =1$) using Eq. (5.67) as shown in Figure 5.20 (e). The final cake thickness predicted by the new kinetic model was 62% of the cake thickness predicted by API model.

When the bentonite concentration increased from 2% to 8% in the drilling mud, the fluid loss decreased by 57% after 7.5 min and 61% after 30 min at a temperature of 100° C. When the bentonite concentration increased from 2% to 8%, the fluid loss was less at the same time period and under the same experimental conditions.

(iii) Case 3

Both models were used to predict the fluid loss, the cake porosity, the changes of cake permeability, the ratio of the solid content in the cake to the mud, and the cake thickness with the time for the HPHT data in the literature as shown in Figure 5.21 (Elkatatny et al., 2012). The test was performed in HPHT cell under 300 psi and at 225° F. As shown in Figure 5.21, the fluid loss after 7.5 min and 30 min for 5% bentonite under 300 psi and at 225 °F were 6 cm³, and 8 cm³ respectively.

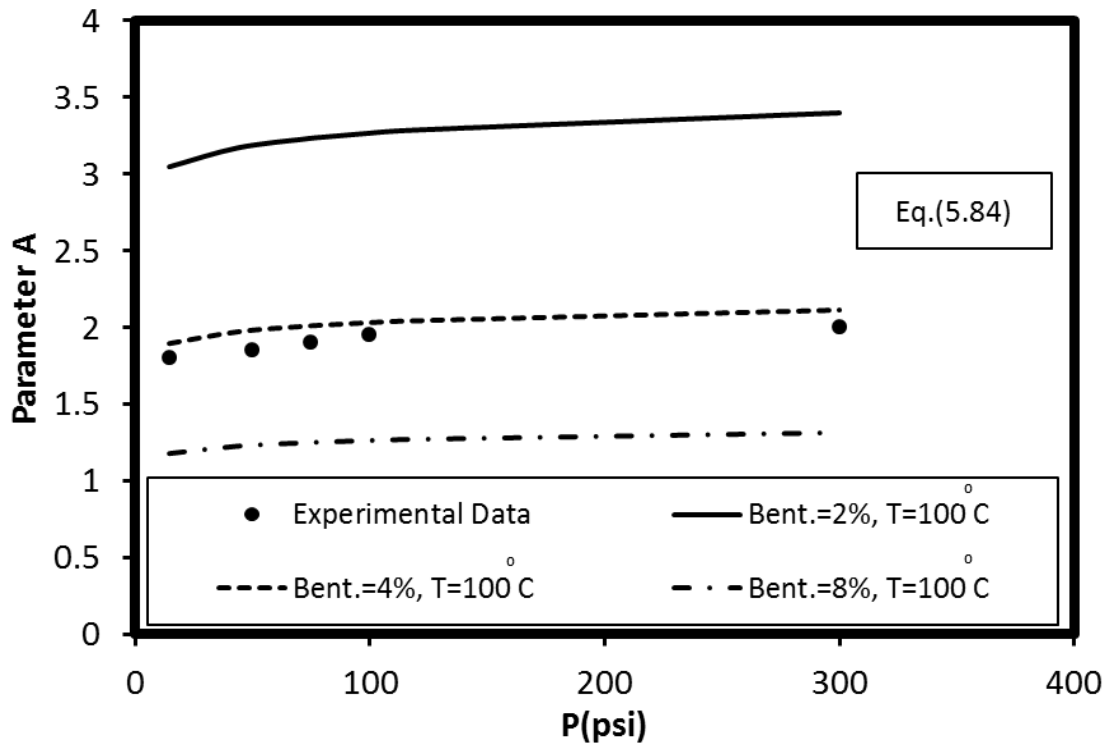


Figure 5.16 Variation of parameter a with pressure at 100° C.

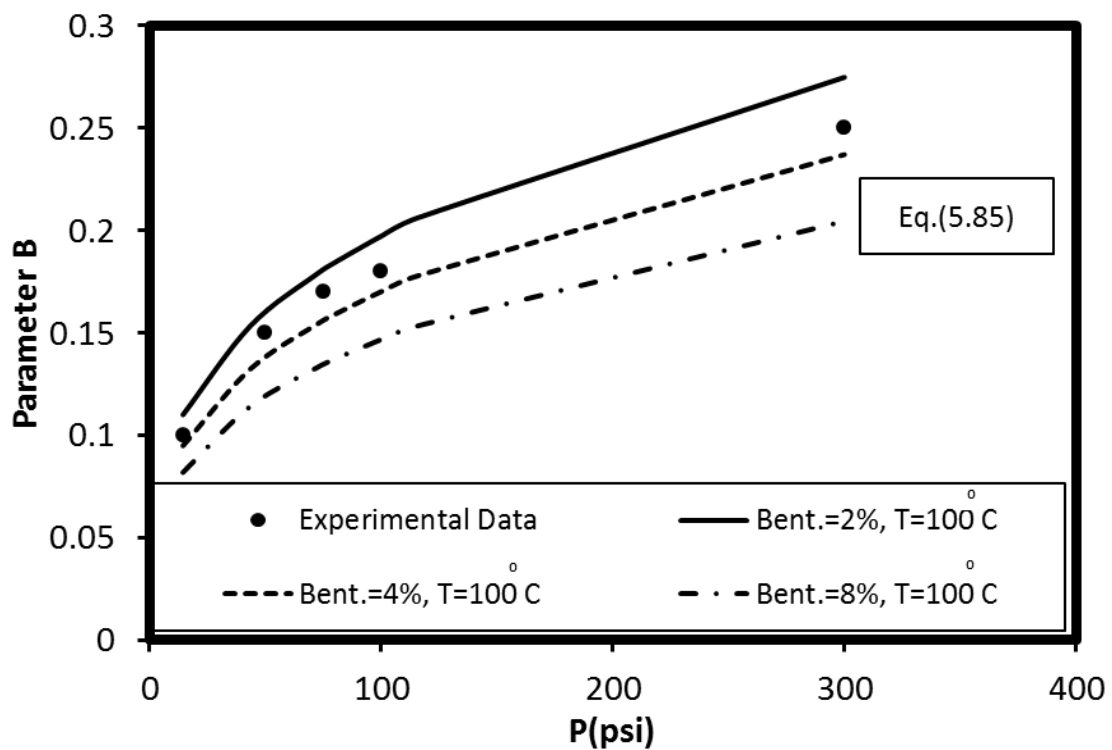


Figure 5.17 Variation of the parameter b with pressure at 100° C.

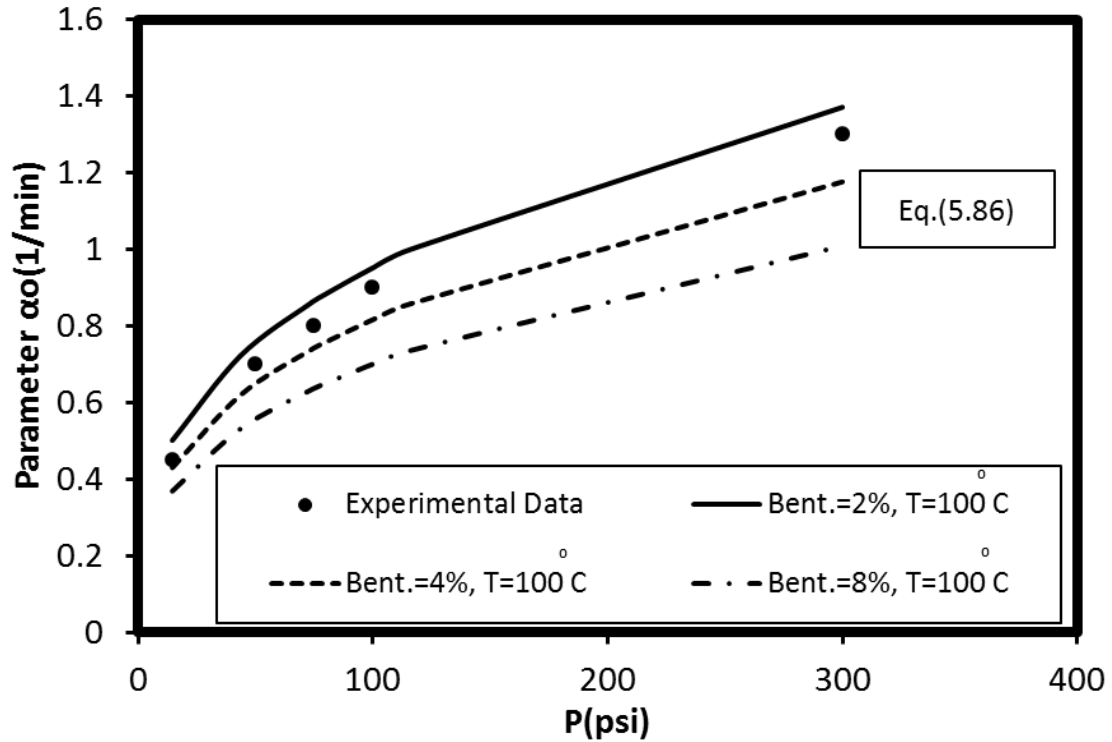


Figure 5.18 Variation of the parameter α_0 with pressure at 100° C.

API-Model

Fluid Loss (V_f)

API model predicted the fluid loss very well with R^2 and parameter M of 0.99 and $0.68 \text{ cm}^3/\text{min}^{0.5}$ respectively as shown in Figure 5.21(a). The spurt fluid loss was 4.3 cm^3 . The predicted fluid loss using the API model after 30 min was 22.4 cm^3 (100.1% of experimental value). API model predicted the maximum fluid loss to be infinity.

Porosity (n)

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the changes in the cake porosity with time (Eq. (5.77)). At the end of the test and based on the solid content in the filter cake, the cake porosity was determined to be

91% as shown in Figure 5.21 (b). Hence, the porosity was assumed to be 91% for the API model.

Permeability (k)

The API model does not predict the changes in the cake permeability (k) with time having a low R^2 of 0.43. The used viscosity of the water at 225° F was 0.26 cp. Based on filter cake solid content (f_{sc}), viscosity (μ), and using Eq. (5.71), the API cake permeability was determined and it was constant with a value of 0.002 m.Darcy as shown in Figure 5.21 (c).

Relative Solid Content (f_{sc}/f_{sm})

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the change in the solid content in filter cake (f_{sc}) to the solid content in the mud (f_{sm}) with time. At the end of the test, the API prediction for the ratio of solid content in the cake to the solid content in the mud was 5% higher than the kinetic model prediction as shown in Figure 5.21 (d).

Cake Thickness (h_{mc})

The cake thickness variation with the time was under-predicted in the API model with an unrealistic R^2 value of -2.81 as shown in Figure 5.21 (e).

New Kinetic Model

Fluid Loss (V_f)

New kinetic model predicted the fluid loss very well with R^2 of 0.97. For the best model prediction, the parameters N , A , and B were $0.2 \text{ cm}^3/\text{min}$, 0.45 and $0.04/\text{min}$ respectively as shown in Figure 5.21 (a). Using the nonlinear model (Eqs. 5.84 and 5.85), A and B were 0.44 and $0.033/\text{min}$ respectively. The predicted fluid loss for new kinetic model at 30 min was 7.9 cm^3 (99% of experimental value). The maximum predicted fluid loss by the new kinetic model (V_o+N/B) was 9.3 cm^3 , which was equivalent to 2.5% of the total volume of the drilling mud.

Porosity (n)

Based on the solid content in cake (f_{sc}) at the end of the test, the parameter α_o is determined using Eq. (5.74). Then, the variation of porosity with time predicted using Eq. (5.77) as shown in Figure 5.21 (b). After 30 minutes, the predicted porosity using Eq. (5.77) was 91%. This also agreed with the porosity obtained from experimental test.

Permeability (k)

The variation of permeability with the time in the kinetic model (Eq. (5.73)) was predicted using $k_o=0.5 \text{ m.Darcy}$, $A=0.45$, $B=0.04/\text{min}$ and $p=1$ as shown in Figure 5.21 (c). Parameter k_o was obtained using Eq. (5.76) with $\alpha_o=0.2/\text{min}$, $\mu=0.26 \text{ cp}$ and $N=0.2 \text{ cm}^3/\text{min}$. The final cake permeability was 0.000163 m.Darcy 12 times smaller than the permeability predicted by the API model. Using the porosity with the Khatib (1994) permeability relationship (Eq. (5.88)), it over predicted the cake permeability.

Relative Solid Content (f_{sc}/f_{sm})

The variation of the solid content in the cake (f_{sc}) to the mud (f_{sm}) with the time was determined using Eq. (5.74) with $\alpha_o=0.2/\text{min}$ as shown in Figure 5.21 (d). Using the nonlinear model (Eq. 5.86), α_o was 0.22/min. At the end of the test, the ratio of the solid content in the cake to the solid content in the mud predicted by kinetic model was 5% lower than the API model prediction.

Cake Thickness (h_{mc})

The cake thickness variation with the time was estimated reasonably well by the new kinetic model (assuming the parameters $A=0.45$, $B=0.04/\text{min}$, and $p=1$) using Eq. (5.67) as shown in Figure 5.21 (e). The final cake thickness predicted by the new kinetic model was 209% of the cake thickness predicted by API model.

5.5.2.Long-term tests ($t > 30 \text{ min}$)

Most of the fluid loss tests are performed for more than 30 min either in the laboratory or in the real field conditions so it is important to quantify the filter cake formation real time taking into account the changes in the cake porosity, permeability, the solids content in the cake to the mud, and the variation of the cake thickness with the time. In addition, comparing the predictions of both API and hyperbolic models would be more appropriate.

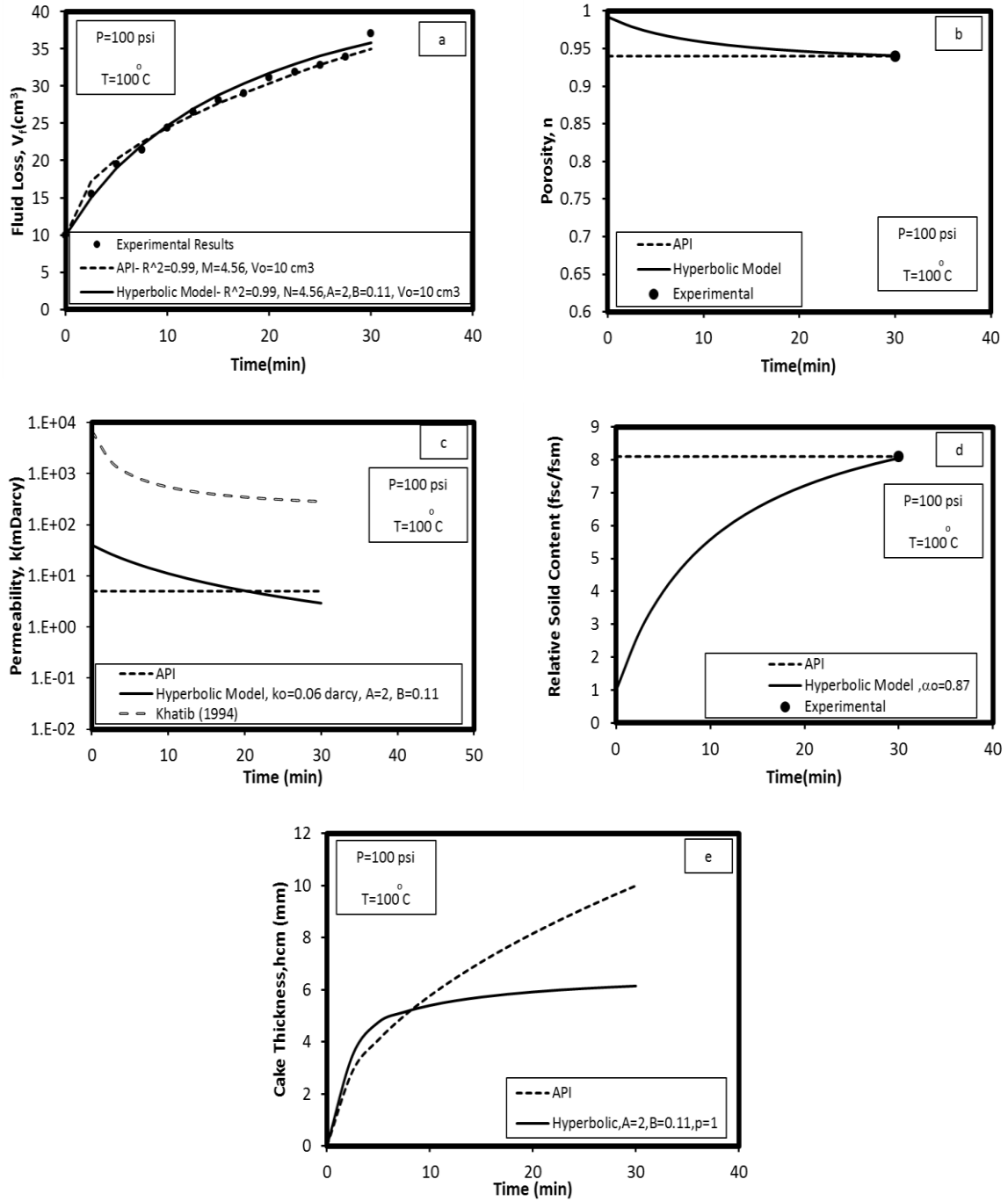


Figure 5.19 Model predictions of current study on 2% bentonite drilling mud at 100° C (a) fluid loss versus time, (b) variation of filter cake porosity with time, (c) variation of permeability with time, (d) variation of solid contents in the filter cake to mud ratio time, and (e) variation of cake thickness with time.

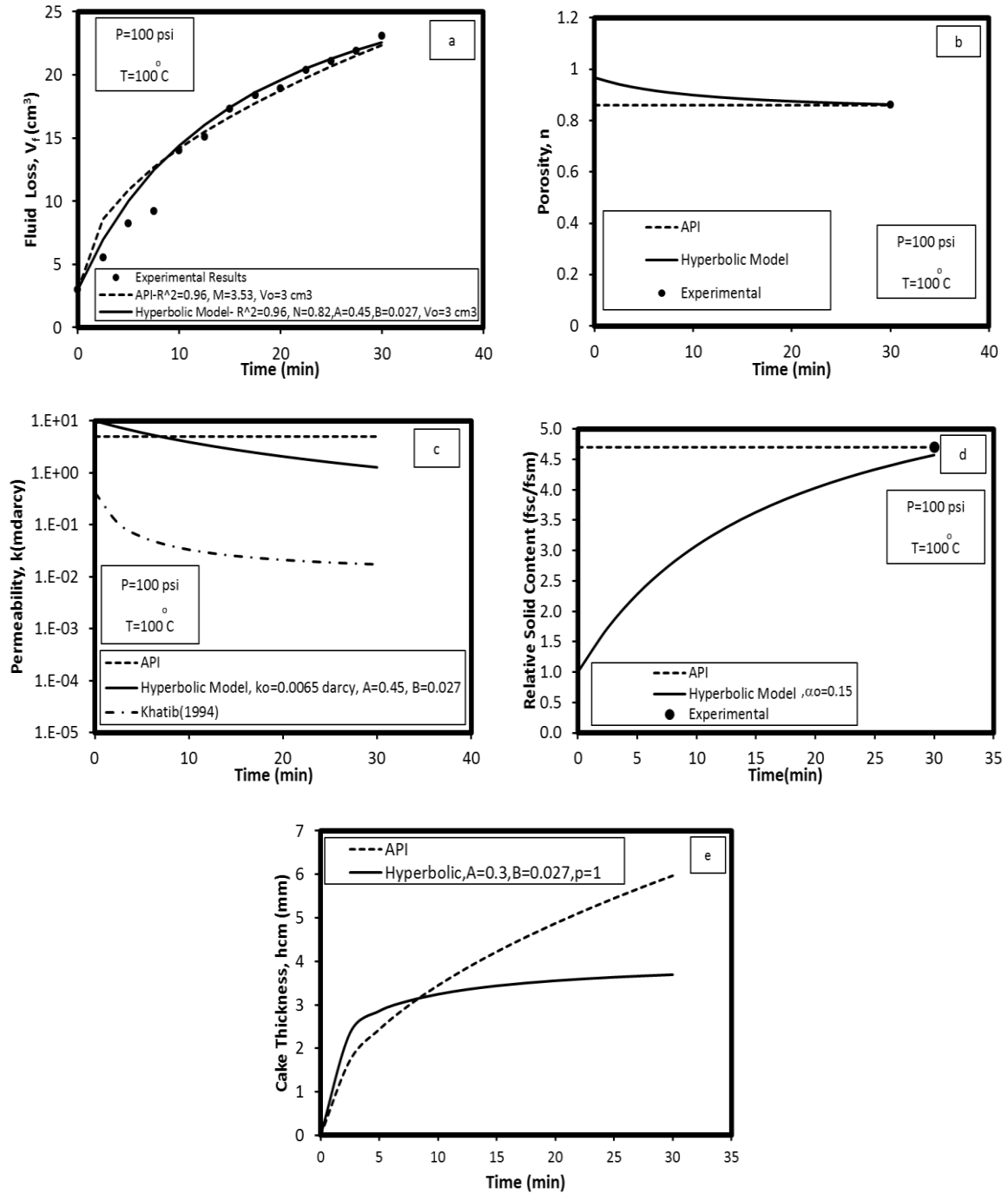


Figure 5.20 Model predictions of current study on 8% bentonite drilling mud at 100° C (a) fluid loss versus time, (b) variation of filter cake porosity with time, (c) variation of permeability with time, (d) variation of solid contents in the filter cake to mud ratio time, and (e) variation of cake thickness with time.

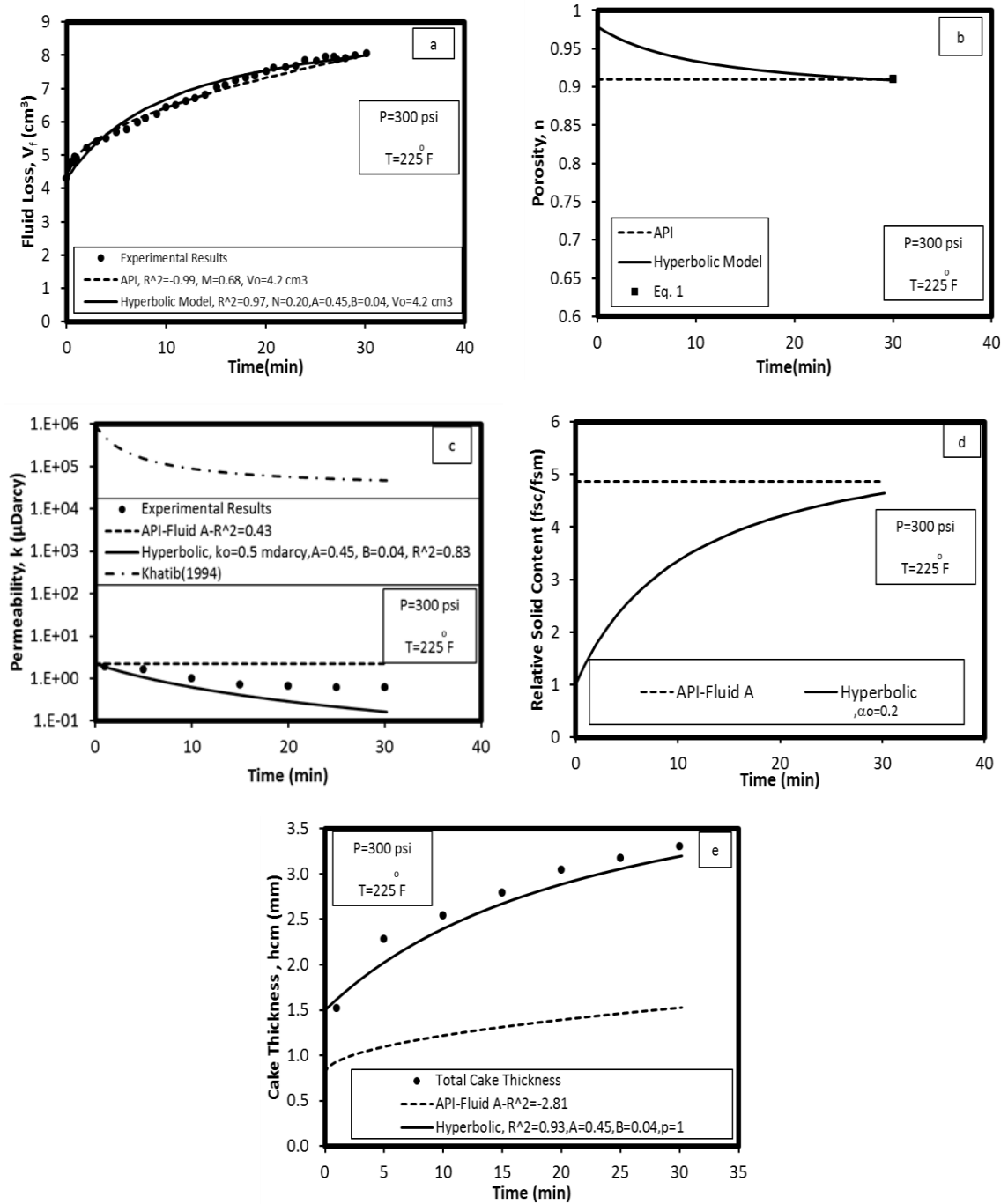


Figure 5.21 Model predictions of data in the literature on 5% bentonite drilling mud at 225° C (a) fluid loss versus time, (b) permeability with time, (c) variation of solid contents in the filter cake to mud ratio time, (d) variation of filter cake porosity with time, and (e) variation of cake thickness with time (Elkatatny et al. 2012 a).

(i) Case 4

A drilling mud with 2% bentonite content was prepared and tested in 100 psi pressure at 100° C as shown in Figure 5.22. The pressure and the temperature during the testing were maintained constant and the testing time lasted for 420 min.

API-Model

Fluid Loss (V_f)

API model predicted the fluid loss very well with R^2 and parameter M of 0.95 and $5.88 \text{ cm}^3/\text{min}^{0.5}$ respectively as shown in Figure 5.22 (a). The spurt fluid loss was 10 cm^3 . The predicted fluid loss using the API model after 420 min was 130.7 cm^3 (109% of experimental value). API model predicted the maximum fluid loss to be infinity.

Porosity (n)

Based on the moisture content (w) of the filter cake at the end of the test, the porosity using Eq. (5.87) was determined to be 92% and the solid content by volume (f_{sc}) in the cake was 8% (Eq. 5.77). Since the filter cake porosity in the API model is assumed to be a constant with time, the variation is shown in Figure 5.22 (b).

Permeability (k)

The API model does not predict the changes in the cake permeability (k) with time. The used viscosity of the water at 100° C was 0.28 cp. Based on filter cake solid content (f_{sc}), viscosity (μ), parameter M and using Eq. (5.71), the API cake permeability was determined and it was constant with a value of 0.015 m.Darcy as shown in Figure 5.22 (c).

Relative Solid Content (f_{sc}/f_{sm})

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the change in the solid content in filter cake (f_{sc}) to the solid content in the mud (f_{sm}) with time. At the end of the test, the ratio of the solid content in the cake to the solid content in the mud was the same as the API model prediction as shown in Figure 5.22 (d).

Cake Thickness (h_{mc})

Using Eq. (5.67), the cake thickness (h_{mc}) was predicted to be 46 mm after 420 minutes as shown in Figure 5.22 (e). The cake thickness variation with the time was over-predicted in the API model. API model predicted the maximum cake thickness to be infinity. The cake thickness at the end of the test was 24% of the API model prediction.

New Kinetic Model

Fluid Loss (V_f)

New kinetic model predicted the fluid loss very well with R^2 of 0.99. For the best model prediction, the parameters N, A, and B were 4.56 cm³/min, 3 and 0.035/min respectively as shown in Figure 5.22 (a). Using the nonlinear model (Eqs. 5.84 and 5.85), A and B were 2.8 and 0.033/ min respectively. The predicted fluid loss for new kinetic model at 420 min was 118.3 cm³ (99% of experimental value). The maximum predicted fluid loss by the new kinetic model (V_o+N/B) was 140.3 cm³, which was equivalent to 37% of the total volume of the drilling mud.

Porosity (n)

Based on the solid content in cake (f_{sc}) at the end of the test, the parameter α_o is determined using Eq. (5.74). Then, the variation of porosity with time predicted using Eq. (5.77) as shown in Figure 5.22 (b). After 420 minutes, the predicted porosity using Eq. (5.77) was 92%. This also agreed with the porosity obtained from experimental test.

Permeability (k)

The variation of permeability with the time in the kinetic model (Eq. (5.73)) was predicted using $k_o=0.015$ Darcy, $A=3$, $B=0.035/\text{min}$ and $p=1$ as shown in Figure 5.22 (c). Parameter k_o was obtained using Eq. (5.76) with $\alpha_o=0.4/\text{min}$, $\mu=0.28$ cp and $N=4.56$ cm^3/min . The final cake permeability was 0.000373 m.Darcy 40 times smaller than the permeability predicted by the API model. Using the porosity with the Khatib (1994) permeability relationship (Eq. (5.88)), it over predicted the cake permeability.

Relative Solid Content (f_{sc}/f_{sm})

The variation of the solid content in the cake (f_{sc}) to the mud (f_{sm}) with the time was determined using Eq. (5.74) with $\alpha_o=0.4/\text{min}$ as shown in Figure 5.22 (d). Using the nonlinear model (Eq. 5.86), α_o was 0.43/min. The ratio of solid content in the cake to the solid content in the mud at the end of the test was 103% of the kinetic model prediction.

Cake Thickness (h_{mc})

The cake thickness variation with the time was estimated reasonably well by the new kinetic model (assuming the parameters $A=0.3$, $B=0.035/\text{min}$, and $p=1$) using Eq. (5.67) as shown in Figure 5.22 (e). The final cake thickness predicted by the new kinetic model

was 30% of the cake thickness predicted by API model. The cake thickness at the end of the test was 79% of the kinetic model's value.

(ii) Case 5

A drilling mud with 8% bentonite content was prepared and tested under 100 psi pressure at 100° C as shown in Figure 5.23. The pressure and the temperature during the testing were maintained constant and the testing time lasted for 420 min.

API-Model

Fluid Loss (V_f)

API model predicted the fluid loss very well with R^2 and parameter M of 0.65 and $2.57 \text{ cm}^3/\text{min}^{0.5}$ respectively as shown in Figure 5.23 (a). The spurt fluid loss was 3 cm^3 . The predicted fluid loss using the API model after 420 min was 44.65 cm^3 (123% of experimental value). API model predicted the maximum fluid loss to be infinity.

Porosity (n)

Based on the moisture content (w) of the filter cake at the end of the test, the porosity using Eq. (5.87) was determined to be 86% and the solid content by volume (f_{sc}) in the cake was 14% (Eq. 5.77). Since the filter cake porosity in the API model is assumed to be a constant with time, the variation is shown in Figure 5.23 (b).

Permeability (k)

The API model does not predict the changes in the cake permeability (k) with time. The used viscosity of the water at 100° C was 0.28 cp. Based on filter cake solid content

(f_{sc}), parameter M and using Eq. (5.71), the API cake permeability was determined and it was constant with a value of 0.006 Darcy as shown in Figure 5.23 (c).

Relative Solid Content (f_{sc}/f_{sm})

Since the solid content (f_{sc}) in the filter cake is assumed constant, the API model does not predict the change in the solid content in filter cake (f_{sc}) to the solid content in the mud (f_{sm}) with time. At the end of the test, the ratio of the solid content in the cake to the solid content in the mud was the same as the API model prediction as shown in Figure 5.23 (d).

Cake Thickness (h_{mc})

Using Eq. (5.67), the cake thickness (h_{mc}) was predicted to be 27 mm after 420 minutes as shown in Figure 5.23 (e). The cake thickness variation with the time was over-predicted in the API model. API model predicted the maximum cake thickness to be infinity. The cake thickness at the end of the test was 67% of the API model prediction.

New Kinetic Model

Fluid Loss (V_f)

New kinetic model predicted the fluid loss very well with R^2 of 0.97. For the best model prediction, the parameters N, A, and B were $0.725 \text{ cm}^3/\text{min}^{0.5}$, 0.6 and 0.016/min respectively as shown in Figure 5.23 (a). Using the nonlinear model (Eqs. 5.84 and 5.85), A and B were 0.56 and 0.015/min respectively. The predicted fluid loss for new kinetic model at 420 min was 44.65 cm^3 (99% of experimental value). The maximum predicted

fluid loss by the new kinetic model (V_o+N/B) was 48.3 cm^3 , which was equivalent to 13% of the total volume of the drilling mud.

Porosity (n)

Based on the solid content in cake (f_{sc}) at the end of the test, the parameter α_o is determined using Eq. (5.74). Then, the variation of porosity with time predicted using Eq. (5.77) as shown in Figure 5.23 (b). After 420 minutes, the predicted porosity using Eq. (5.77) was 86%. This also agreed with the porosity obtained from experimental test.

Permeability (k)

The variation of permeability with the time in the kinetic model (Eq. (5.73)) was predicted using $k_o=0.00095$ Darcy, $A=0.6$, $B=0.016/\text{min}$ and $p=1$ as shown in Figure 5.23 (c). Parameter k_o was obtained using Eq. (5.76) with $\alpha_o=0.65 \text{ /min}$, $\mu=0.28 \text{ cp}$ and $N=0.725 \text{ cm}^3/\text{min}$. The final cake permeability was 0.0217 m.Darcy 277 times smaller than the permeability predicted by the API model. Using the porosity with the Khatib (1994) permeability relationship (Eq. (5.88)), it over predicted the cake permeability.

Relative Solid Content (f_{sc}/f_{sm})

The variation of the solid content in the cake (f_{sc}) to the mud (f_{sm}) with the time was determined using Eq. (5.74) with $\alpha_o=0.65/\text{min}$ as shown in Figure 5.23 (d). Using the nonlinear model (Eq. 5.86), α_o was $0.67/\text{min}$. The ratio of solid content in the cake to the solid content in the mud at the end of the test was 101% of the kinetic model prediction.

Cake Thickness (h_{mc})

The cake thickness variation with the time was estimated reasonably well by the new kinetic model (assuming the parameters $A=0.6$, $B=0.016/\text{min}$, and $p=1$) using Eq. (5.67) as shown in Figure 5.23 (e). The final cake thickness predicted by the new kinetic model was 67% of the cake thickness predicted by API model. The cake thickness at the end of the test was 99% of the kinetic model's value.

As the Bentonite concentration was increased from 2% to 8%, the maximum fluid loss value was decreased by 62 % for 100 °C. The reported decrease in the fluid loss was mainly due to the increase in the slurry density, which blocks the pores of the developed filter cake and reduces the flow of the infiltrated fluid.

5.5.3. Model validation for different drilling fluids

The accuracy for both API and kinetic models were validated using fluid loss versus time data for different drilling fluids reported in the literature as

(i) Case 1

Three different drilling fluids have been investigated (Basirat et al., 2013). Fluid A consisted of water, 4% bentonite and 0.5 lb/bbl (equivalent to 1.43 g/L) xanthan gum ($\text{C}_{35}\text{H}_{49}\text{O}_{29}$). Fluid B consisted of water, 4% bentonite, 0.5 lb/bbl (equivalent to 1.43 g/L) xanthan gum ($\text{C}_{35}\text{H}_{49}\text{O}_{29}$) and 0.5% salt (sodium chloride, NaCl). Fluid C consisted of water, 4% bentonite and 0.5 lb/bbl (equivalent to 1.43 g/L) xanthan gum ($\text{C}_{35}\text{H}_{49}\text{O}_{29}$), 0.5% salt (sodium chloride, NaCl) and 0.5% surfactant. The fluid loss versus time was measured for up to 30 minutes. Both API and kinetic models were used to predict the fluid loss over the time as shown in Figure 5.24. The coefficient of correlation for both

API and kinetic models were summarized in Table 5.4. Both models predicted the fluid loss with high accuracy where kinetic model prediction was higher for fluid A and C and lower for fluid B.

(ii) Case 2

Three ester based drilling fluid samples with different salt (NaCl) contents were used (Prasad and Vipulanandan, 2013). Drilling fluids contained salt with 5%, 10%, and 20% and denoted as fluid A, B, and C respectively. The fluid loss versus time was measured for up to 30 minutes. Both API and kinetic models were used to predict the fluid loss over the time as shown in Figure 5.25. The coefficient of correlation for both API and kinetic models were summarized in Table 5.5. Both models predicted the fluid loss with high accuracy where kinetic model prediction was higher for fluid A and the same as API model for fluid B, and C.

(iii) Case 3

Two modified cement samples were investigated without and with 1% nanosilica keeping water-to-cement ratio at 0.4 for both samples (Amani et al., 2014). Non treated and treated cement samples were denoted as fluid A, and B respectively. The fluid loss versus time was measured for up to 30 minutes. Both API and kinetic models were used to predict the fluid loss over the time as shown in Figure 5.26. The coefficient of correlation for both API and kinetic models were summarized in Table 5.6. Both models predicted the fluid loss with high accuracy where kinetic model prediction was higher for both fluid A and fluid B and up to 0.99.

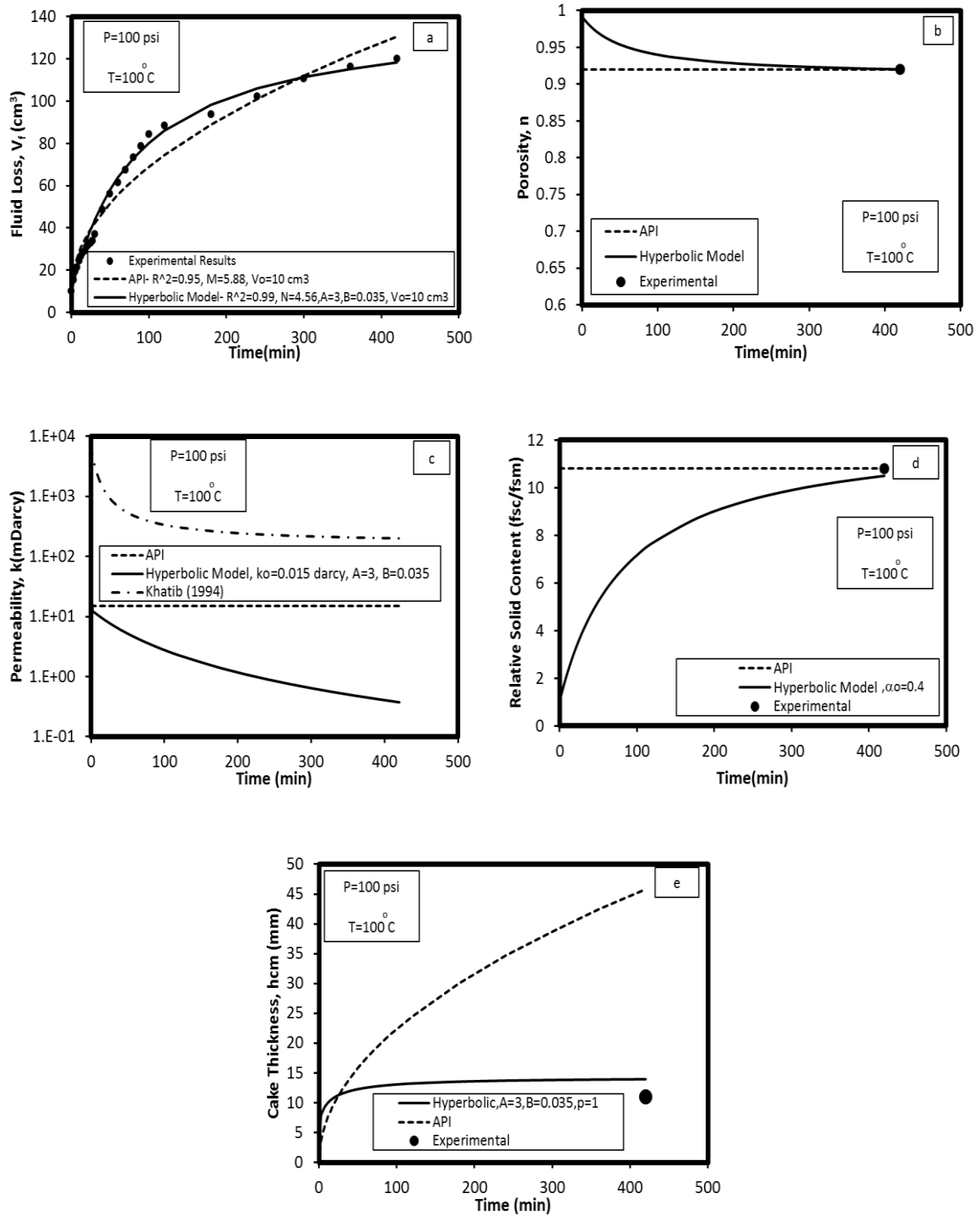


Figure 5.22 Long-term model predictions of current study on 2% bentonite drilling mud at 100°C (a) fluid loss versus time, (b) variation of filter cake porosity with time, (c) variation of permeability with time, (d) variation of solid contents in the filter cake to mud ratio time, and (e) variation of cake thickness with time.

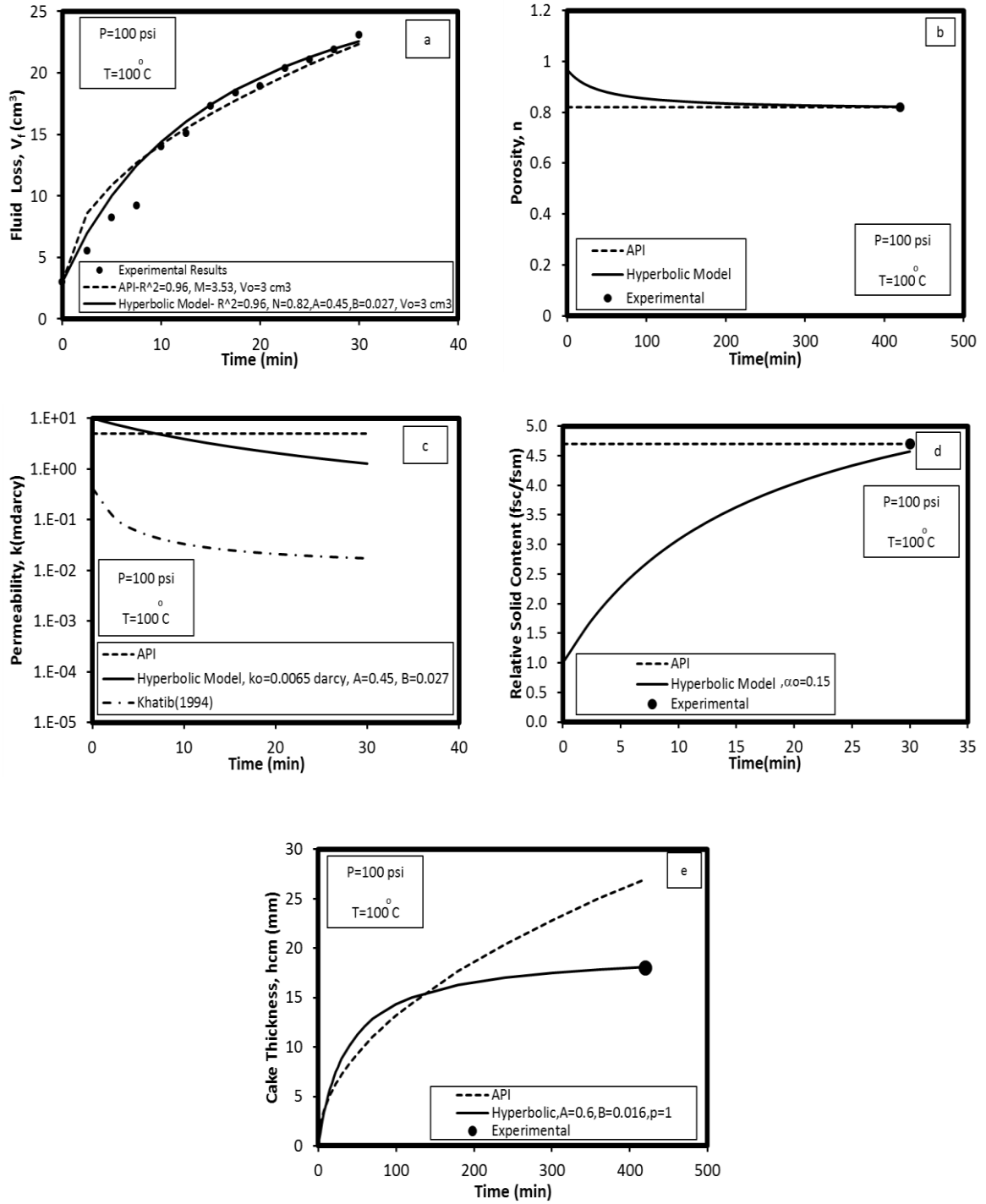


Figure 5.23 Long-term model predictions of current study on 8% bentonite drilling mud at 100°C (a) fluid loss versus time, (b) variation of filter cake porosity with time, (c) variation of permeability with time, (d) variation of solid contents in the filter cake to mud ratio time, and (e) variation of cake thickness with time.

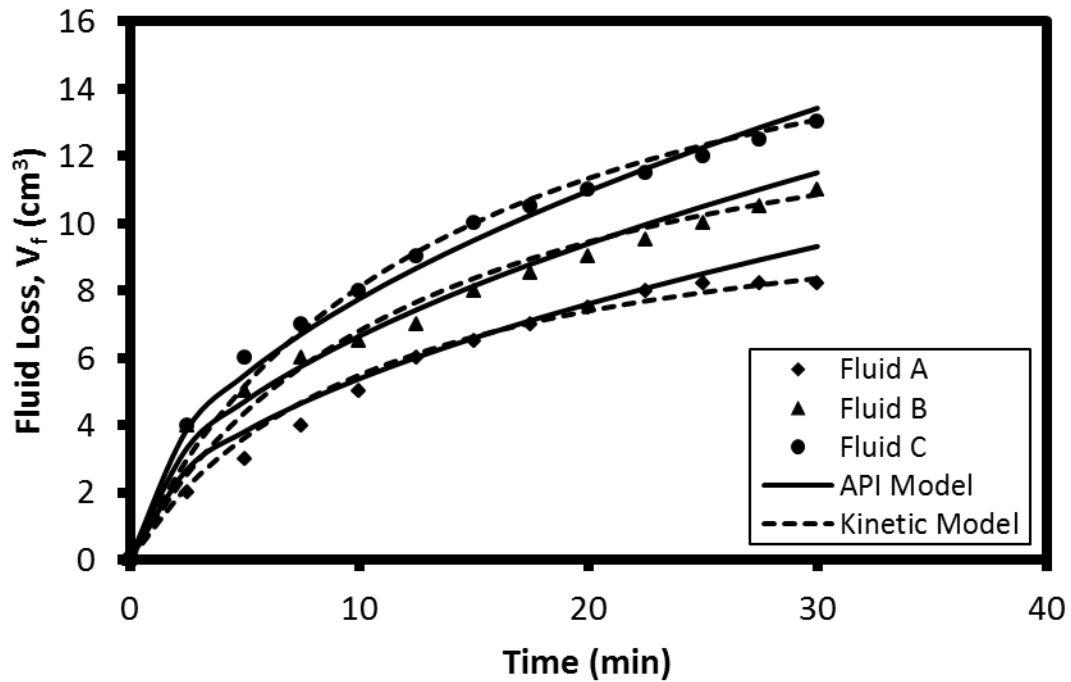


Figure 5.24 Short-term predictions of fluid loss versus time of three different drilling fluids using API and kinetic models (Basirat et al. 2013).

Table 5.4 The coefficient of correlation of API and kinetic model for three different drilling fluids (Basirat et al. 2013).

Model	Fluid A	Fluid B	Fluid C
	R^2	R^2	R^2
API	0.99	0.98	0.96
Kinetic	0.99	0.97	0.98

(iv) Case 4

Three drilling mud samples of 4% bentonite contents were used (Basirat et al., 2014). The 4% untreated bentonite drilling mud was denoted as fluid A. The 4% treated bentonite drilling mud with 1% and 3% of biopolymer powder samples were denoted as fluid B, and C respectively. The fluid loss versus time was measured for up to 30 minutes. Both API and kinetic models were used to predict the fluid loss over the time as shown in Figure 5.27. The coefficient of correlation for both API and kinetic models

were summarized in Table 5.7. Both models predicted the fluid loss with high accuracy where kinetic model prediction was higher for both fluids A and C and the same as API model for fluid B.

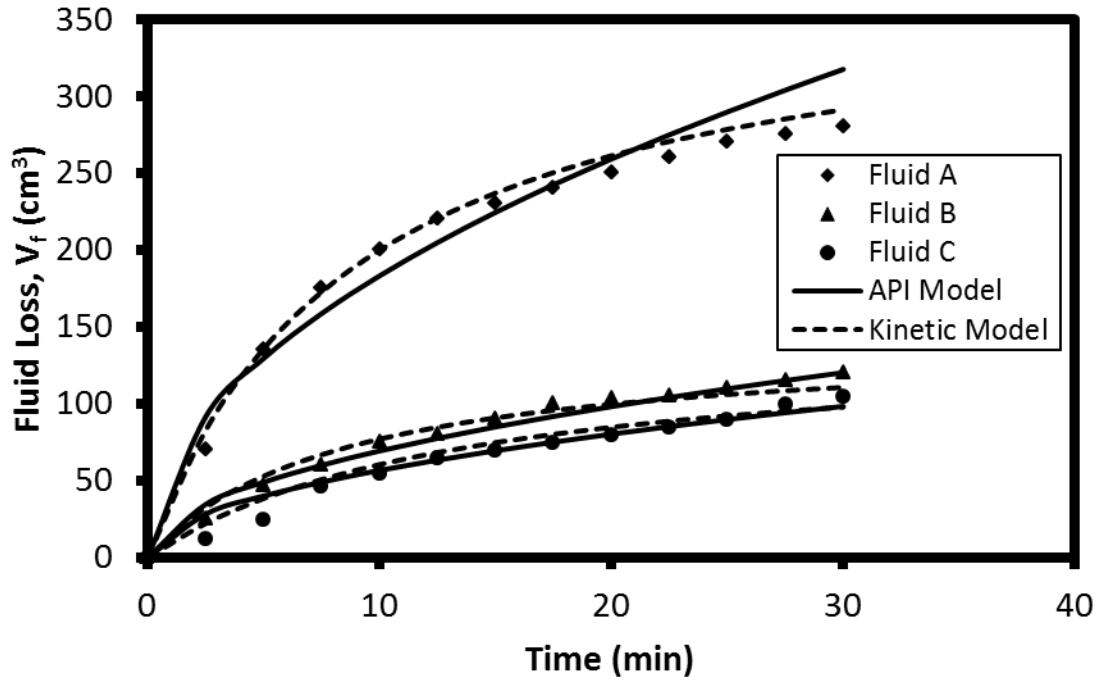


Figure 5.25 Short-term predictions of fluid loss versus time of three different drilling fluids using API and kinetic models (Prasad and Vipulanandan 2013).

Table 5.5 The coefficient of correlation of API and kinetic model for three different drilling fluids (Prasad and Vipulanandan 2013).

Model	Fluid A	Fluid B	Fluid C
	R^2	R^2	R^2
API	0.95	0.98	0.94
Kinetic	0.99	0.98	0.94

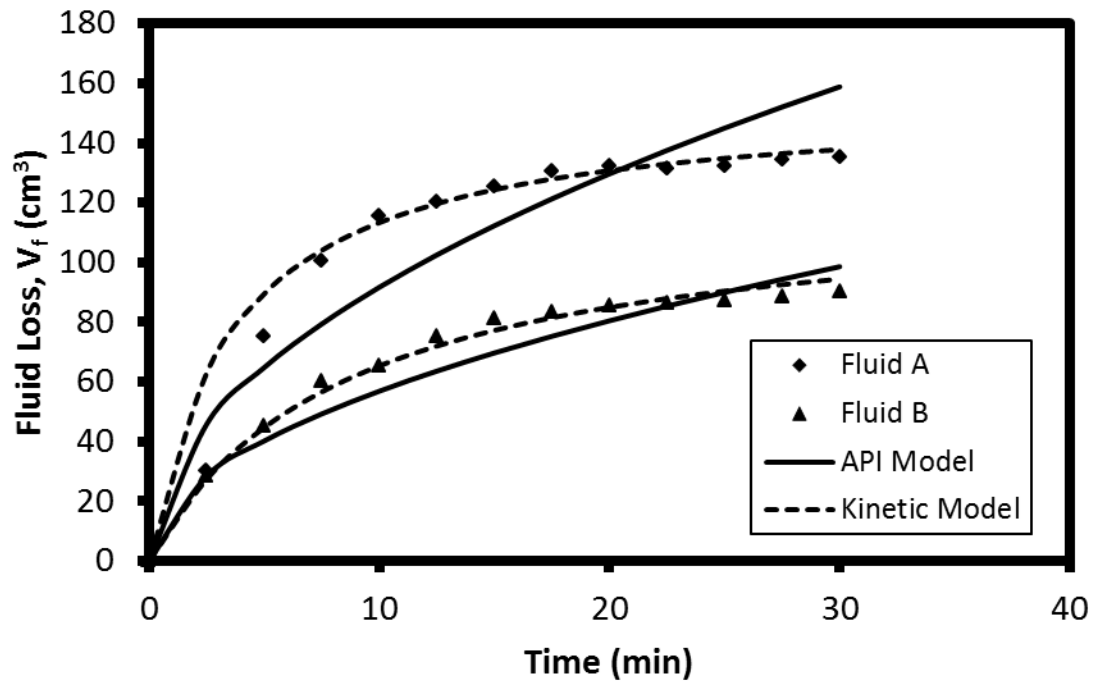


Figure 5.26 Short-term predictions of fluid loss versus time of three different drilling fluids using API and kinetic models (Amani et al. 2014).

Table 5.6 The coefficient of correlation of API and kinetic model for three different drilling fluids (Amani et al. 2014).

Model	Fluid A	Fluid B
	R^2	R^2
API	0.87	0.93
Kinetic	0.94	0.99

5.6.Summary

In this chapter, the following points can be summarized:

- 1.A new method for pore water pressure, coupling solution, was provided to solve consolidation equation. This method assumed the permeability and coefficient of consolidation of the filter cake to be a time dependent function.

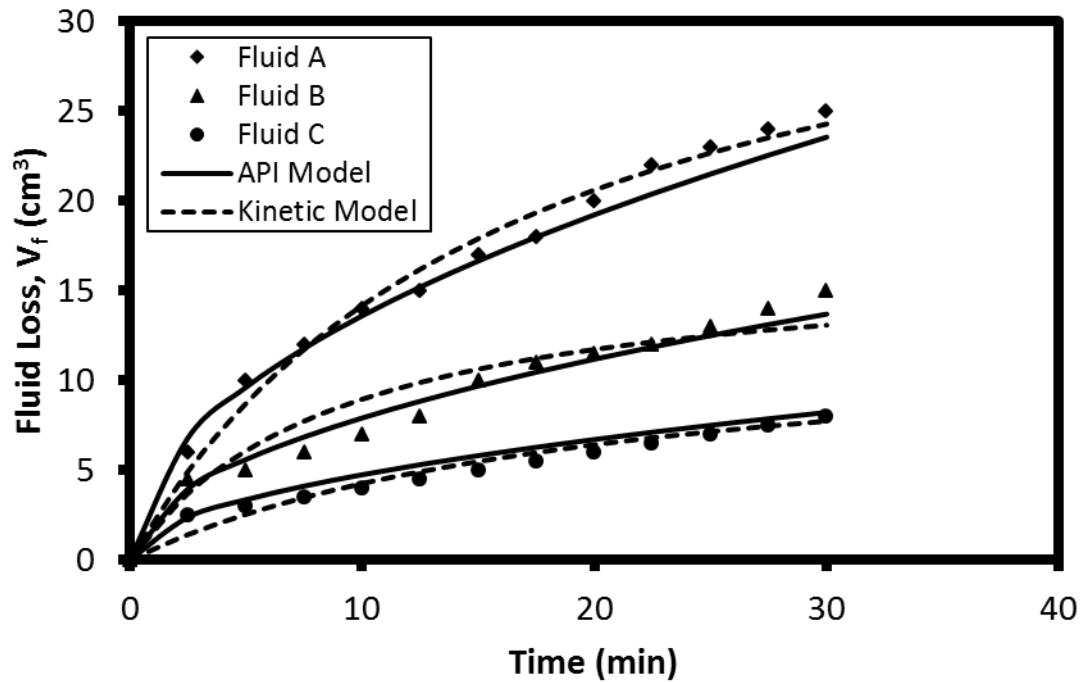


Figure 5.27 Short-term predictions of fluid loss versus time of three different drilling fluids using API and kinetic models (Basirat et al. 2014).

Table 5.7 The coefficient of correlation of API and kinetic model for three different drilling fluids (Basirat et al. 2014).

Model	Fluid A	Fluid B	Fluid C
	R^2	R^2	R^2
API	0.98	0.97	0.93
Kinetic	0.99	0.97	0.96

2. The new method provided consolidation solution was compared with Terzaghi solution and API- method. The new method and Terzaghi solution were used to predict the variations of the fluid loss, cake permeability, coefficient of consolidation of the cake, and the pore water pressure with the time. The new method incorporated better prediction for the experimental results than Terzaghi consolidation solution and API-model.

3. The coefficient of permeability for 2% bentonite drilling mud at 25° C was constant over the time in Terzaghi method while the coefficient of permeability decreased by 126000 times over 420 minutes in the new new method.
4. The pore water pressure for 2% bentonite drilling mud at 25° C decreased by 24% and 26% over 420 minutes in Terzaghi and new method respectively.
5. The coefficient of permeability for 8% bentonite drilling mud at 25° C was constant over the time in Terzaghi method while the coefficient of permeability decreased by 365396 times over 420 minutes in new method and 2.
6. The pore water pressure for 8% bentonite drilling mud at 25° C decreased by 24% and 23% over 420 minutes in Terzaghi and new method respectively.
7. The API fluid loss model was made based on several assumptions, such as the percentage of solids in the cake is a constant, the permeability of the cake is constant, and viscosity is a constant at the testing temperature.
8. The kinetic model is made based on the rate of infiltration is dependent on the permeability of the filter cake and the ratio of the solid content in the cake to the solid content in the mud and the time.
9. The proposed kinetic model has a limit on the total amount of the fluid loss, whereas the API-model has no limit on the fluid loss and the fluid loss increased proportionally to the squared root of the time.
10. The kinetic model can quantify the variation of the cake porosity with the time, however; the porosity is constant in the API-model.
11. The kinetic model can take in consider the effect of both pressure and temperature on the fluid loss, whereas the API - model is independent on pressure and temperature.

12. All kinetic model parameters are strongly correlated to the applied pressure and tested temperature.
13. The fluid loss after 7.5 min and 30 min for 2% bentonite drilling mud under 100 psi and 100 °C were 21.4 cm³, 37.1 cm³ respectively. Both API and Hyperbolic models predicted the fluid loss very well.
14. The API model did not predict the changes in the permeability, the solid content in the cake to the mud and the porosity with the time while the new kinetic hyperbolic model predicted the changes in the properties with time.
15. The cake thickness variation with the time was over-predicted in the API model while it was estimated reasonably well by the hyperbolic model.
16. The fluid loss after 7.5 min and 30 min for 8% bentonite drilling mud under 100 psi and 100 °C were 9.2 cm³, 23.1 cm³ respectively. Both API and hyperbolic models predicted the fluid loss with time very well.
17. When the bentonite concentration increased from 2% to 8% in the drilling mud, the fluid loss decreased by 57% after 7.5 min and 61% after 30 min at a temperature of 100 °C.
18. The fluid loss for 5% bentonite drilling mud after 7.5 min and 30 min under 300 psi and 225° F were 6 cm³ and 8 cm³ respectively. Both API and hyperbolic models predicted the fluid loss with time very accurately.
19. When the bentonite concentration increased from 2% to 5% in the drilling mud with an increase in the applied pressure from 100 psi to 300 psi, the fluid loss decreased by 72% after 7.5 min and 78% after 30 min at almost the same tested temperatures (100°C and 107°C (225°F) respectively).

20. When the bentonite concentration increased from 5% to 8% in the drilling mud with a decrease in the applied pressure from 300 psi to 100 psi, the fluid loss increased by 53% after 7.5 min and 189% after 30 min at almost the same tested temperatures (107°C (225°F) and 100°C respectively).
21. Experimental tests showed that the maximum fluid loss in 2% bentonite mud slurry at final test time (420 min) for tested temperature 100 °C was 120 cm³; due to the decrease in the fluid viscosity at higher temperature.
22. Experimental tests showed that the maximum fluid loss of 8% bentonite mud slurry at final test time (420 min) for tested temperature 100 °C was 45.2 cm³.
23. As the bentonite concentration was increased from 2% to 8%, the maximum fluid loss value was decreased by 62 % for 100 °C after 420 minutes.
24. Kinetic model prediction was better than API-model prediction through several different drilling mud reported in the literature such as xanthan gum drilling mud, easter based drilling mud, modified cement with nanosilica, and biopolymer drilling mud.

Chapter 6 - Drilling Mud Contamination

6.1. Introduction

In this chapter, the effect of salt contamination and temperature on the drilling mud have been investigated. The effect of 3% salt contamination on the density of drilling mud with 2% to 10% bentonite contents at different temperatures from 5° C to 85° C was inspected. The impact of 3% salt on the pH of the 2% to 10% drilling mud was studied. The variation of the electrical resistivity with the salt content for the 2% and 10% bentonite drilling mud at different temperatures from 5° C to 85° C was examined. Impedance spectroscopy, electrical resistivity, and CIGMAT penetrometer were used to sense the salt contamination during downward and upward flow into 6% bentonite drilling mud through laboratory model test. Linear model was used to predict the relationship between the CIGMAT penetration versus different weights while non linear model was used to model the changes in the electrical resistivity and shear strength of drilling mud due to salt contamination. Several statistical models such as regression analysis, error estimation analysis, classical statistical method, and cumulative probability function were used to check the accuracy of the CIGMAT penetration, electrical resistivity, and shear strength models. In addition, an extensive study was done on the rheological properties of the drilling mud under the effects of both salt contamination and changes in temperature. Both Herschel-Bulkley and hyperbolic models were used to predict the variation of the shear stress with shear strain rate for the bentonite drilling mud under the effects of both salt contamination and changes in temperature. Also, hyperbolic constitutive models were used to model the yield shear stress, maximum shear stress, and electrical resistivity of bentonite drilling muds under the effect of both salt

contamination and changes in the temperature. Nonlinear models were used to investigate the combined effects of bentonite content, salt contamination contents, and the changes in the temperatures on the fundamental properties of the drilling mud such as yield shear stress, maximum shear stress, electrical resistivity and hyperbolic model parameters. Finally, finite element method was used to predict the downward flow of salt contamination to the bentonite drilling mud and it was verified with the experimental results.

6.2.Density

(i) Room temperature

The impact of salt effect on the density of the drilling mud with different bentonite content in room temperature has been shown in Figure 6.1. As the bentonite content increased, the density increased linearly. The density increased by 1.65% as the salt content increased from 0% to 3%.

(ii) Temperature effect

The impact of the salt effect on the density of the drilling mud with different bentonite content in different temperature has been shown in Figure 6.2. The density increased linearly with the increase of the bentonite content in the drilling mud in both tested temperatures (5°C to 85°C) with higher density for higher salt content. As the temperature changed from 5°C to 85°C, the density changed by 0.4% and 3% for 0% and 3% salt contents respectively.

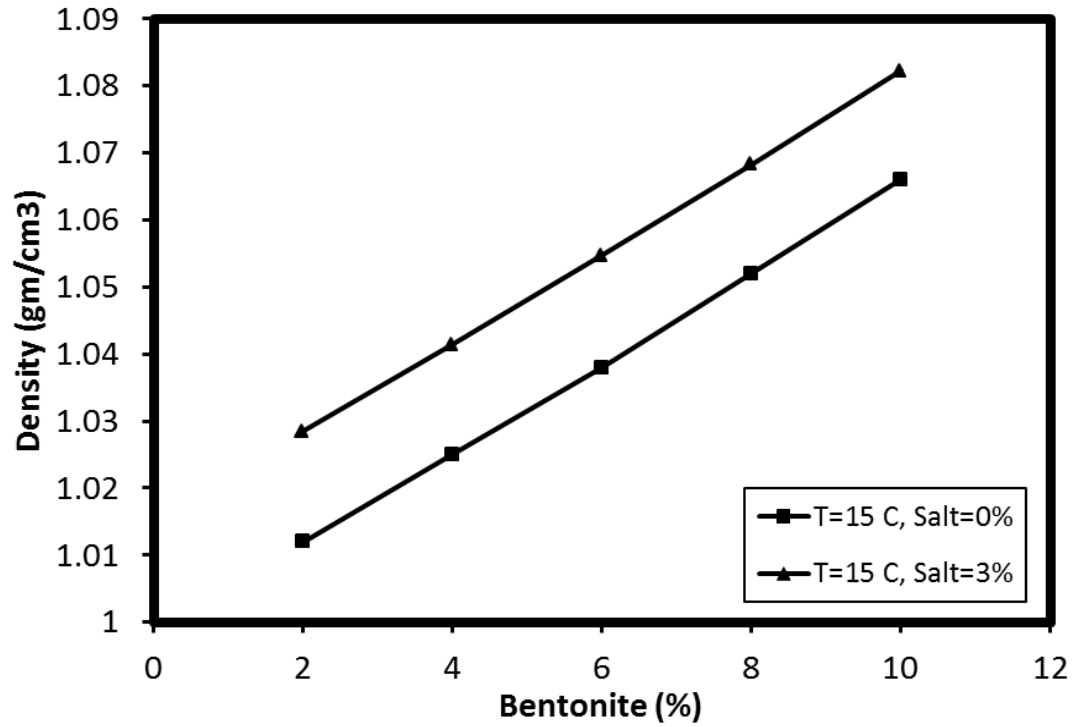


Figure 6.1 Salt effect on the density of the drilling mud with different bentonite content in room temperature.

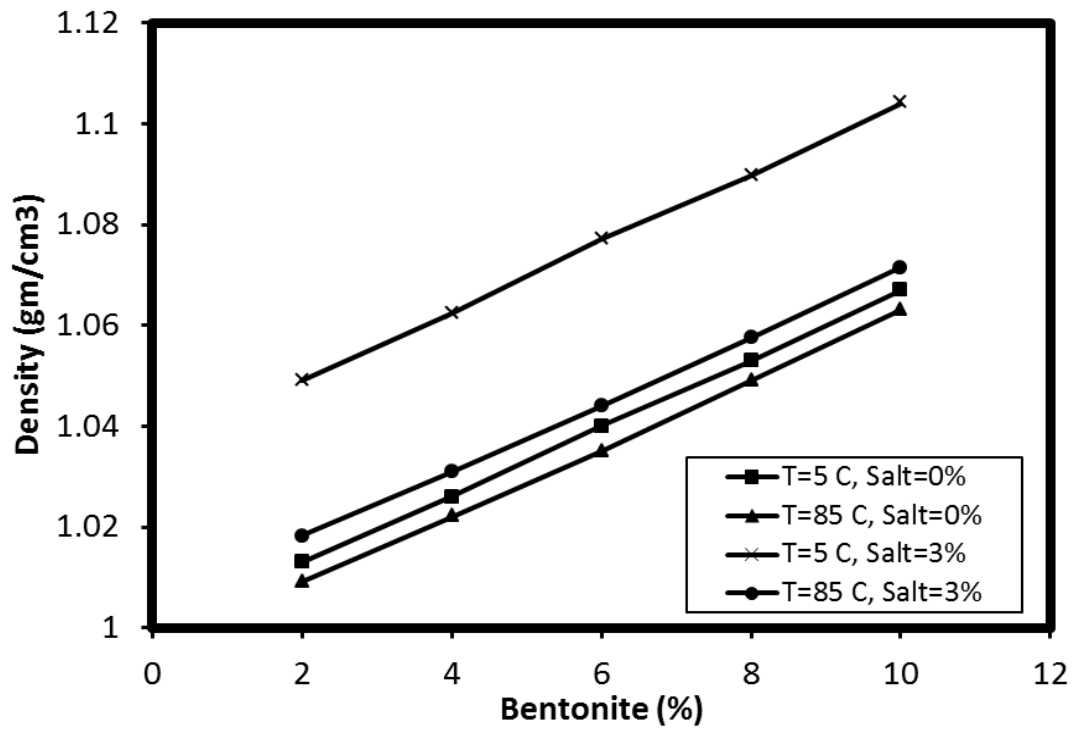


Figure 6.2 Salt effect on the density of the drilling mud with different bentonite content in different temperatures.

6.3.pH

The variation of pH with different bentonite contents under different salt contamination impact have been shown in Figure 6.3. In general, as the salt content was increased to 3%, the pH decreased, possibly due to the interaction between bentonite and salt. The pH reduced from 8.8 to 8.2 with 3% salt contamination with a change of 7 %.

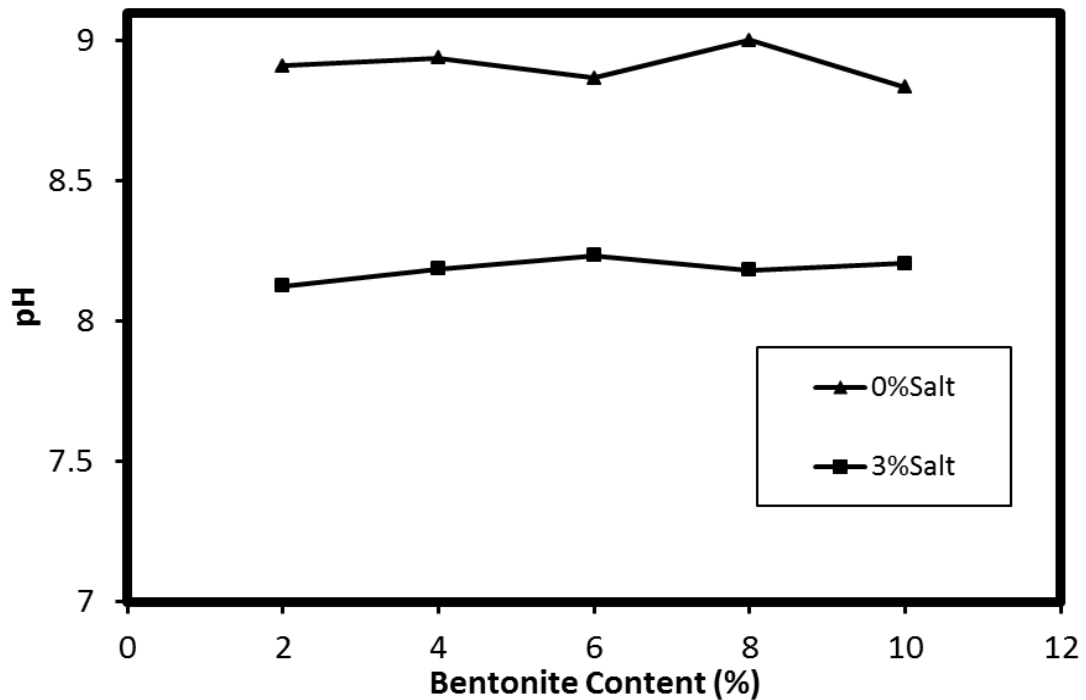


Figure 6.3 The variation of pH change with bentonite content for different salt contamination(%).

6.4.Resistivity

(i) Room temperature

The variation of electrical resistivity versus salt content of 2% and 10% bentonite drilling mud at room temperature has been shown in Figure 6.4. As the salt content increased, the electrical resistivity decreased having the highest impact at 0.1% salt content. As the salt content increased from 0.1% to 3%, the average decrease in the

electrical resistivities were 1600% and 918% for 2% and 10% bentonite content respectively.

(ii) Temperature effect

The variation of electrical resistivity versus salt content of (2% and 10%) bentonite content at different temperatures can be seen in Figure 6.5. As the temperature increased from 5° C to 85° C, the electrical resistivity decreased by 500% and 750% as an average rate for 2% and 10% bentonite content respectively.

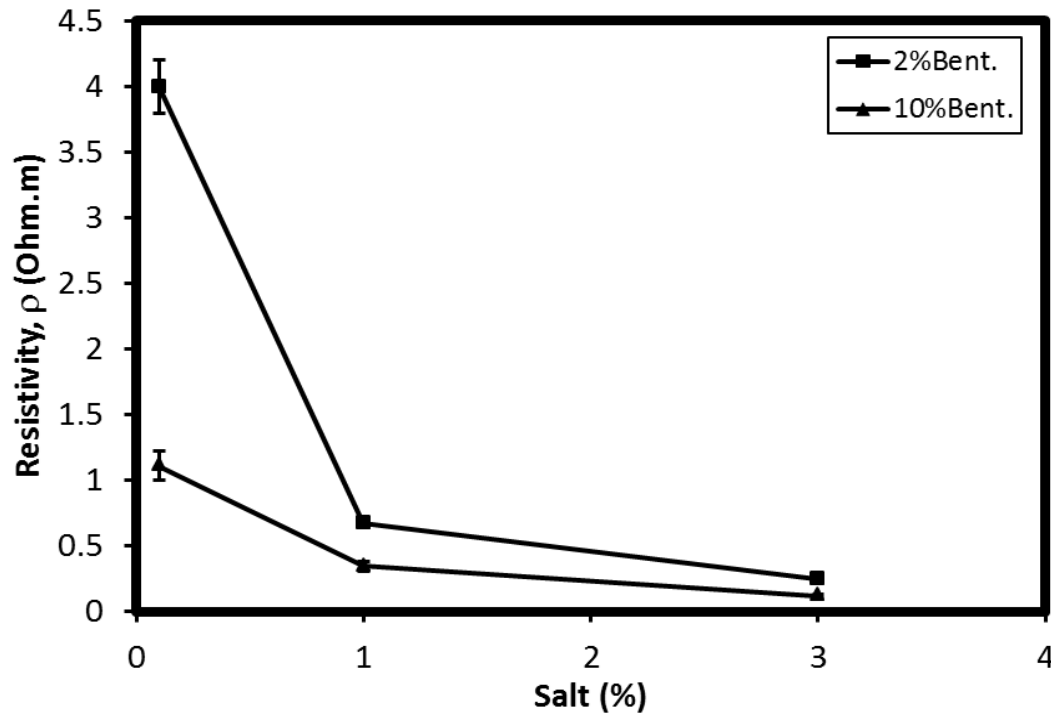


Figure 6.4 The variation of electrical resistivity vs. salt content of (2% and 10%) bentonite content at room temperature.

The electrical resistivity versus temperature of 2% and 10% bentonite content of 0% and 3% salt contents can be shown in Figure 6.6 and Figure 6.7 respectively. As the temperature increased from 5° C to 85° C, the electrical resistivity decreased by 130% and

354% for 0% salt content and 870% and 1328% for 3% salt content of 2% and 10% bentonite content respectively.

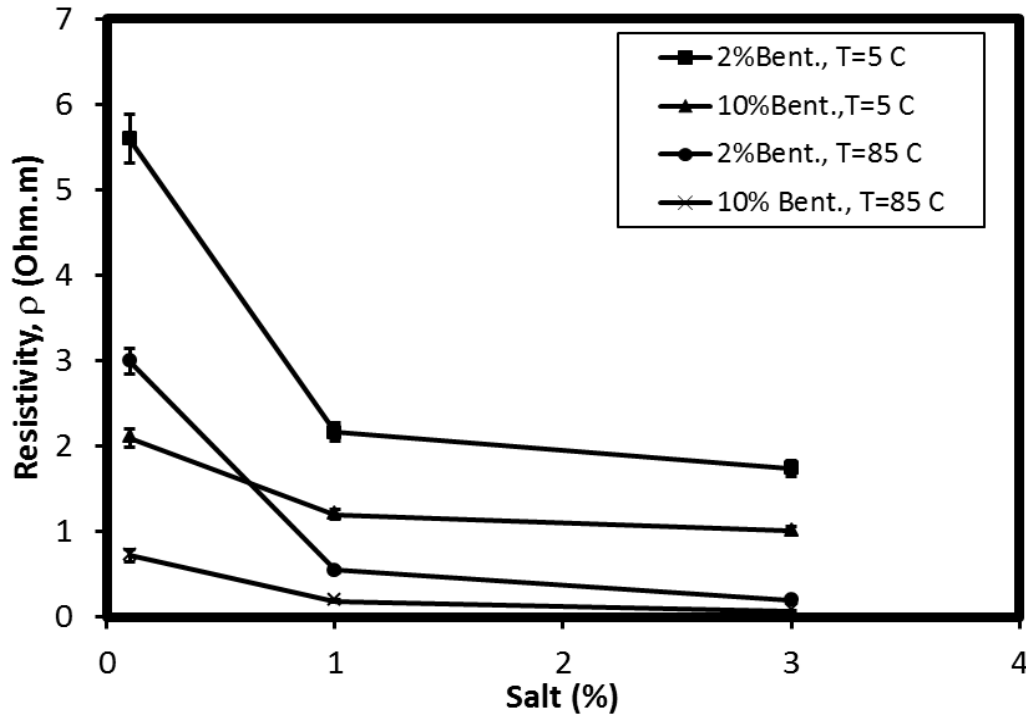


Figure 6.5 The variation of electrical resistivity vs. salt content of (2% and 10%) bentonite content at different temperatures.

6.5.Model test

(i) Method of monitoring

Impedance versus frequency of two drilling mud contents (2% and 10%) under different salt contaminations (0.1%, 1% and 3%) have been studied as shown through Figure 6.8 and Figure 6.9. As the bentonite content increased, the measured impedance decreased for all measured frequencies which indicates that the bentonite is highly conductive material where higher contents give lower electrical resistivity. Salt contamination decreased the measured electrical impedance for both tested bentonite drilling mud contents. All tested bentonite drilling mud with and without salt

contamination showed pure resistance behavior at high measured frequency (300 kHz) where the effect of contact resistance is minor and negligible.

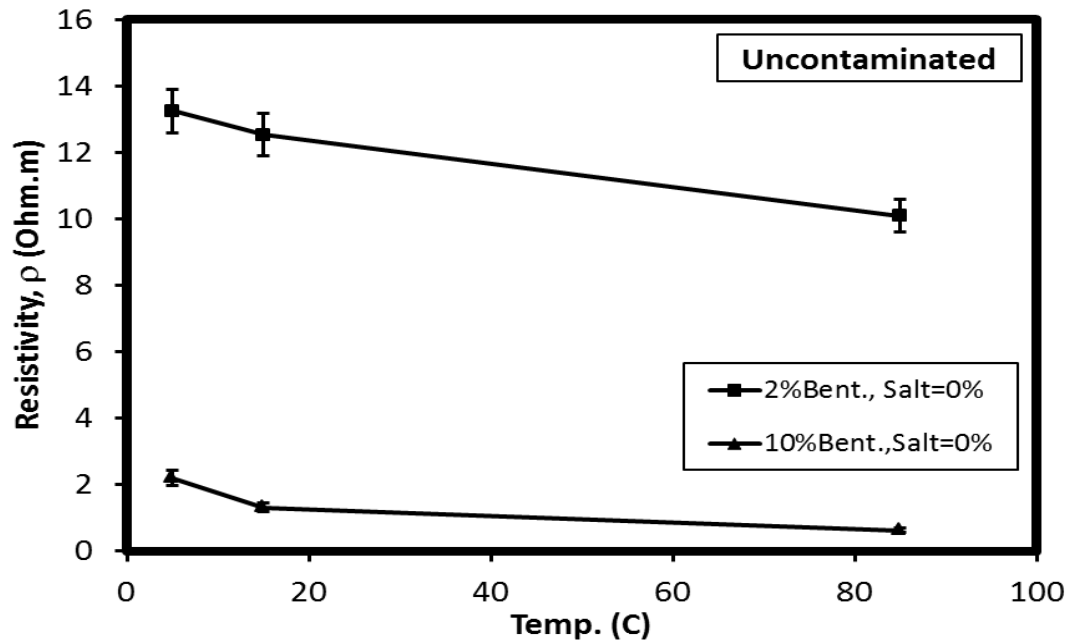


Figure 6.6 The variation of electrical resistivity vs. temperature of (2% and 10%) bentonite content (0% salt).

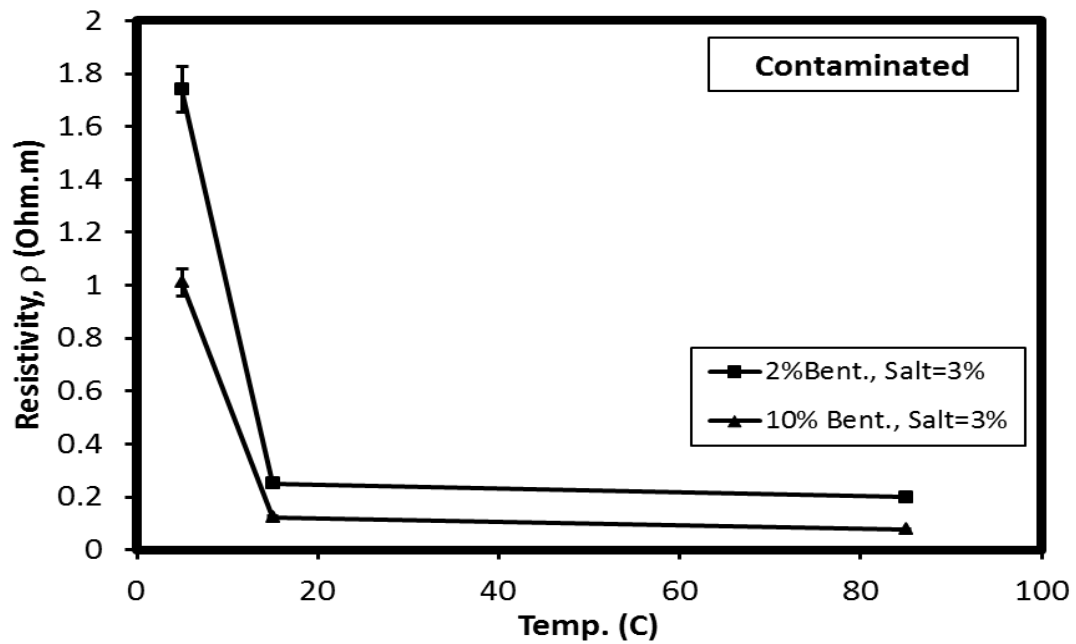


Figure 6.7 The variation of electrical resistivity vs. temperature of (2% and 10%) bentonite content (3% salt).

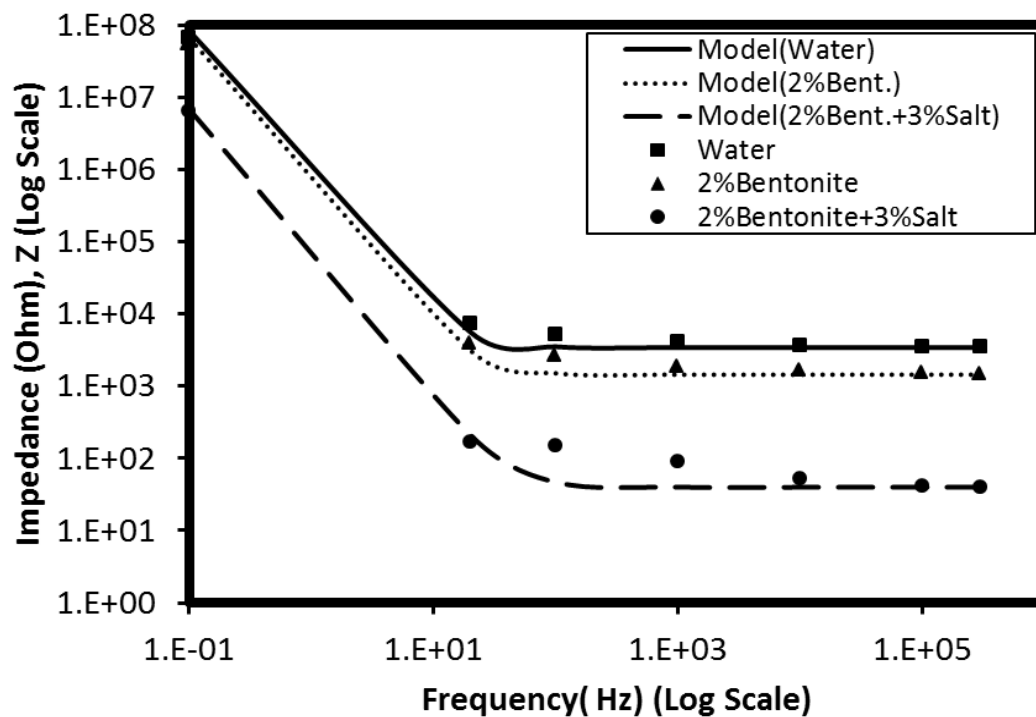


Figure 6.8 Impedance versus frequency modeling of drilling mud (2% bentonite) under different salt contamination.

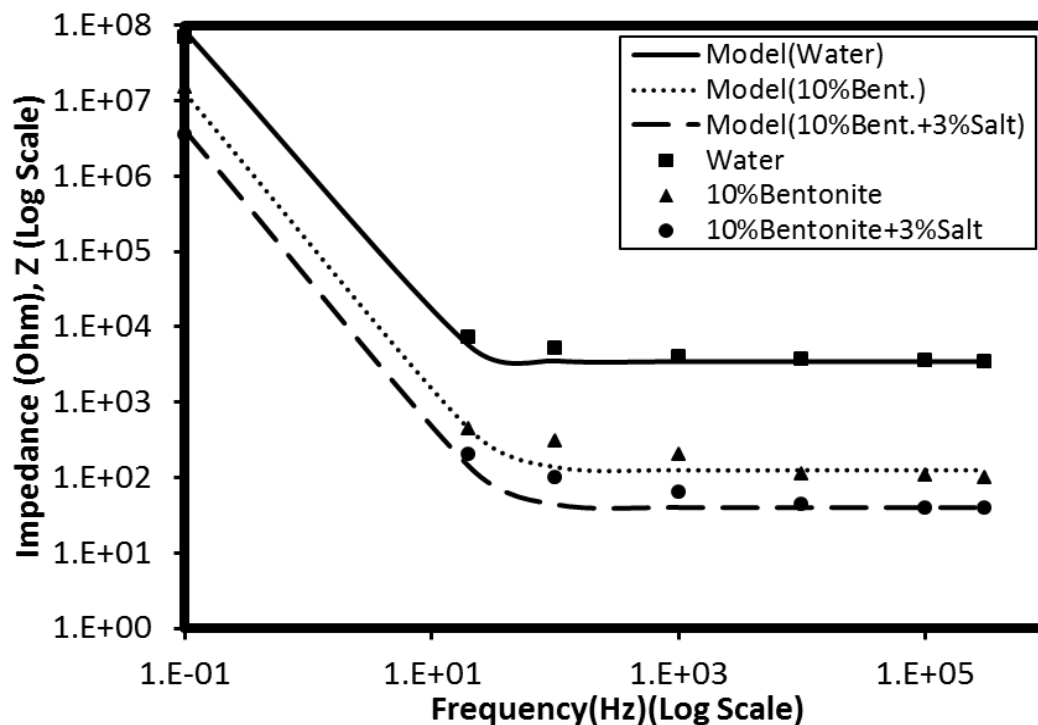


Figure 6.9 Impedance versus frequency modeling of drilling mud (10% bentonite) under different salt contamination.

(ii) Sensing the contamination during downward and upward flow

One of challenging practical issues is to sense the contamination regardless of the position of it with respect to drilling mud. Herein, the flow of salt contamination downward and upward have been studied through experimental models and compared as shown in Figure 6.10. The electrical resistivity versus time (almost 2 days) of 6% bentonite drilling mud under the impact of the downward and upward salt contamination have been investigated. The downward movement of salt was higher than the upward which clearly captured by the change in electrical resistivity of the drilling mud. The salt downward movement last 250% longer than the upward movement for stable resistance reading. From the recorded electrical resistivity at any time, the salt contamination content can be identified as shown in Figure 6.10.

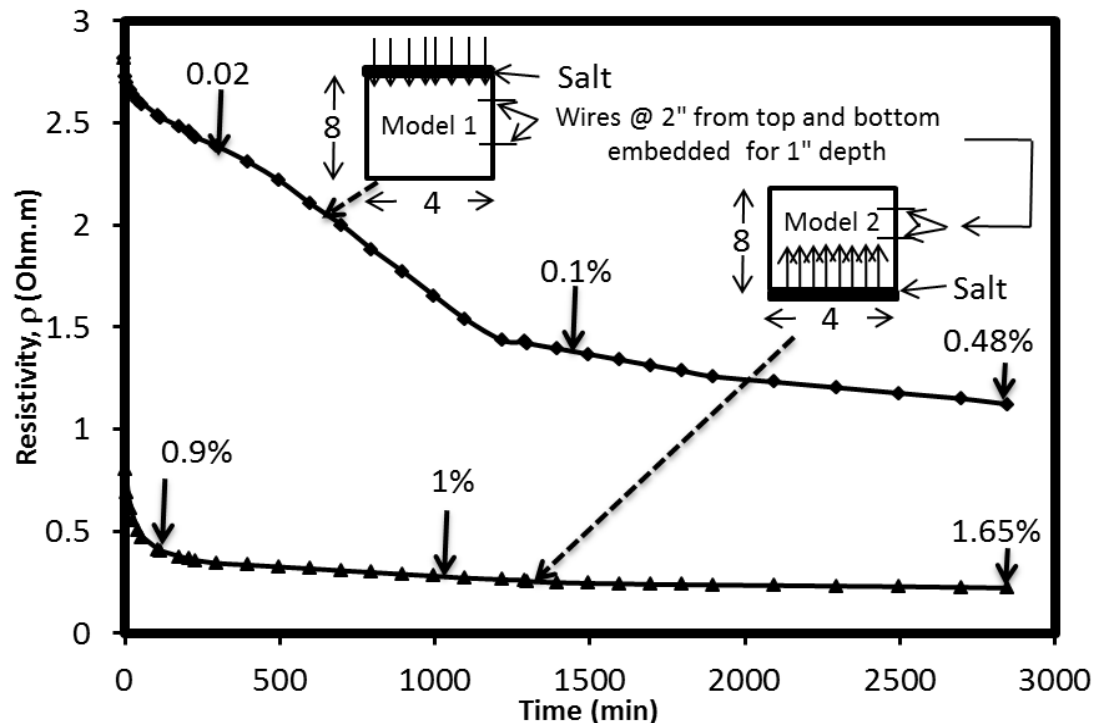


Figure 6.10 The variation of electrical resistivity change with time of 6% bentonite drilling mud in room temperature.

(iii) CIGMAT penetrometer

The CIGMAT penetrometer penetration versus salt content of 2% and 10% bentonite drilling mud under the weight of 7 gm CIGMAT penetrometer has been studied as shown in Figure 6.11. As the salt increased from 0% to 3%, the CIGMAT penetration of 2% and 10% drilling mud increased by 35% to 70% respectively. The variation of the CIGMAT penetrometer penetration versus its weight in different bentonite drilling mud with 3 salt contamination is shown in Figure 6.12. In Figure 6.12, the experimental data was modeled using the following

$$\delta(\text{mm}) = A_1 + B_1 * W(\text{gm}), \quad (6.1)$$

where A_1 (mm), and B_1 (mm/gm) are model parameters, W is the CIGMAT penetrometer weight. A summary of proposed correlation (Eq. 6.1) details for contaminated drilling mud with 3% salt is summarized in Table 6.1.

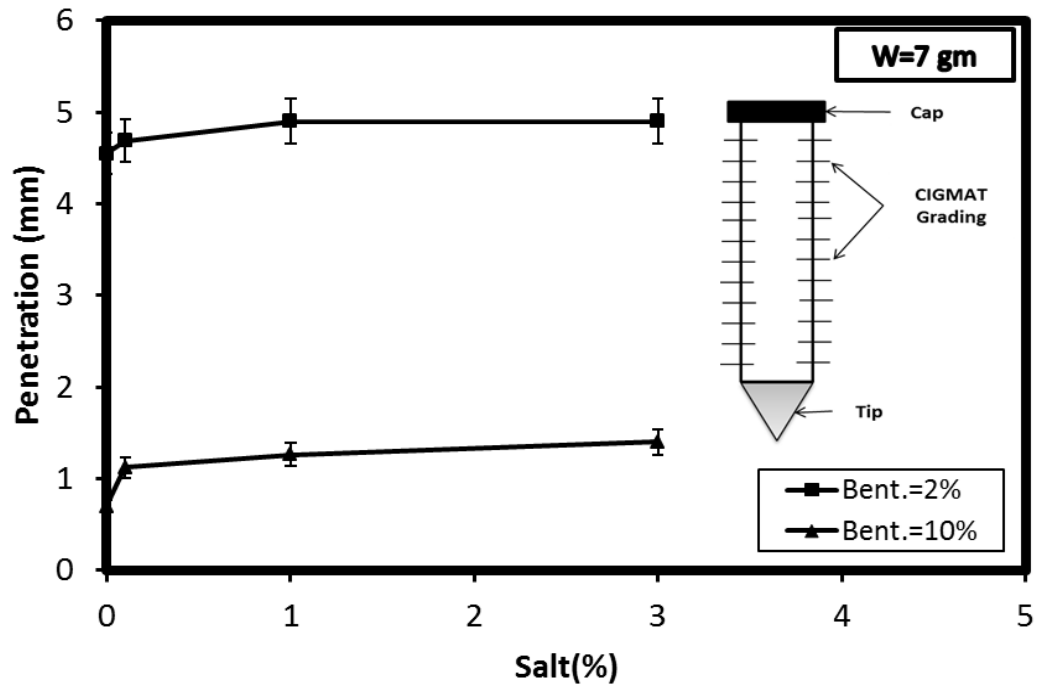


Figure 6.11 The variation of CIGMAT penetration versus salt content of different bentonite content ($W_1=7$ gm).

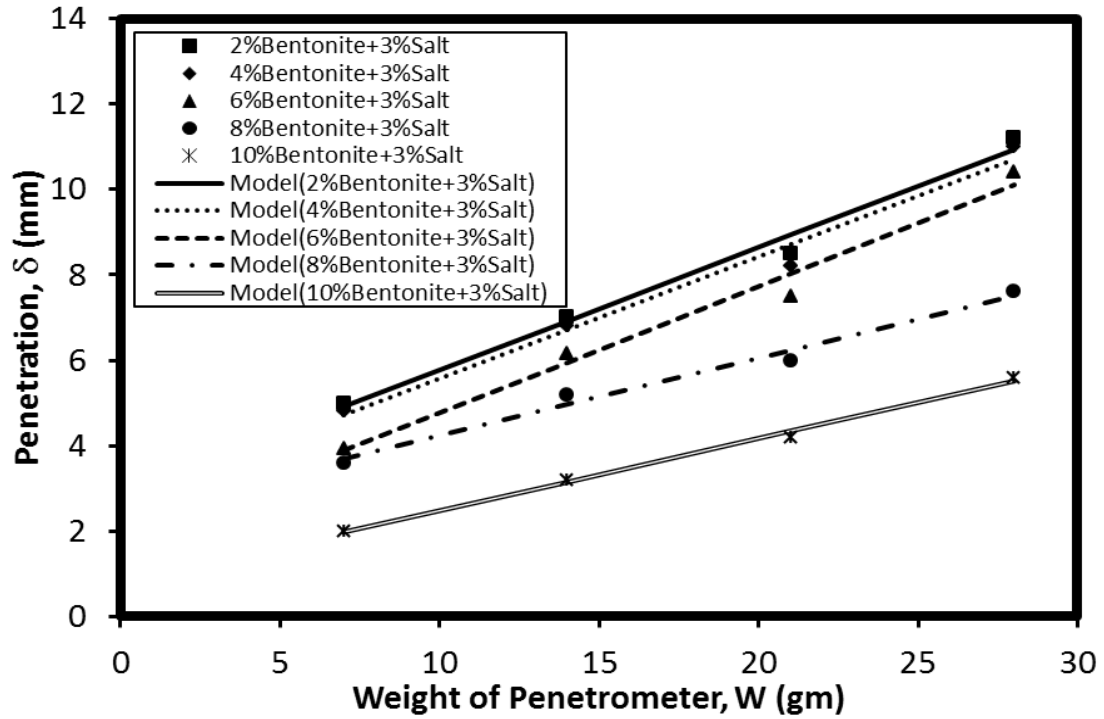


Figure 6.12 The variation of CIGMAT penetrometer penetration versus its weight in different bentonite content with 3% salt contamination content.

Table 6.1 Correlation parameters (Eq. 6.1) of contaminated drilling mud with 3% salt.

Bentonite (%)	Salt (%)	A ₁ (mm)	B ₁ (mm/gm)	R2	RMSE (mm)
2	3	2.9	0.29	0.99	0.258
4	3	2.7	0.28	0.98	0.302
6	3	1.8	0.30	0.98	0.320
8	3	2.4	0.18	0.98	0.178
10	3	0.8	0.17	0.99	0.086

Both parameters A_1 and B_1 for the CIGMAT penetrometer penetration-weight relationship have nonlinear correlation with material properties such as bentonite and salt contents. A_1 and B_1 correlations can be represented as

$$A_1 = 0.56 * (\text{Bent. \%})^{-0.57} + 0.35 * (\text{Bent. \%})^{-0.49} * (\text{Salt\%})^{-0.07} \text{ and} \quad (6.2)$$

$$B_1 = -0.14 * (\text{Bent. \%})^{-0.5} - 0.06 * (\text{Bent. \%})^{0.08} * (\text{Salt\%})^{0.02}. \quad (6.3)$$

Both predictions of Eqs.(6.2 and 6.3) can be compared with the experimental data as shown in Figure 6.13 and Figure 6.14 respectively. A good prediction can be concluded for both Eqs. (6.2 and 6.3) with a coefficient of correlation (R^2) up to 0.96.

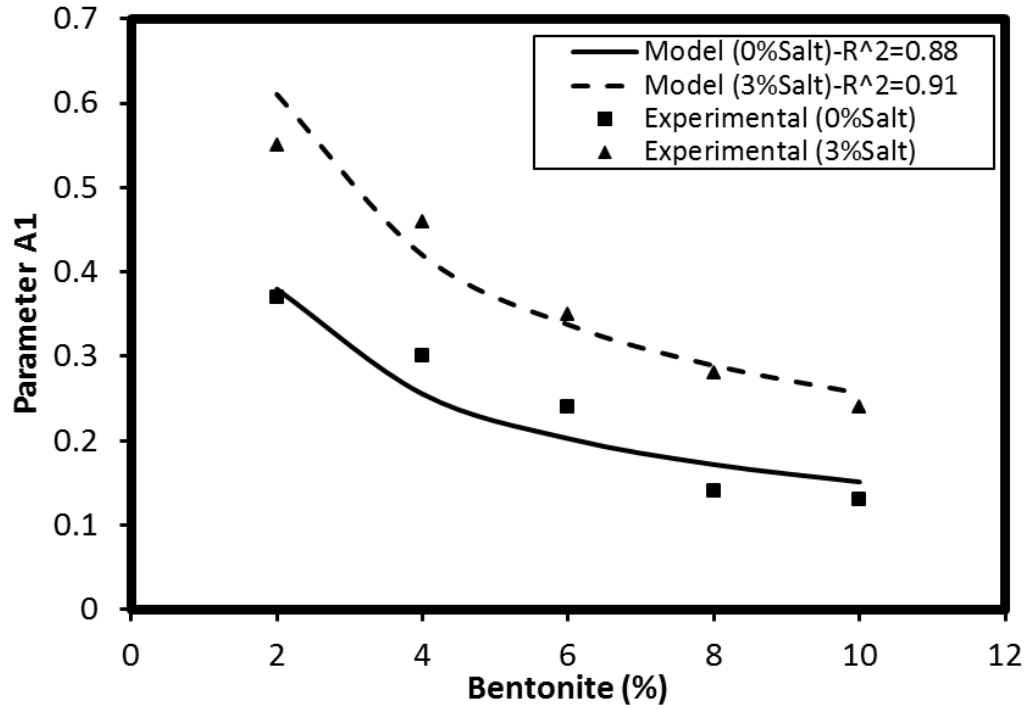


Figure 6.13 Correlation of parameter A_1 with the bentonite and salt content compared with the experimental results.

(iv) Model

The following mathematical model was used to predict the changes in the electrical resistivity of drilling mud due to salt contamination

$$\left(\frac{\Delta\rho}{\rho_o}\right) - \left(\frac{\Delta\rho}{\rho_o}\right)_o = \frac{\text{Salt}(\%)}{A_2 + B_2 * \text{Salt}(\%)}, \quad (6.4)$$

where A_2 and B_2 are model parameters depend on the bentonite contents, $(\Delta\rho/\rho_o)$ and $(\Delta\rho/\rho_o)_o$ are the changes in the electrical resistivity to the initial resistivity and the initial changes of the electrical resistivity, respectively. As the salt content increased from 0% to

3%, the changes in the electrical resistivity decreased by 98%, and 94% for 2% and 10% bentonite drilling mud respectively (Figure 6.15). The model parameters for Eq. 6.4 is summarized in Table 6.2. The provided model (Eq. 6.4) predicted the experimental data very well with a coefficient of correlation (R^2) up to 0.99.

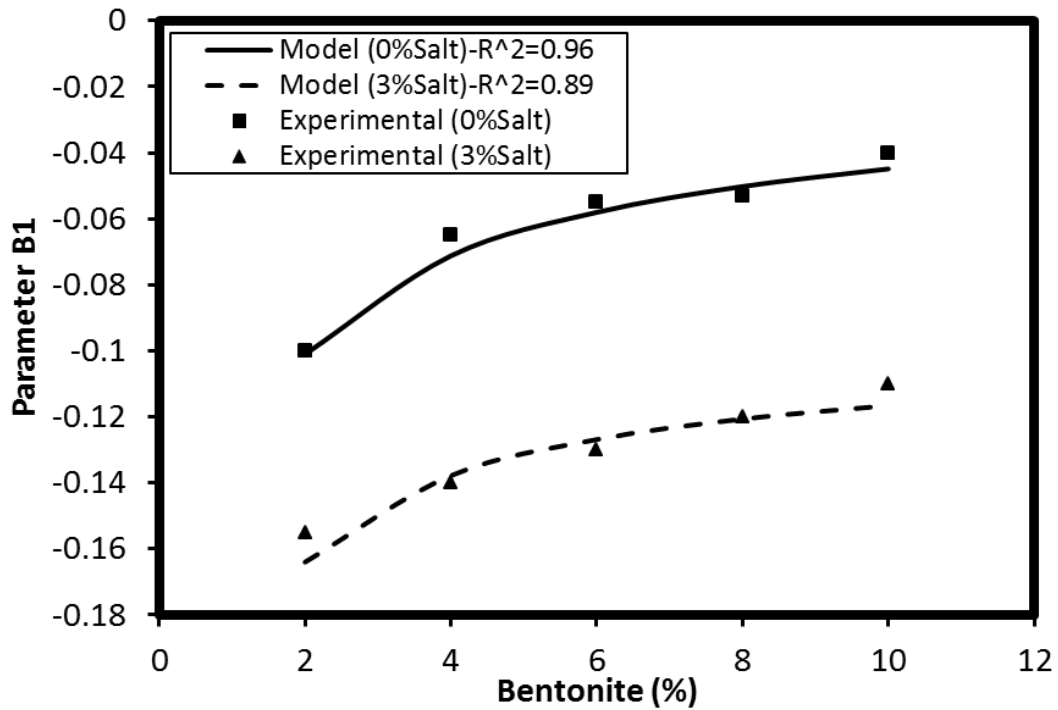


Figure 6.14 Correlation of parameter B_1 with the bentonite and salt content compared with the experimental results.

Table 6.2 Model parameters of Eq. (6.4)

Drilling Mud	Model Parameters			
	A_2	B_2	R^2	RMSE (%)
2% Bent.	-0.00425	-0.01	0.97	6.9494
10% Bent.	-0.03	-0.16	0.99	0.0755

Both A_2 and B_2 parameters of the electrical resistivity model (Eq.6.4) can be correlated to the bentonite content in the drilling mud as

$$A_2 = -0.004 * \text{Bent. \%} + 0.005 \quad (6.5)$$

$$B_2 = -0.02 * \text{Bent. \%} + 0.04. \quad (6.6)$$

Model predictions for both Eqs. (6.5) and (6.6) can be shown in Figure 6.16 and Figure 6.17 respectively. In addition, another mathematical model was proposed to predict the relationship between the shear strength of the soft soil and salt content as

$$\text{Shear strength (kPa)} - S_{uo}(\text{kPa}) = \frac{\text{Salt (\%)}}{A_3 + B_3 * \text{Salt (\%)}} \quad (6.7)$$

where A_3 (1/kPa) and B_3 (1/kPa) are model parameters, S_{uo} is the initial shear strength. As the salt content increased from 0% to 3%, the shear strength decreased by 75%, and 22% for 2% and 10% bentonite drilling mud respectively (Figure 6.18). The model parameters for Eq. 6.7 is summarized in Table 6.3. The provided model (Eq.6.7) predicted the experimental data very well with a coefficient of correlation (R^2) up to 0.95.

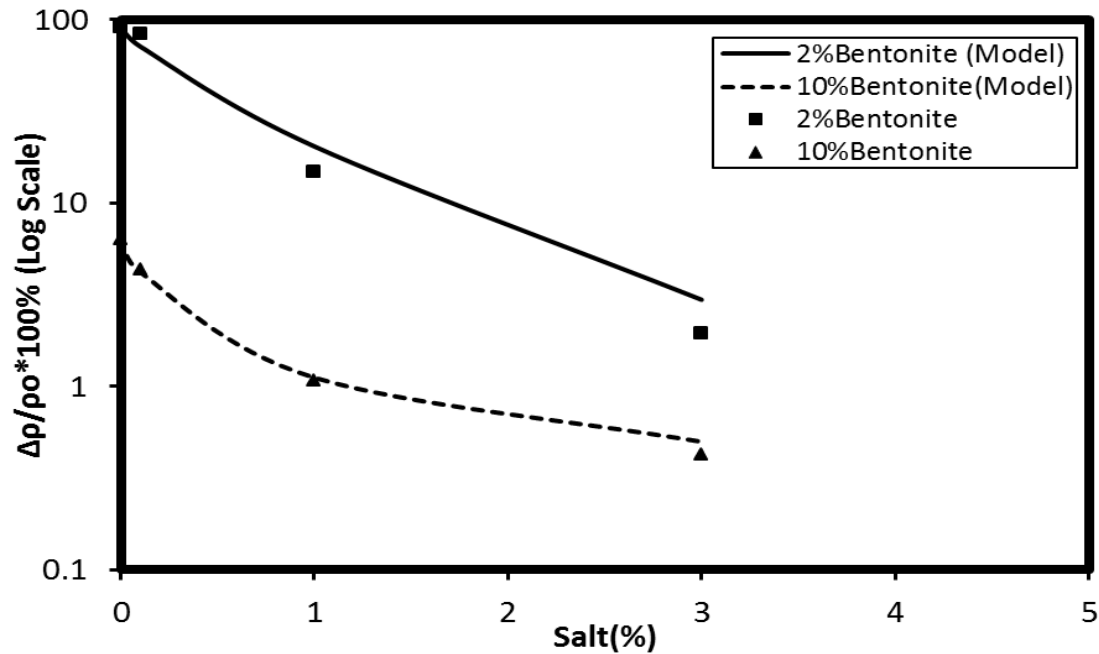


Figure 6.15 Salt effect on the resistivity change of the drilling mud with different bentonite content.

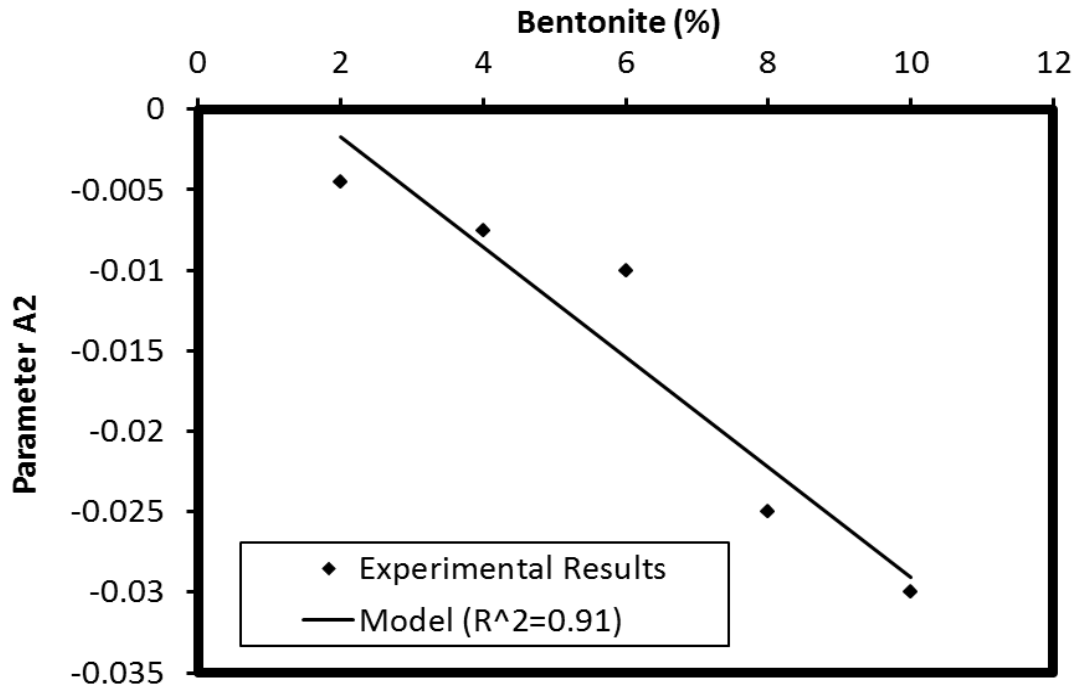


Figure 6.16 Correlation of parameter A_2 with the bentonite content compared with the experimental results.

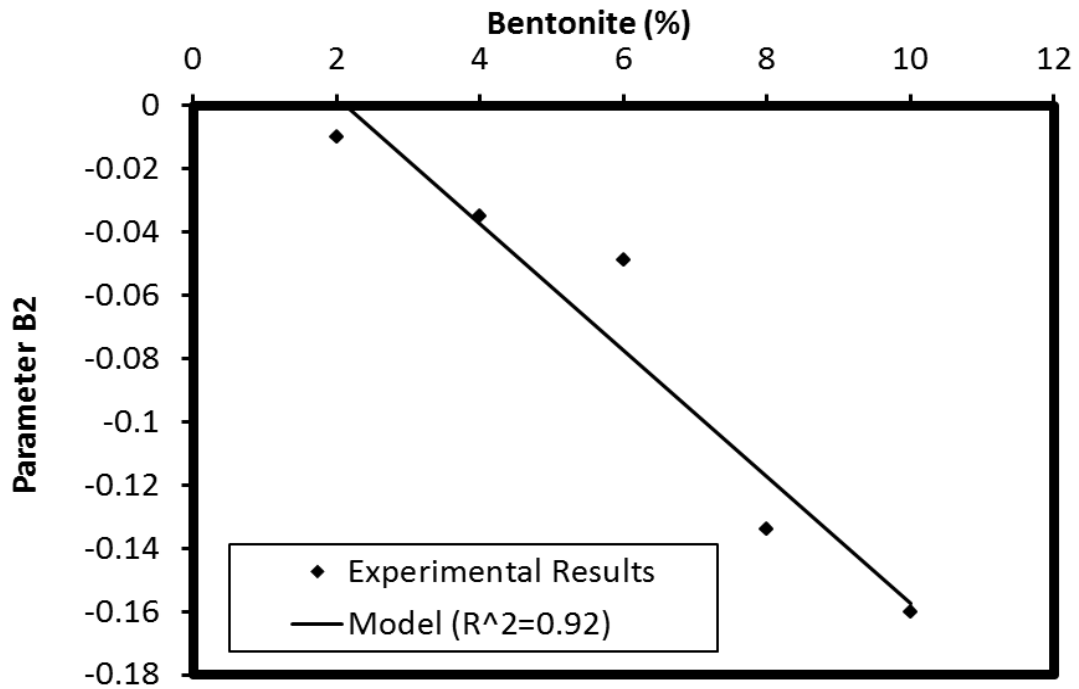


Figure 6.17 Correlation of parameter B_2 with the bentonite content compared with the experimental results.

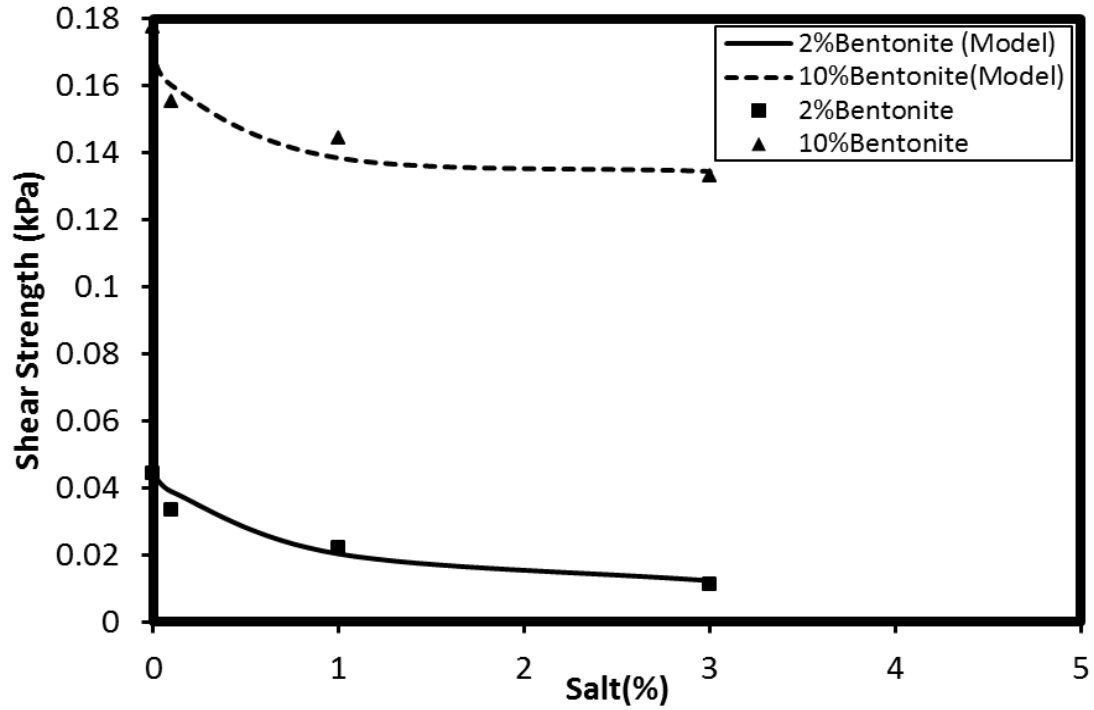


Figure 6.18 Salt effect on the shear strength of the drilling mud with different bentonite content.

Table 6.3 Model parameters of Eq. (6.7)

Drilling Mud	Model Parameters			
	A_3 (1/kPa)	B_3 (1/kPa)	R^2	RMSE (kPa)
2% Bent.	-15.5	-26	0.94	0.003
10% Bent.	-3.5	-22	0.95	0.003

Both A_3 and B_3 parameters of the shear strength model (Eq.6.7) can be correlated to the bentonite content in the drilling mud as

$$A_3 = -0.29 * \text{Bent. \%} - 1.23 \text{ and} \quad (6.8)$$

$$B_3 = 0.48 * \text{Bent. \%} - 26.35. \quad (6.9)$$

Model predictions for both Eqs. (6.8) and (6.9) can be shown in Figure 6.19 and Figure 6.20 respectively.

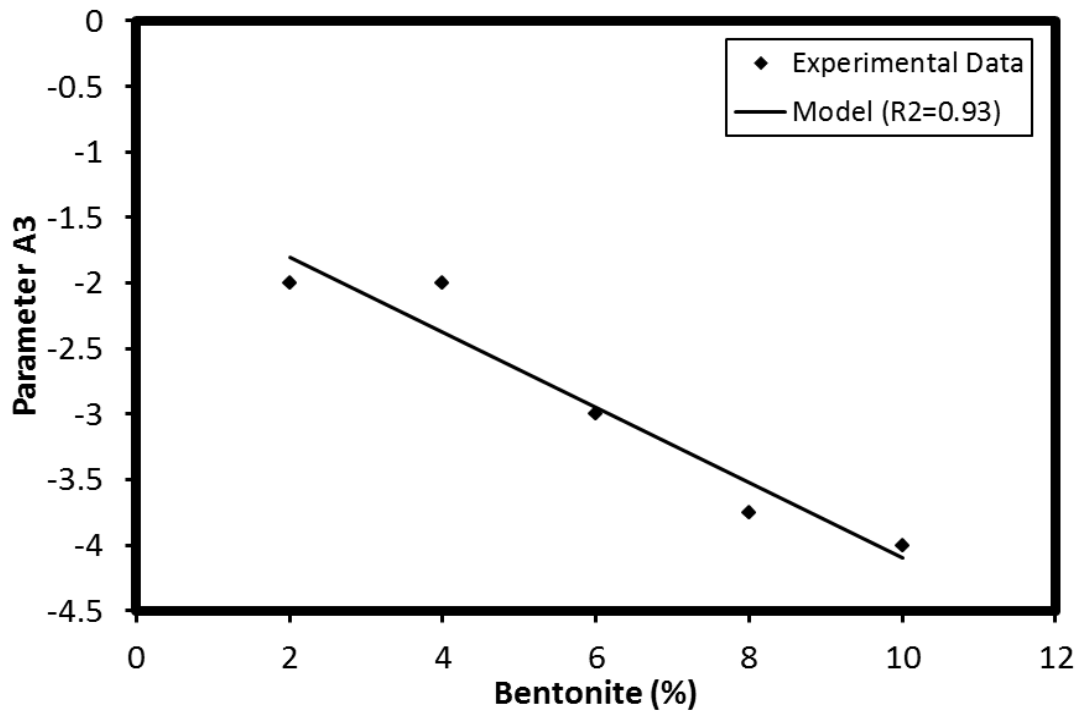


Figure 6.19 Correlation of parameter A_3 with the bentonite content compared with the experimental results.

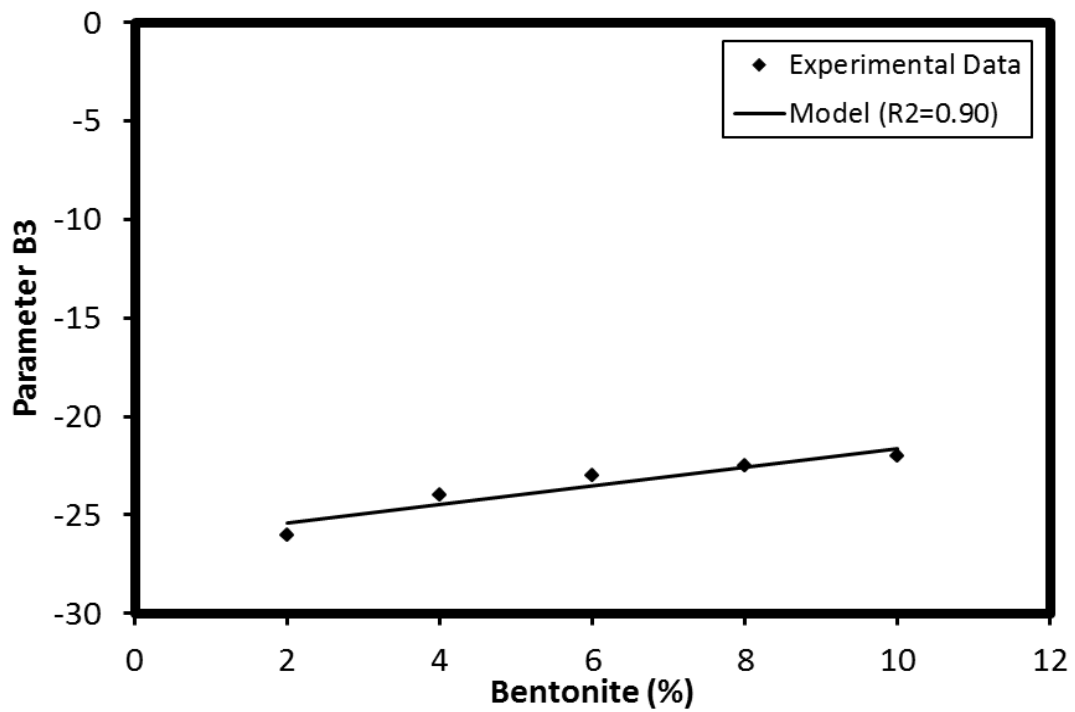


Figure 6.20 Correlation of parameter B_3 with the bentonite content compared with the experimental results.

(v) Statistical analysis

In this section, a statistical analysis has been performed on the shear strength model, electrical resistivity model and CIGMAT penetrometer penetration model using four different strategies as follows (El-Sakhawy et al., 2008):

1. Regression Analysis.
2. Error Estimation Analysis.
3. Classical Statistical Method.
4. Cumulative Probability Function.

In each of the above methods, the accuracy of model prediction has been evaluated and the best ranking was given to the more accurate one; then followed by the overall ranking procedure which summed up all the ranking numbers for each one from every single method and the lowest accumulative number was ranked as the best one.

Based on the results shown in Table 6.4, Table 6.6, Table 6.7, Table 6.8, Table 6.9, and Table 6.10; the regression analysis alone might be misleading in terms of best data modeling where it could give a good agreement with the experimental data while the real error estimation might be high in parallel. It is necessary and required to have more than one statistical method for best data modeling procedure. Through overall ranking procedure, the real and accurate data modeling can be established since it is an accumulative result of several applied statistical methods. The shear strength-salt model is analyzed as shown in Table 6.4. Based on the statistical analysis results, the shear strength-salt model had best prediction for 6% and 10% bentonite content and lowest prediction for 2% bentonite content. The electrical resistivity-salt model is analyzed as

shown in Table 6.5. Based on the statistical analysis results, the electrical resistivity-salt model had best prediction for 10% bentonite content and lowest prediction for 2% bentonite content. The CIGMAT penetrometer penetration-weight model for 2%, 4%, 6%, 8%, and 10% are analyzed as shown in Table 6.6, Table 6.7, Table 6.8, Table 6.9, and Table 6.10 respectively. For 2% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 1% salt content and lowest prediction for 0% salt content. For 4% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 0% salt content and lowest prediction for 0.1% salt content. For 6% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 1% salt content and lowest prediction for 0.1% salt content. For 8% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 1% salt content and lowest prediction for 0% and 3% salt content. For 10% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 0.1% salt content and lowest prediction for 1% salt content.

Table 6.4 Summary of statistical analysis of shear strength model.

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R^2	r_1 Rank	RMSE	r_2 Rank	Mean of T	r_3 Rank	T at P_{50}	r_4 Rank	r_g gross Rank	E
2	0-3	10	0.942	5	0.00301	1	1.0484	5	1.0555	5	16	4
4	0-3	10	0.985	1	0.00322	2	1.0203	4	1.0357	4	11	2
6	0-3	10	0.959	2	0.00337	3	1.0028	2	1.0028	2	9	1
8	0-3	10	0.946	4	0.00384	5	1.0071	3	1.0071	3	15	3
10	0-3	10	0.947	3	0.00382	4	0.9997	1	0.9997	1	9	1

Table 6.5 Summary of statistical analysis for electrical resistivity model.

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
2	0-3	15	0.969	5	6.94944	5	1.187	4	1.1846	4	18	4
4	0-3	15	0.984	3	1.35078	4	1.0709	2	1.0303	1	10	2
6	0-3	15	0.983	4	1.04533	3	1.2169	5	1.2169	5	17	3
8	0-3	15	0.994	2	0.21677	2	1.0969	3	1.0969	3	10	2
10	0-3	15	0.999	1	0.07560	1	1.0471	1	1.0471	2	5	1

Table 6.6 Summary of statistical analysis for CIGMAT penetrometer (2% bentonite).

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
2	0	20	0.8975	1	0.79855	4	1.0861	4	1.1122	4	13	4
2	0.1	20	0.8971	2	0.7837	2	1.081	3	1.1062	3	10	3
2	1	20	0.8970	3	0.74964	1	1.0722	2	1.0722	2	8	1
2	3	20	0.8885	4	0.78415	3	1.0673	1	1.0673	1	9	2

Table 6.7 Summary of statistical analysis for CIGMAT penetrometer (4% bentonite).

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
4	0	20	0.9525	1	0.56663	1	1.0549	1	1.0802	3	6	1
4	0.1	20	0.933	3	0.64001	4	1.0617	4	1.0809	4	15	4
4	1	20	0.9349	2	0.60867	2	1.0563	3	1.0563	2	9	2
4	3	20	0.9297	4	0.6192	3	1.0552	2	1.0552	1	10	3

Table 6.8 Summary of statistical analysis for CIGMAT penetrometer (6% bentonite).

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
6	0	20	0.9524	4	0.45096	1	1.0156	1	1.03	4	10	2
6	0.1	20	0.9545	3	0.45427	2	1.0198	3	1.0249	3	11	3
6	1	20	0.9548	2	0.46565	4	1.0174	2	1.0174	1	9	1
6	3	20	0.9645	1	0.45731	3	1.0205	4	1.0205	2	10	2

Table 6.9 Summary of statistical analysis for CIGMAT penetrometer (8% bentonite).

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
8	0	20	0.952	3	0.33528	4	1.0336	4	1.0089	3	14	3
8	0.1	20	0.959	2	0.27875	2	1.0152	2	0.9946	2	8	2
8	1	20	0.995	1	0.08005	1	1.0006	1	1.0006	1	4	1
8	3	20	0.948	4	0.32554	3	1.016	3	1.016	4	14	3

Table 6.10 Summary of statistical analysis for CIGMAT penetrometer (10% bentonite).

Comp.			Regression Analysis		Error Estimation Analysis		Classical Statistical Analysis		Cumulative Probability Function		Overall Evaluation	
Bent. (%)	Salt (%)	No. of Data	R ²	r ₁ Rank	RMSE	r ₂ Rank	Mean of T	r ₃ Rank	T at P ₅₀	r ₄ Rank	r _g gross Rank	E
10	0	20	0.9207	2	0.3037	1	1.0863	4	1.1165	4	11	3
10	0.1	20	0.8998	3	0.34187	2	1.0354	1	1.0246	1	7	1
10	1	20	0.8948	4	0.38059	3	1.0573	3	1.0573	3	13	4
10	3	20	0.9286	1	0.41462	4	1.0541	2	1.0541	2	9	2

6.6. Rheological properties

The rheological properties initial shear stress or yield point (τ_o) and maximum shear stress (τ_{max}) of the drilling mud were measured using electrical viscometer with high speed range up to 600 rpm (1024 s^{-1}) and accuracy of $\pm 1.0\%$. Behavior of the drilling fluids at high shearing rate is important to better model the drilling mud behavior with the maximum shear stress tolerance. In this study, 36 tests were conducted to evaluate the effect of salt contamination and temperature on both yield and maximum shear stress of the drilling mud.

(i) Modeling

The drilling muds showed non-linear shear thinning behavior with a yield stress. Based on the test results, following conditions have to be satisfied for the model to represent the observed behavior. Hence, the conditions are as

$$\tau = \tau_o \text{ when } \dot{\gamma} = 0,$$

$$\frac{d\tau}{d\dot{\gamma}} > 0, \quad (6.10)$$

$$\frac{d^2\tau}{d\dot{\gamma}^2} < 0, \text{ and} \quad (6.11)$$

$$\dot{\gamma} \rightarrow \infty \Rightarrow \tau = \tau^*. \quad (6.12)$$

The rheological models used for predicting the shear thinning behavior of drilling mud are as follows:

1- Herschel-Bulkley (H-B) rheological model

The Bingham plastic model includes both yield stress (τ_o) and a limiting viscosity (μ_p) at finite shear rates. For a nonlinear flow relationship shear thinning or shear thickening behavior may be observed and the assumption of constant plastic viscosity is not valid. The Herschel-Bulkley model (Tang and Kalyon 2004) (Eq. (6.13)) defines a fluid with three parameters and can be represented mathematically as

$$\tau = \tau_{o1} + k_1 \dot{\gamma}^n, \quad (6.13)$$

where τ , τ_{o1} , $\dot{\gamma}$, k_1 and n represent the shear stress (Pa), yield stress (Pa), shear strain rate (1/s), correction parameter and flow behavior index respectively. If the material remains rigid, then the model assumes that below the yield stress (τ_{o1}), the slurry behaves as a rigid solid, similar to Bingham plastic model. If the material flows as a Power law fluid, then the exponent n describes the shear thinning and shear thickening behavior. Slurries

are considered as shear thinning when $n < 1$ and shear thickening when $n > 1$. A fluid becomes shear thinning when the apparent viscosity decreases with the increase in shear strain rate. Hence, the model should satisfy the following conditions (Eq. (6.10), (6.11) and (6.12))

$$\frac{d\tau}{d\dot{\gamma}} = k_1 n \dot{\gamma}^{(n-1)} > 0 \Rightarrow k_1 n > 0 \text{ and} \quad (6.14)$$

$$\frac{d^2\tau}{d\dot{\gamma}^2} = k_1 n(n-1) \dot{\gamma}^{(n-2)} > 0 \Rightarrow k_1 n(n-1) < 0. \quad (6.15)$$

As shown in Eqs. (6.14) and (6.15), one condition when both Eqs. (6.10) and (6.11) will be satisfied is as

$$0 < n < 1 \text{ and } k_1 > 0.$$

From Eq. (6.13), when $\dot{\gamma} \rightarrow \infty \Rightarrow \tau_{\max} = \infty$. Hence, Herschel-Bulkley model does not satisfy the upper limit condition for the shear stress limit.

2- Hyperbolic rheological model

The relationship between shear stress with shear strain rate of contaminated bentonite drilling mud with salt under different temperatures was investigated. Based on the inspection of the test data following hyperbolic relationship is proposed (Usluogullari and Vipulanandan, 2011; and Vipulanandan and Mohammed, 2014)

$$\tau - \tau_{o2} = \frac{\dot{\gamma}}{A_4 + B_4 \dot{\gamma}}, \quad (6.16)$$

where τ , τ_{o2} , $\dot{\gamma}$, A_4 and B_4 represent the shear stress (Pa), yield stress (Pa), shear strain rate (s^{-1}), and A ($Pa \cdot s$)⁻¹ & B (Pa)⁻¹ model parameters respectively. The first and second derivatives of hyperbolic model are as

$$\frac{d\tau}{d\dot{\gamma}} = \frac{(A_4 + B_4\dot{\gamma}) - \dot{\gamma} * B_4}{(A_4 + B_4\dot{\gamma})^2} = \frac{A_4}{(A_4 + B_4\dot{\gamma})^2} > 0 \Rightarrow A_4 > 0 \text{ and}$$

$$\frac{d^2\tau}{d\dot{\gamma}^2} = \frac{-2A_4B_4}{(A_4 + B_4\dot{\gamma})^3} < 0 \Rightarrow B_4 > 0.$$

Also when $\dot{\gamma} \rightarrow \infty \Rightarrow \tau_{\max} = \frac{1}{B_4} + \tau_{o2}$. Hence, this model has a limit on the maximum shear stress the fluid will produce at a relatively high rate of shear strains.

(ii) Hyperbolic constitutive relationships

1- Yield shear stress

The yield shear stress versus salt content of 2% and 8% bentonite drilling mud under different temperatures 25° C to 75° C were modeled through Eq.6.17

$$\tau_Y - \tau_{Y0} = \frac{\text{Salt (\%)}}{R(T, \text{Bent}) + S(T, \text{Bent}) * \text{Salt (\%)}} \quad (6.17)$$

where τ_{Y0} (Pa) & τ_Y (Pa) are initial and evaluated yield shear stress as a function of temperature respectively, R (Pa)⁻¹ & S (Pa)⁻¹ are model parameters as a function of bentonite content and temperature. The number of data used in this analysis was 22.

2- Maximum shear stress

The maximum shear stress versus salt content of 2% and 8% bentonite drilling mud under different temperatures 25° C to 75° C were modeled through Eq.6.18 as

$$\tau_{\max} - \tau_{\max,0} = \frac{\text{Salt (\%)}}{G(T, \text{Bent}) + H(T, \text{Bent}) * \text{Salt (\%)}} \quad (6.18)$$

where $\tau_{\max,0}$ (Pa) and $\tau_{\max}$ (Pa) are initial and evaluated maximum shear stress as a function of temperature respectively, G (Pa)⁻¹ & H (Pa)⁻¹ are model parameters as a

function of bentonite content and temperature. The number of data used in this analysis was 22.

3- Electrical resistivity

Based on the experimental results, the electrical resistivity of 2% and 8% bentonite drilling mud under different salt contamination up to 3% at two different temperatures 25° C to 75° C was modeled using the hyperbolic type relationship (Eq.6.19)

$$\rho - \rho_o = \frac{\text{Salt (\%)}}{W_1(T, \text{Bent}) + X_1(T, \text{Bent}) * \text{Salt (\%)}} \quad (6.19)$$

where ρ_o (Ohm.m) and ρ (Ohm.m) are initial and evaluated electrical resistivity as functions of temperature respectively, W_1 (Ohm.m)⁻¹ & X_1 (Ohm.m)⁻¹ are model parameters as functions of temperature and bentonite content. The number of data used in this analysis was 36. For any individual relationship, such as yield shear stress (τ_Y) versus salt content, the relationship was studied for a specific bentonite content and constant temperature then both model parameters (R and S) were optimized accordingly. The same procedure has been implemented for both maximum shear stress ($\tau_{\max}$) and electrical resistivity (ρ) versus salt content.

(iii) Nonlinear models (NLM)

1- Material and rheological

The electrical resistivity (ρ) is the material property and yield shear stress (τ_o), maximum shear stress ($\tau_{\max}$) and A_4 & B_4 parameters (Eq. 6.16) are the rheological properties of bentonite drilling mud that was influenced by the percentage of the bentonite content, tested temperature and salt content. Further extension of nonlinear

model that proposed by Vipulanandan and Mohammed (2015) has been used to model parameters to the independent variables (bentonite content, temperature and salt content) using a nonlinear power law relationship as

$$\rho, \tau_o, \tau_{max}, A_4 \& B_4 - \text{Parameters} = y * (\text{Bent. \%})^z + c * (\text{Bent. \%})^d * (\Delta T)^e + f * (\text{Bent. \%})^g * (\text{Salt \%})^h + i * (\text{Bent. \%})^j * (\Delta T)^k * (\text{Salt \%})^l, \quad (6.20)$$

where y, z, c, d, e, f, g, h, i, j, k and l are model parameters and ΔT is the change in temperature.

2- Electrical resistivity

Changes in rheological properties (τ_o and τ_{max}) and hyperbolic model parameters (A_4 and B_4 in Eq. 6.16) with temperature and salt change for the bentonite drilling mud were related to the electrical resistivity (ρ) as

$$\tau_o, \tau_{max}, A_4 \& B_4 - \text{Parameters} = P * (\rho)^M + Q * (\rho)^N, \quad (6.21)$$

where P, M, Q and N are model parameters.

(iv) Results and analyses

1- Impedance-spectroscopy characterization

Impedance versus frequency for 2% bentonite drilling mud with different salt contamination up 3% was measured and the results are shown Figure 6.21. Salt contamination decreased the measured electrical impedance for tested bentonite drilling mud content. All tested bentonite drilling mud with and without salt contamination showed pure resistance behavior at high measured frequency (300 kHz) where the effect of contact resistance is minor and negligible. As the salt contamination increased by 3 %,

the bulk resistivity decreased by 37 times. Similarly, impedance versus frequency for 8% bentonite drilling with different salt contamination up 3% was studied and the results are shown in Figure 6.22.

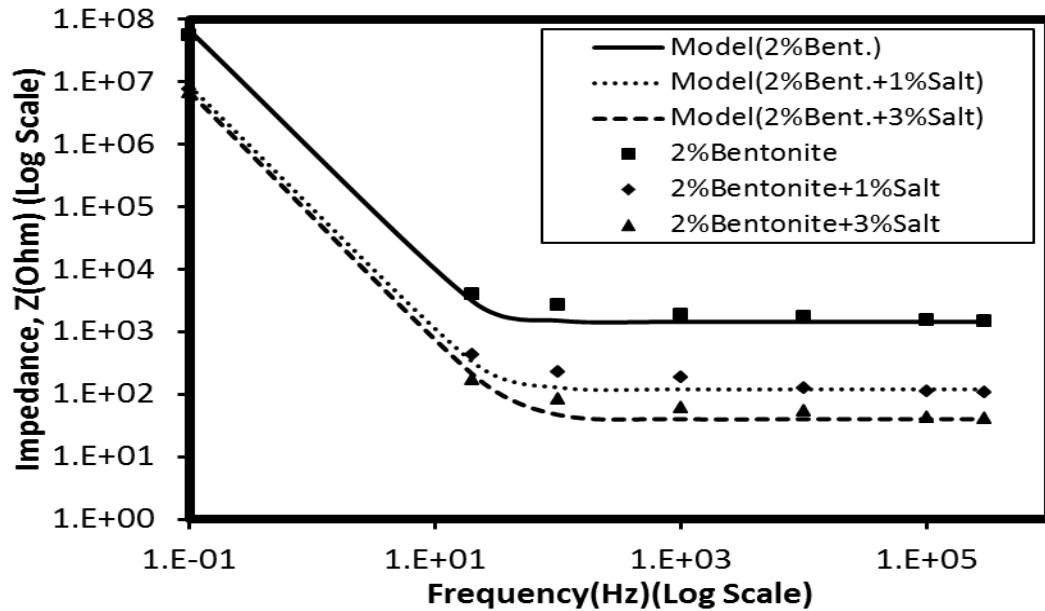


Figure 6.21 Impedance versus frequency modeling of drilling mud under different salt contamination (0%, 1% and 3%) for 2% bentonite.

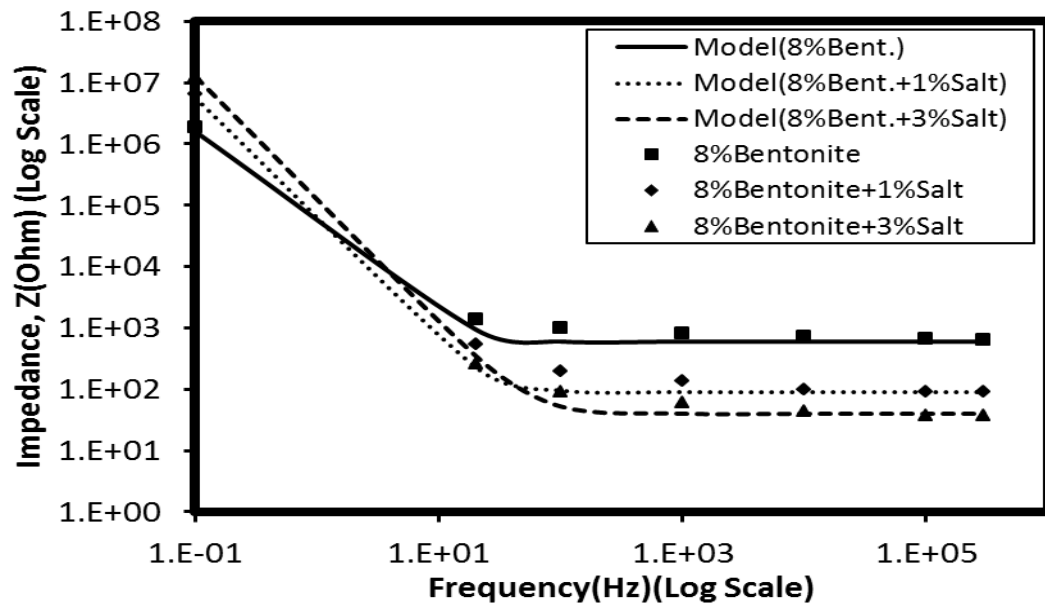


Figure 6.22 Impedance versus frequency modeling of drilling mud under different salt contamination (0%, 1% and 3%) for 8% bentonite.

The shape of the electrical impedance-frequency relationship was similar to the 2% bentonite mud. As the salt contamination increased by 3 %, the bulk resistance decreased by 17 times. A bulk resistivity decreased by 46% when the bentonite content increased from 2% to 8% with 3% salt contamination.

2- Characterization of rheological behavior

Based on the experimental results, the rheological responses of the water based drilling muds were modeled using the Herschel–Bulkley (H-B) model and hyperbolic model.

H-B rheological model

(i) 2% Bentonite content

0% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 0% salt contaminations is shown in Figure 6.23. As the temperature increased from 25° C to 75° C, the yield stress was the same (0.6 Pa) while the parameters k_1 and n values decreased from 0.2 (Pa.s) to 0.125 (Pa.s) and 0.52 to 0.5 respectively.

1% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 1% salt contaminations is shown in Figure 6.24. As the temperature increased from 25° C to 75° C, the yield stress increased from 0.1 Pa to 0.2 Pa while the parameters k_1 and n values reduced from 0.2 (Pa.s) to 0.15 (Pa.s) and 0.48 to 0.46 respectively.

3% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 3% salt contaminations is shown in Figure 6.25. As the temperature increased from 25° C to 75° C, the yield stress increased from 0.2 Pa to 0.5 Pa while the parameters k_1 and n values reduced from 0.19 (Pa.s) to 0.10 (Pa.s) and 0.46 to 0.45 respectively.

(ii) 8% Bentonite content

0% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 0% salt contaminations is shown in Figure 6.26. As the temperature increased from 25° C to 75° C, the yield stress was the same (35 Pa) while the parameters k_1 and n values reduced from 7 (Pa.s) to 5 (Pa.s) and 0.44 to 0.43 respectively.

1% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 1% salt contaminations is shown in Figure 6.27. As the temperature increased from 25° C to 75° C, the yield stress decreased from 4.4 Pa to 1.2 Pa while the parameter k_1 value increased from 5 (Pa.s) to 7 (Pa.s) and the parameter n value decreased from 0.40 to 0.27 respectively.

3% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 3% salt contaminations is shown in Figure 6.28. As the temperature increased from 25° C to 75° C, the yield stress increased from 1.2 Pa to 1.3 Pa while the parameters k_1 and n values reduced from 5 (Pa.s) to 3.2 (Pa.s) and 0.31 to 0.27 respectively.

Hyperbolic rheological model

(i) 2% Bentonite content

0% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 0% salt contaminations is shown in Figure 6.23. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 64 (Pa.s)⁻¹ to 160 (Pa.s)⁻¹ and the parameter- B_4 was the same. The maximum shear stress ($\tau_{\max}$) decreased from 8.9 Pa to 5.2 Pa when the temperature was increased from 25° C to 75° C respectively.

1% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 1% salt contaminations is shown in Figure 6.24. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 95 (Pa.s)⁻¹ to 180 (Pa.s)⁻¹ and the parameter- B_4 increased slightly from 0.067 (Pa)⁻¹ to 0.07 (Pa)⁻¹. The maximum shear stress ($\tau_{\max}$) decreased from 6.6 Pa to 4.4 Pa when the temperature was increased from 25° C to 75° C respectively. Adding

1% salt to the 2% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 26% and 15% at 25° C and 75° C respectively.

3% Salt

The shear stress-shear strain rate relationships for 2% bentonite drilling mud at two different temperatures 25° C and 75° C with 3% salt contaminations is shown in Figure 6.25. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 130 (Pa.s)⁻¹ to 280 (Pa.s)⁻¹ and the parameter- B_4 increased slightly from 0.066 (Pa)⁻¹ to 0.07 (Pa)⁻¹. The maximum shear stress ($\tau_{\max}$) decreased from 5.5 Pa to 3.5 Pa when the temperature was increased from 25° C to 75° C respectively. Adding 3% salt to the 2% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 38% and 33% at 25° C and 75° C respectively.

(ii) 8% Bentonite content

0% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 0% salt contaminations is shown in Figure 6.26. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 0.8 (Pa.s)⁻¹ to 1 (Pa.s)⁻¹ and the parameter- B_4 increased slightly from 0.007 (Pa)⁻¹ to 0.009 (Pa)⁻¹. The maximum shear stress ($\tau_{\max}$) decreased from 167.7 Pa to 124 Pa when the temperature was increased from 25° C to 75° C respectively.

1% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 1% salt contaminations is shown in Figure 6.27. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 1 (Pa.s)⁻¹ to 4 (Pa.s)⁻¹ and the parameter- B_4 was the same. The maximum shear stress ($\tau_{\max}$) decreased from 83.1 Pa to 43 Pa when the temperature was increased from 25° C to 75° C respectively. Adding 1% salt to the 8% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 50% and 65% at 25° C and 75° C respectively.

3% Salt

The shear stress-shear strain rate relationships for 8% bentonite drilling mud at two different temperatures 25° C and 75° C with 3% salt contaminations is shown in Figure 6.28. As the temperature increased from 25° C to 75° C, the parameter- A_4 value increased from 4 (Pa.s)⁻¹ to 11 (Pa.s)⁻¹ and the parameter- B_4 increased from 0.021 (Pa)⁻¹ to 0.033 (Pa)⁻¹. The maximum shear stress ($\tau_{\max}$) decreased from 44.5 Pa to 24.5 Pa when the temperature was increased from 25° C to 75° C respectively. Adding 3% salt to the 8% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 73% and 80% at 25° C and 75° C respectively. Model parameters for both Herschel-Bulkley and hyperbolic rheological model are summarized in Table 6.11.

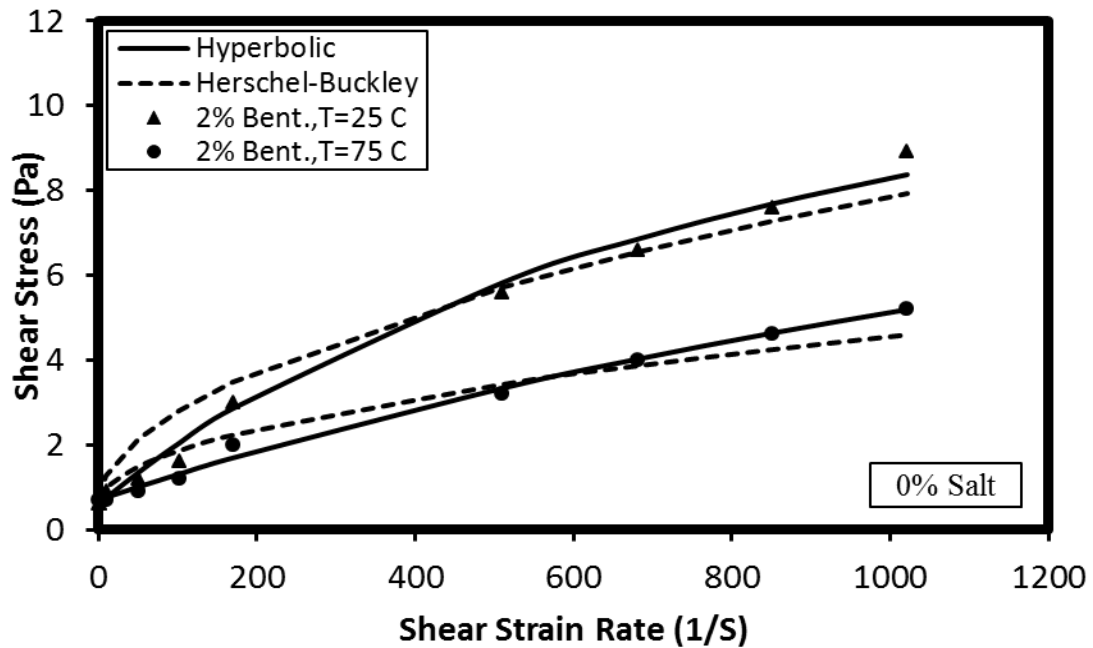


Figure 6.23 Shear stress-shear strain rate modeling of drilling mud (2% bentonite) at two different temperatures (25° C and 75° C) with 0% salt contamination.

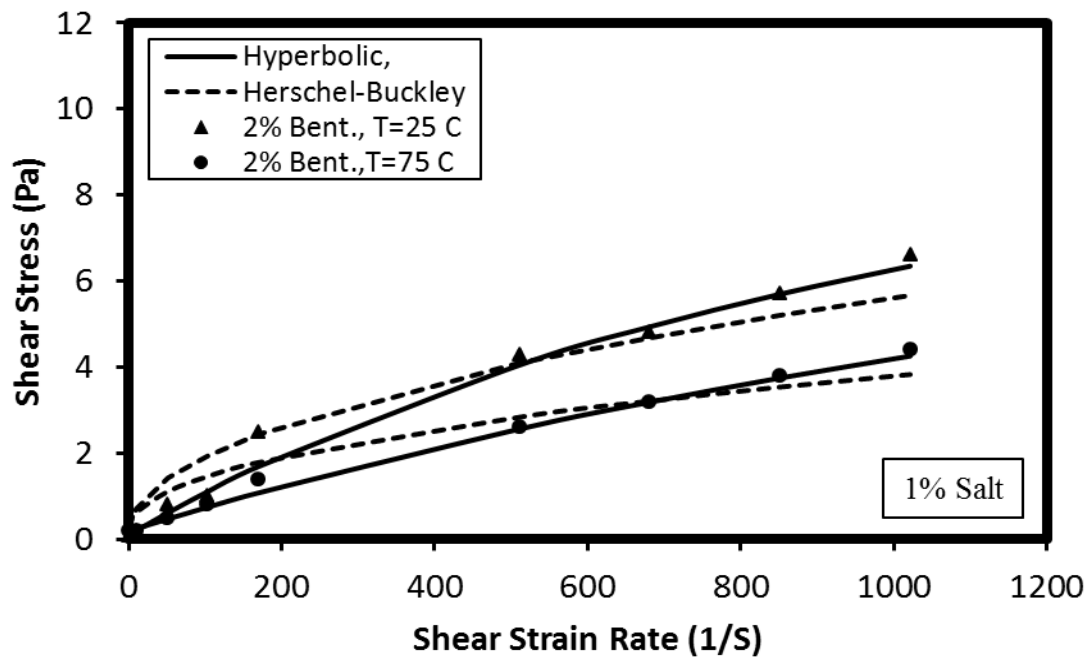


Figure 6.24 Shear stress-shear strain rate modeling of drilling mud (2% bentonite) at two different temperatures (25° C and 75° C) with 1% salt contamination.

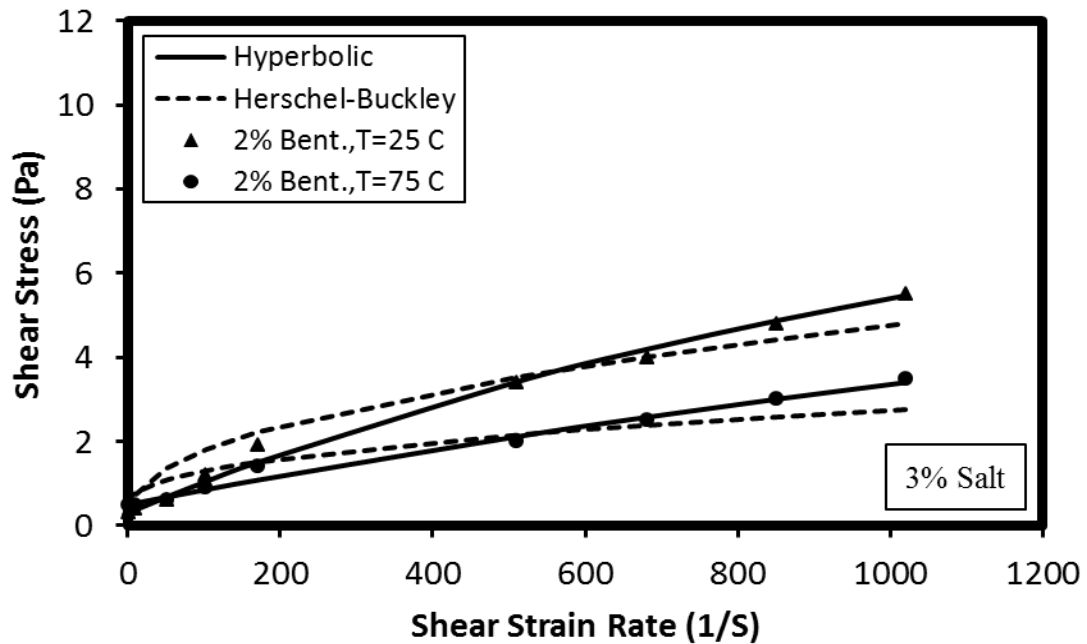


Figure 6.25 Shear stress-shear strain rate modeling of drilling mud (2% bentonite) at two different temperatures (25° C and 75° C) with 3% salt contamination.

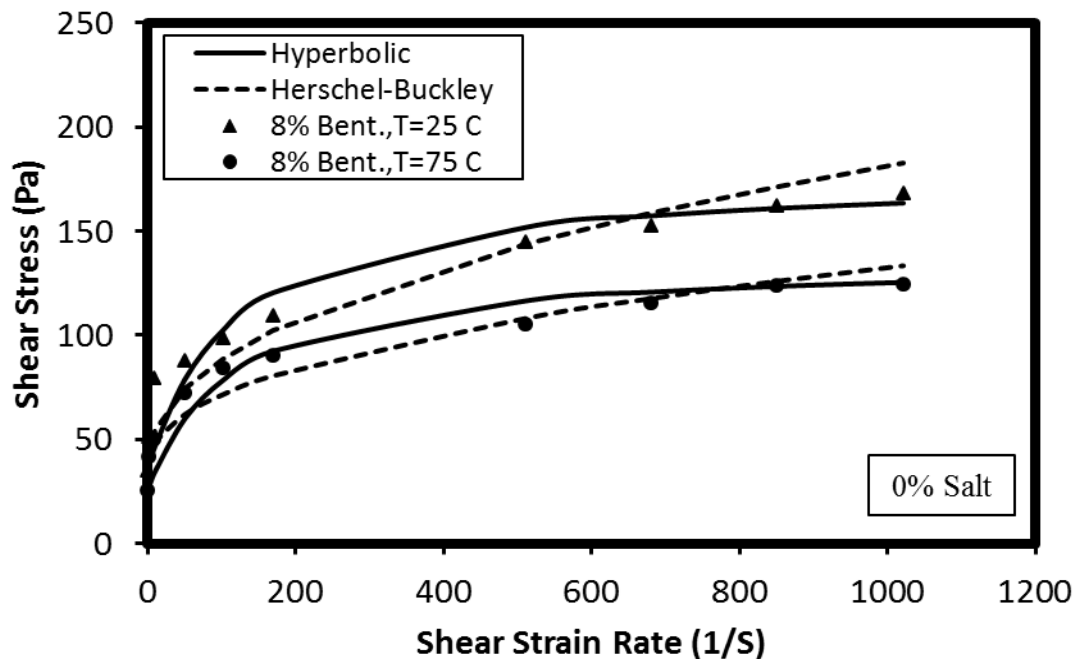


Figure 6.26 Shear stress-shear strain rate modeling of drilling mud (8% bentonite) at two different temperatures (25° C and 75° C) with 0% salt contamination.

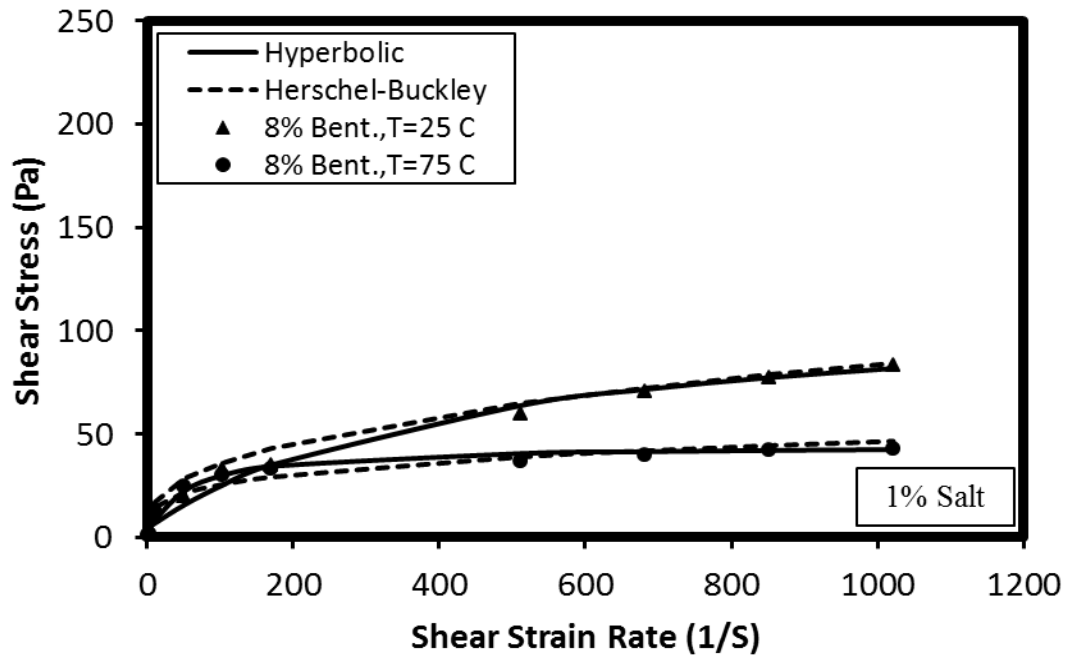


Figure 6.27 Shear stress-shear strain rate modeling of drilling mud (8% bentonite) at two different temperatures (25° C and 75° C) with 1% salt contamination.

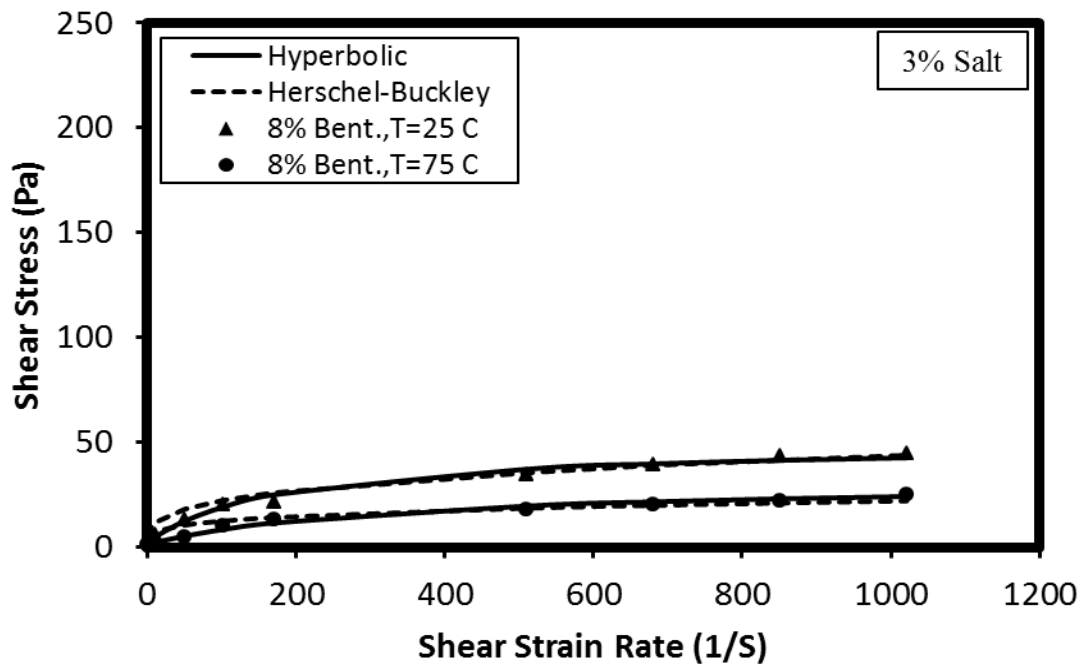


Figure 6.28 Shear stress-shear strain rate modeling of drilling mud (8% bentonite) at two different temperatures (25° C and 75° C) with 3% salt contamination.

Table 6.11 Herschel-Bulkley and hyperbolic rheological model parameters for bentonite drilling mud contaminated with salt under different temperatures.

Bent (%)	Salt (%)	Temp. (C)	Herschel-Bulkley model Eq. (6.13)					Hyperbolic model Eq. (6.16)				
			$\tau_{01}(\text{Pa})$	$k_1(\text{Pa.s})$	n	RMSE (Pa)	R^2	$\tau_{02}(\text{Pa})$	$A_4(\text{Pa.s})^{-1}$	$B_4(\text{Pa})^{-1}$	RMSE (Pa)	R^2
2	0	25	0.6	0.2	0.52	0.63	0.96	0.7	64	0.066	0.26	0.99
2	0	75	0.6	0.125	0.50	0.39	0.95	0.4	160	0.066	0.12	0.99
2	1	25	0.1	0.2	0.48	0.53	0.95	0.65	95	0.067	0.288	0.99
2	1	75	0.2	0.15	0.46	0.41	0.93	0.38	180	0.07	0.11	0.99
2	3	25	0.2	0.19	0.46	0.43	0.95	0.6	130	0.066	0.15	0.99
2	3	75	0.5	0.1	0.45	0.36	0.89	0.35	280	0.07	0.18	0.99
8	0	25	35	7	0.44	11.6	0.93	28	0.8	0.007	12.42	0.92
8	0	75	35	5	0.43	7.98	0.94	20	1	0.009	8.78	0.93
8	1	25	4.4	5	0.40	5.33	0.97	4.4	4	0.009	3.53	0.99
8	1	75	1.2	7	0.27	4.24	0.93	1.2	1.2	0.023	2.19	0.98
8	3	25	1.2	5	0.31	3.51	0.95	2.5	4	0.021	1.77	0.99
8	3	75	1.3	3.2	0.27	3.22	0.86	1.1	11	0.033	1.11	0.98

3- Hyperbolic constitutive relationship

Rheological

(i) Yield shear stress

2% Bentonite content

As the salt contamination and temperature increased, the yield shear stress decreased for the 2% bentonite drilling mud (Figure 6.29). When the salt contamination in 2% bentonite drilling mud increased from 0% to 3%, the yield shear stress decreased by 39% and 35% at 25° C and 75° C respectively. When the temperature was increased from 25° C to 75° C, the yield shear stress decreased by 42% for the 2% bentonite drilling mud with 0% salt contamination. When the temperature was increased from 25° C to 75° C, the yield shear stress decreased by 34% and 37% for the 2% bentonite drilling mud contaminated with 1% and 3% salt contents respectively.

8% Bentonite content

As the salt contamination in 8% bentonite drilling mud increased from 0% to 3%, the yield shear stress decreased by 74% and 84% at 25° C and 75° C respectively (Figure 6.30). As the temperature was increased from 25° C to 75° C, the yield shear stress decreased by 24% for 8% bentonite drilling mud with 0% salt contamination. As the temperature was increased from 25° C to 75° C, the yield shear stress decreased by 46% and 51% for 8% bentonite drilling mud contaminated with 1% and 3% salt contents respectively.

Model parameters correlation

The three model parameters $(\tau_Y)_o$, R and S parameters were correlated with temperature and bentonite contents as follows (Eqs. 6.22, 6.23 and 6.24)

$$(\tau_Y)_o = Y_1 + Y_2 * (\text{Bent.}) + Y_3 * (T) + Y_4(\text{Bent.})(T), \quad (6.22)$$

$$R = R_1 + R_2 * (\text{Bent.}) + R_3 * (T) + R_4 * (\text{Bent.}) * (T), \text{ and} \quad (6.23)$$

$$S = S_1 + S_2 * (\text{Bent.}) + S_3 * (T) + S_4 * (\text{Bent.}) * (T), \quad (6.24)$$

where Bent. and T are bentonite content and absolute temperature respectively and Y_1 to Y_4 , R_1 to R_4 and S_1 to S_4 are model parameters.

The coefficient of correlation (R^2) for 2% bentonite drilling mud at 25° C and 75° C were 0.98 and 0.98 respectively. The RMSE for 2% bentonite drilling mud at 25° C and 75° C were 0.0054 Pa and 0.0029 Pa respectively. The coefficient of correlation (R^2) for 8% bentonite drilling mud at 25° C and 75° C were 0.98 and 0.99 respectively. The RMSE for 8% bentonite drilling mud at 25° C and 75° C were 0.1633 Pa and 0.2369 Pa respectively. Model parameters of Eqs. 6.22 to 6.24 are given in Table 6.12.

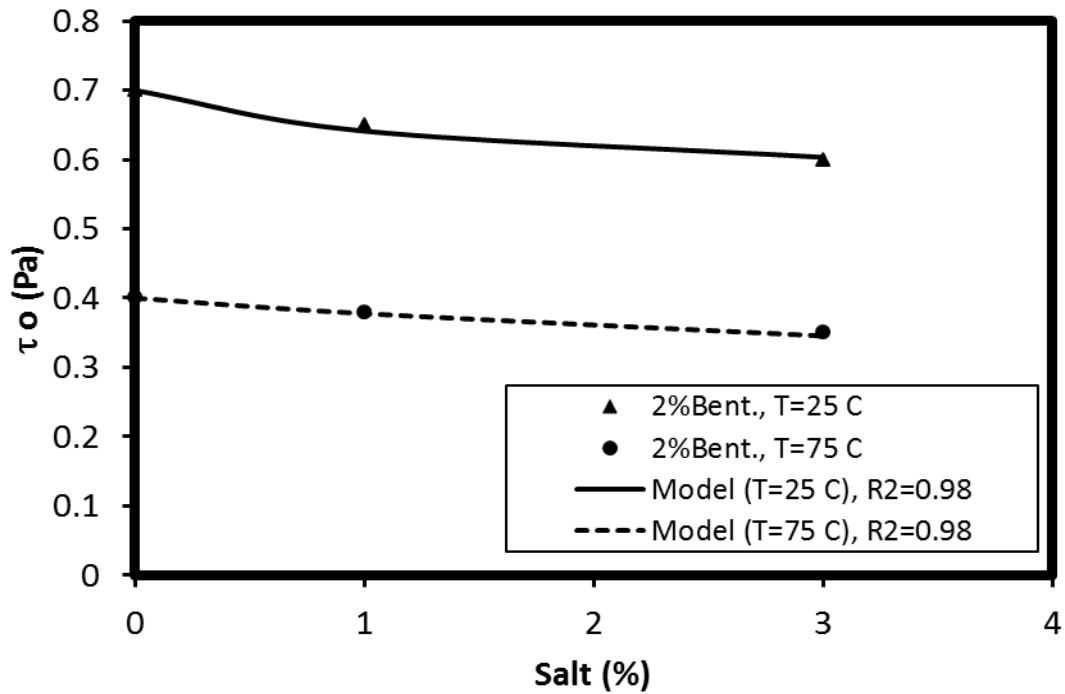


Figure 6.29 Modeling of yield shear stress vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 2% bentonite.

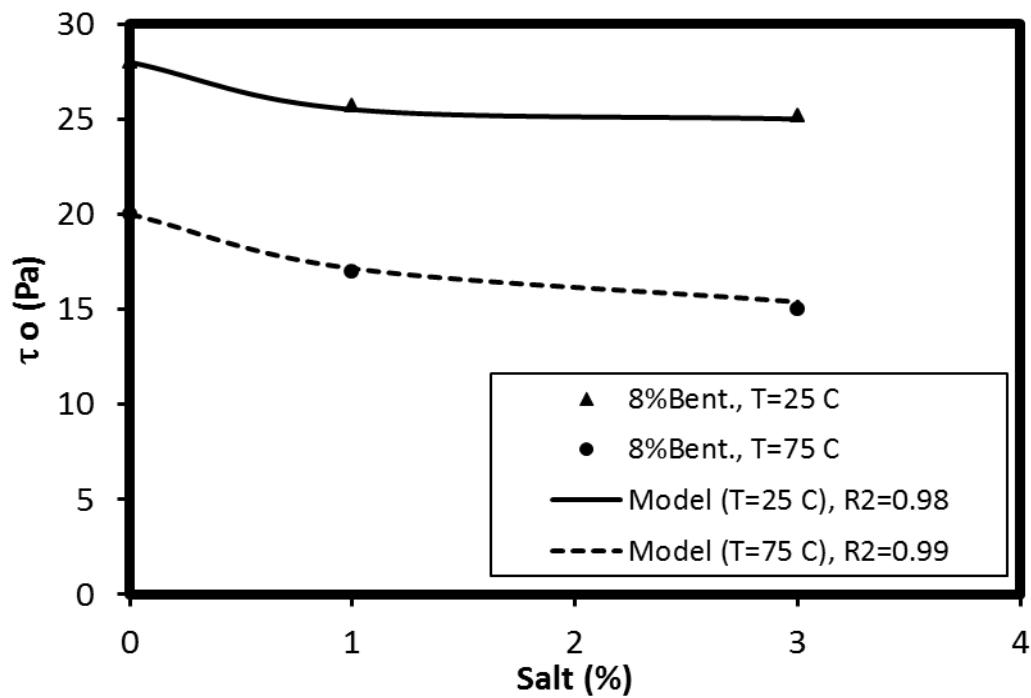


Figure 6.30 Modeling of yield shear stress vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 8% bentonite.

It is clearly indicated the contribution of bentonite content to have the highest impact on the $(\tau_v)_o$ and R and S parameters while the contribution of the combined effect of bentonite content and temperature was the minimal.

(ii) Maximum shear stress

2% Bentonite content

As the salt contamination and temperature increased, the maximum shear stress decreased for the 2% bentonite drilling mud (Figure 6.31). When the salt contamination in the 2% bentonite drilling mud increased from 0% to 3%, the maximum shear stress decreased by 39% and 35% at 25° C and 75° C respectively. As the temperature increased from 25° C to 75° C, the maximum shear stress decreased by 42%, 34% and 37% for 2% bentonite drilling mud contaminated with 0%, 1% and 3% salt contents respectively.

8% Bentonite content

As the salt contamination of 8% bentonite drilling mud increased from 0% to 3%, the maximum shear stress decreased by 74% and 84% at 25° C and 75° C respectively (Figure 6.32). As the temperature increased from 25° C to 75° C, the maximum shear stress decreased by 24%, 46% and 51% for 8% bentonite drilling mud contaminated with 0%, 1% and 3% salt contents respectively.

Model parameters correlation

The three model parameters $(\tau_{max})_o$, G and H parameters were correlated with temperature and bentonite contents as follows (Eqs. 6.25, 6.26 and 6.27)

$$(\tau_{max})_o = M_1 + M_2 * (Bent.) + M_3 * (T) + M_4(Bent.)(T), \quad (6.25)$$

$$G = G_1 + G_2 * (\text{Bent.}) + G_3 * (T) + G_4 * (\text{Bent.}) * (T), \text{ and} \quad (6.26)$$

$$H = H_1 + H_2 * (\text{Bent.}) + H_3 * (T) + H_4 * (\text{Bent.}) * (T), \quad (6.27)$$

where Bent. and T are bentonite content and absolute temperature respectively and M_1 to M_4 , G_1 to G_4 and H_1 to H_4 are model parameters.

The coefficient of correlation (R^2) for 2% bentonite drilling mud at 25° C and 75° C were 0.99 and 0.99 respectively. The RMSE for 2% bentonite drilling mud at 25° C and 75° C were 0.0069 Pa and 0.0041 Pa respectively. The coefficient of correlation (R^2) for 8% bentonite drilling mud at 25° C and 75° C were 0.99 and 0.99 respectively. The RMSE for 8% bentonite drilling mud at 25° C and 75° C were 1.27 Pa and 2.52 Pa respectively. Model parameters of Eqs. 6.25 to 6.27 are given in Table 6.12. It is clearly indicated the contribution of bentonite content to have the highest impact on the $(\tau_{\max})_o$ and G and H parameters while the contribution of the combined effect of bentonite content and temperature was the minimal.

Table 6.12 Hyperbolic constitutive parameters

		Model coefficients (Eqs. 6.21 to 6.27)			
Property	Parameter	Zero*	(Bent.)	(T)	(Bent.). (T)
Yield shear stress	$(\tau_Y)_o$	-9.64 (Pa)	5.25 (Pa)	0.045 (Pa/C)	-0.026 (Pa/C)
	R	-5.76 (Pa) ⁻¹	0.71 (Pa) ⁻¹	-0.27 (Pa) ⁻¹ /C	0.033 (Pa) ⁻¹ /C
	S	-10.54 (Pa) ⁻¹	1.27 (Pa) ⁻¹	0.05 (Pa) ⁻¹ /C	-0.006 (Pa) ⁻¹ /C
Maximum shear stress	$(\tau_{\max})_o$	-28.24 (Pa)	15.55 (Pa)	-0.193 (Pa/C)	0.1333 (Pa/C)
	G	0.095 (Pa) ⁻¹	-0.013 (Pa) ⁻¹	-0.019 (Pa) ⁻¹ /C	0.0023 (Pa) ⁻¹ /C
	H	-0.25 (Pa) ⁻¹	0.03 (Pa) ⁻¹	-0.0013 (Pa) ⁻¹ /C	0.00016 (Pa) ⁻¹ /C

* Salt contamination

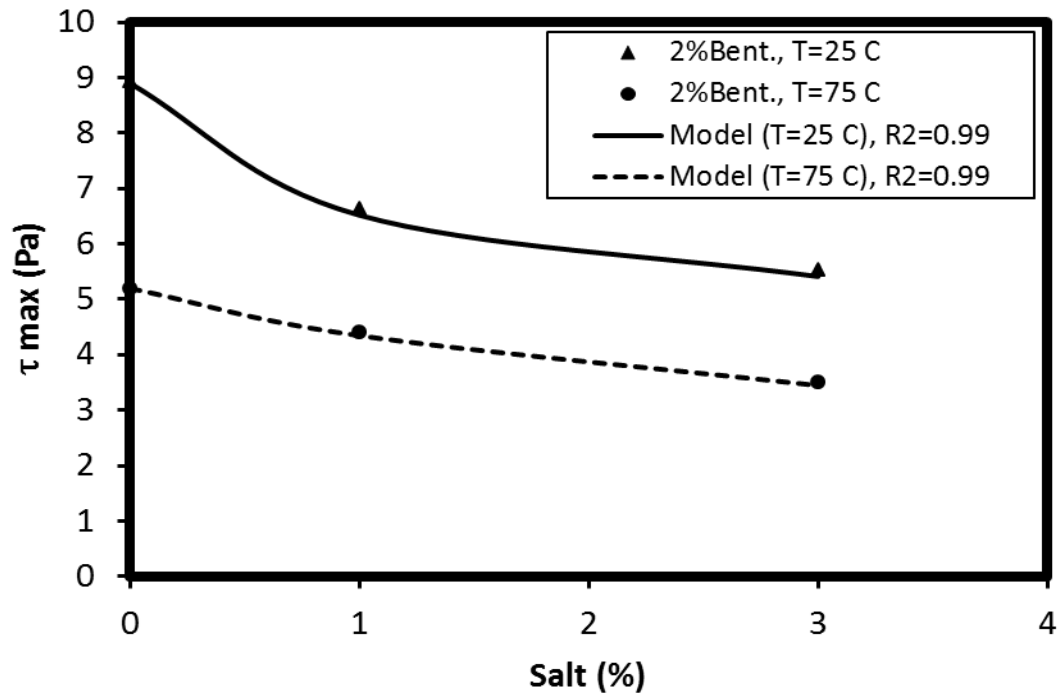


Figure 6.31 Modeling of maximum shear stress vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 2% bentonite.

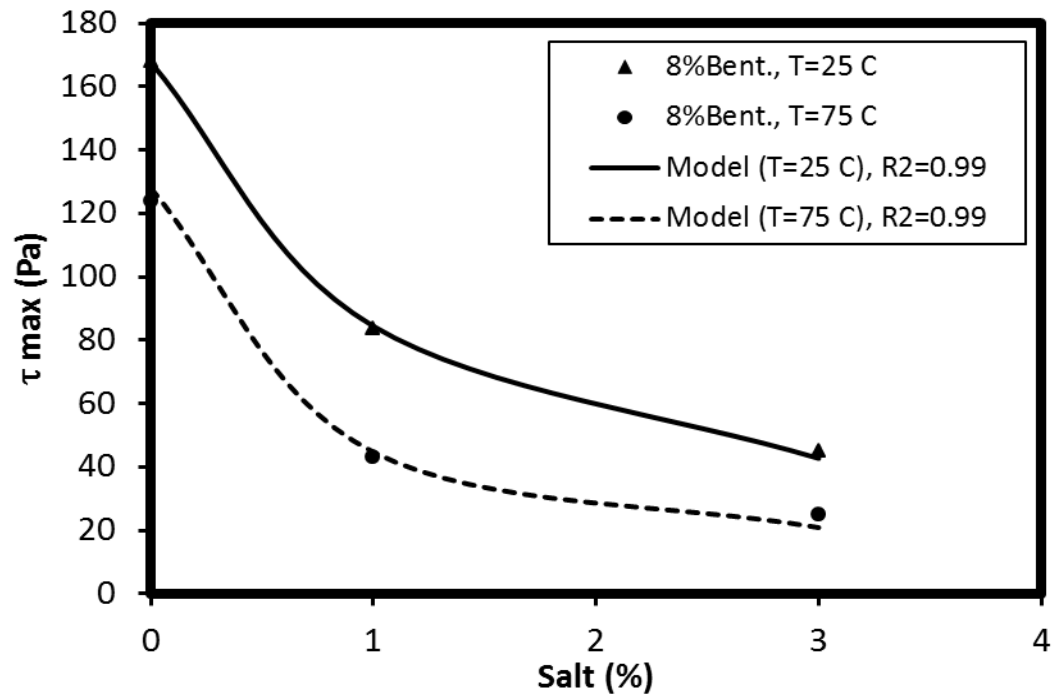


Figure 6.32 Modeling of maximum shear stress vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 8% bentonite.

Resistivity

2% Bentonite content

As the salt contamination and temperature were increased, the electrical resistivity decreased (Figure 6.33). When 3% salt was added to 2% bentonite drilling mud, the electrical resistivity decreased from 5.6 Ohm.m to 1.73 Ohm.m and 3 Ohm.m to 0.2 Ohm.m at 25° C and 75° C respectively. While as the temperature increased from 25° C to 75° C, the electrical resistivity decreased from 5.6 Ohm.m to 3 Ohm.m, 2.21 Ohm.m to 0.55 Ohm.m and 1.73 Ohm.m to 0.2 Ohm.m at 0%, 1% and 3% salt contamination contents respectively.

8% Bentonite content

As the salt content and temperature increased, the electrical resistivity decreased (Figure 6.34). When 3% salt was added to 2% bentonite drilling mud, the electrical resistivity decreased from 2.3 Ohm.m to 1.17 Ohm.m and 0.75 Ohm.m to 0.083 Ohm.m at 25° C and 75° C respectively. While as the temperature increased from 25° C to 75° C, the electrical resistivity decreased from 2.3 Ohm.m to 0.75 Ohm.m, 1.43 Ohm.m to 0.22 Ohm.m and 1.12 Ohm.m to 0.083 Ohm.m at 0%, 1% and 3% salt contamination contents respectively. The proposed hyperbolic constitutive (Eq.6.19) model predicted all the data with coefficient of correlation ($R^2=0.99$) (Figure 6.33 and Figure 6.34).

Model parameters correlation

The three model parameters ρ_o , W and X parameters were correlated with temperature and bentonite contents as follows (Eqs. 6.28, 6.29 and 6.30)

$$\rho_o = R_1 + R_2 * (\text{Bent.}) + R_3 * (T) + R_4(\text{Bent.})(T), \quad (6.28)$$

$$W = W_1 + W_2 * (\text{Bent.}) + W_3 * (T) + W_4 * (\text{Bent.}) * (T), \text{ and} \quad (6.29)$$

$$X = X_1 + X_2 * (\text{Bent.}) + X_3 * (T) + X_4 * (\text{Bent.}) * (T), \quad (6.30)$$

where Bent. and T are bentonite content and absolute temperature respectively and R_1 to R_4 , W_1 to W_4 and X_1 to X_4 are model parameters.

The coefficient of correlation (R^2) for 2% bentonite drilling mud at 25° C and 75° C were 0.99 and 0.99 respectively. The RMSE for 2% bentonite drilling mud at 25° C and 75° C were 0.027 Ohm.m and 0.034 Ohm.m respectively. The coefficient of correlation (R^2) for 8% bentonite drilling mud at 25° C and 75° C were 0.99 and 0.99 respectively. The RMSE for 8% bentonite drilling mud at 25° C and 75° C were 0.026 Ohm.m and 0.0081 Ohm.m respectively. Model parameters of Eqs. 6.28 to 6.30 are given in Table 6.13. It is clearly indicated the contribution of the temperature to have the highest impact on the ρ_0 while the contribution of the bentonite content was the highest on both W and X parameters.

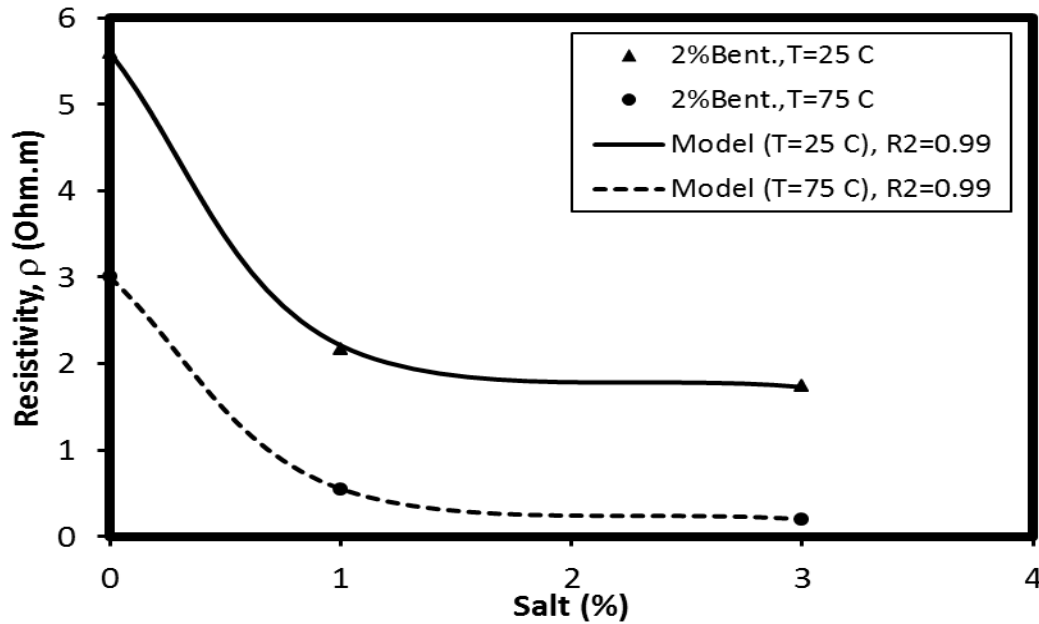


Figure 6.33 Modeling of resistivity vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 2% bentonite.

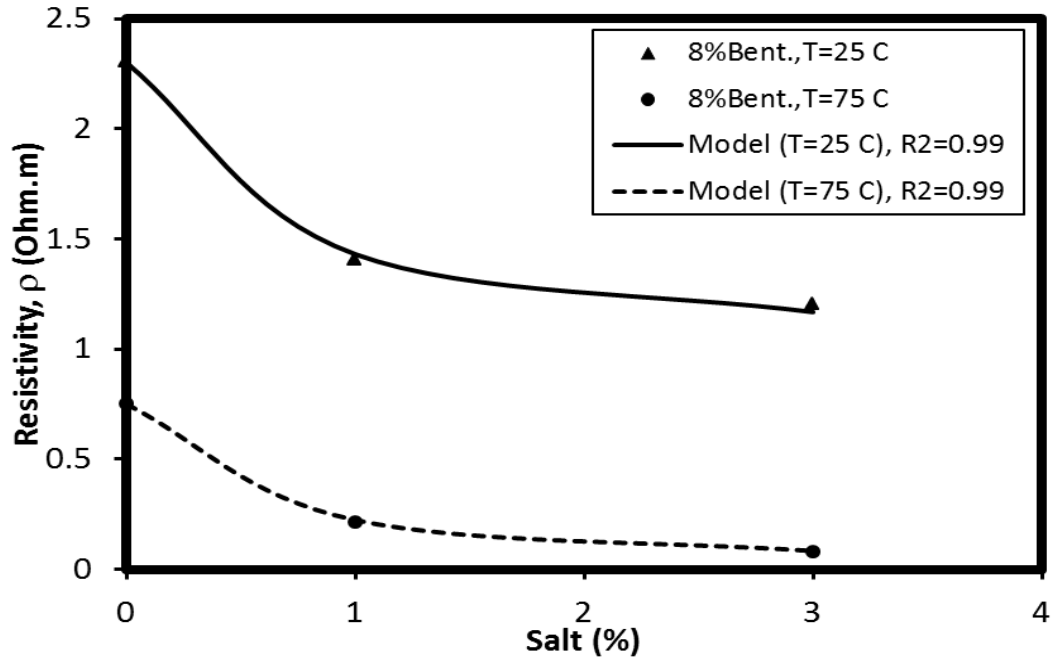


Figure 6.34 Modeling of resistivity vs. salt content of bentonite drilling mud under different temperatures (25° C and 75° C) for 8% bentonite.

Table 6.13 Hyperbolic constitutive parameters

Property	Parameter	Model coefficients (Eqs. 6.28 to 6.30)			
		Zero*	(Bent.)	(T)	(Bent.). (T)
Electrical resistivity	ρ_0	2.19 (Ohm.m)	-0.29 (Ohm.m)	1.39 (Ohm.m)/C	-0.0035 (Ohm.m)/C
	W	0.05 (Ohm.m) ⁻¹	-0.05 (Ohm.m) ⁻¹	0.0004 (Ohm.m) ⁻¹ /C	-0.0006 (Ohm.m) ⁻¹ /C
	X	-0.11 (Ohm.m) ⁻¹	-0.04 (Ohm.m) ⁻¹	0.0006 (Ohm.m) ⁻¹ /C	-0.001 (Ohm.m) ⁻¹ /C

* Salt contamination

4- Nonlinear models (NLM)

(i) Material and rheological

Electrical resistivity (ρ)

Based on 36 data used for electrical resistivity from the current study and using nonlinear optimization scheme, the following relationship was obtained (Eq. 6.31):

$$\rho = 42.1 * (\text{Bent. \%})^{-1.45} + 0.356 * (\text{Bent. \%})^{-0.3} * (\Delta T)^{0.5} + 0.948 * (\text{Bent. \%})^{-0.65} * (\text{Salt \%})^{-0.75} + 6.277 * (\text{Bent. \%})^{-0.74} * (\Delta T)^{0.094} * (\text{Salt \%})^{0.092}. \quad (6.31)$$

The least square method has been implemented to obtain the NLM parameters and the model parameters as shown in Table 6.14 was obtained. The relation between predicated and experimental data for electrical resistivity was shown in Figure 6.35 with the coefficient of determination of 0.93. Based on NLM (Table 6.14), bentonite content had the highest effect on decreasing the electrical resistivity compared to salt contamination and temperature. NLM showed that the salt contamination effect was 2.66 times the temperature effect in reducing the electrical resistivity.

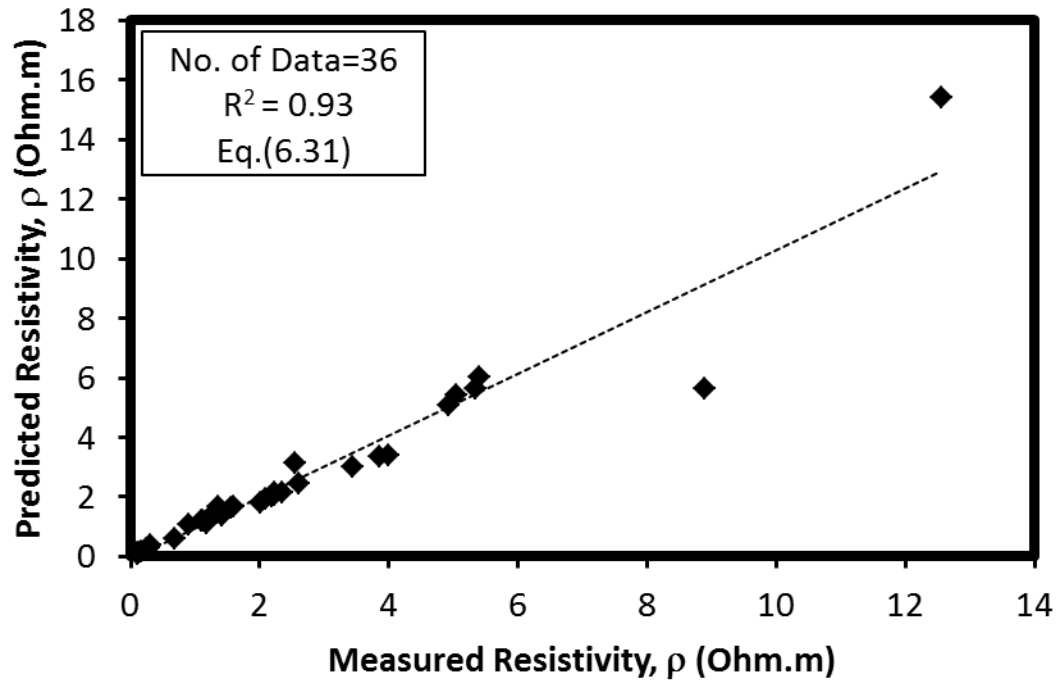


Figure 6.35 Compare the measured and predicated electrical resistivity of bentonite drilling mud contaminated with salt.

Yield shear stress (τ_o)

Based on 22 data used for yield shear stress from the current study and using nonlinear optimization scheme, the following relationship was obtained (Eq.6.32)

$$\tau_o = 0.079 * (\text{Bent. \%})^{2.93} + 4.27 * 10^{-3} * (\text{Bent. \%})^{3.14} * (\Delta T)^{0.33} + 0.073 * (\text{Bent. \%})^{2.87} * (\text{Salt\%})^{0.18} + 0.083 * (\text{Bent. \%})^{2.79} * (\Delta T)^{0.04} * (\text{Salt\%})^{0.16}. \quad (6.32)$$

The least square method has been implemented to obtain the NLM parameters and the model parameters as shown in Table 6.14 was obtained. The relation between predicated and experimental data for yield shear stress (τ_o) was shown in Figure 6.36. The relation between predicated and experimental data for yield shear stress was very good with the coefficient of determination of 0.99. Based on NLM (Table 6.14), bentonite content had the highest effect on increasing the yield shear stress while the salt contamination and temperature has the tendency to decrease the yield shear stress. NLM showed that the salt contamination effect was 3 times the temperature effect in reducing the yield shear stress.

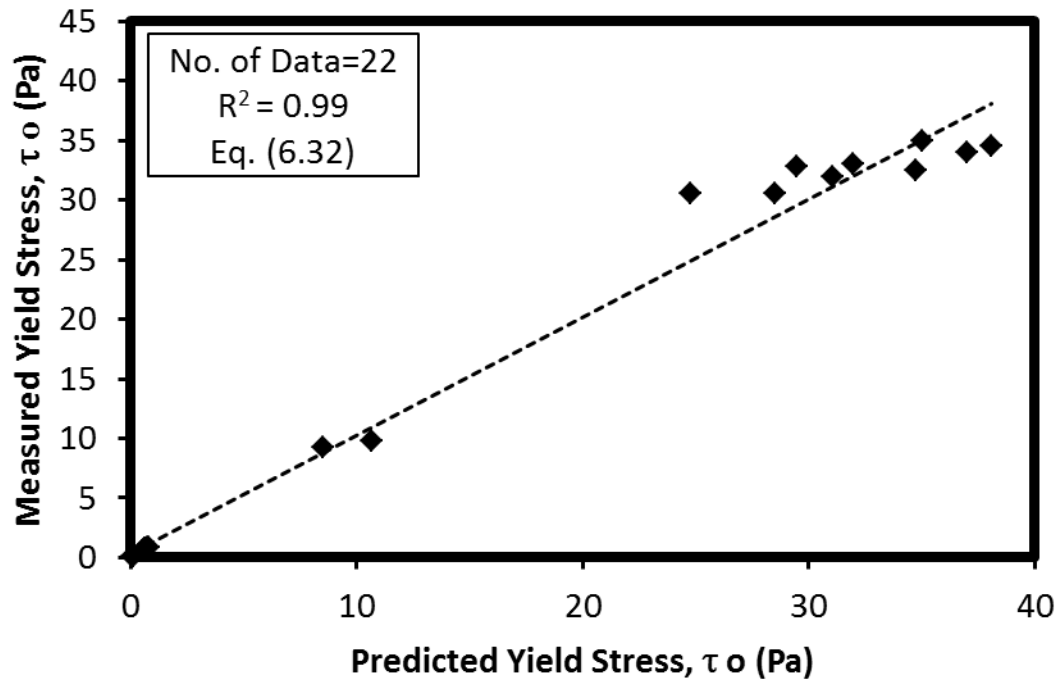


Figure 6.36 Compare the measured and predicated yield shear stress of bentonite drilling mud contaminated with salt (Eq. 6.32).

Maximum shear stress (τ_{max})

Based on 22 data used for maximum shear stress from the current study and using nonlinear optimization scheme, the following relationship was obtained (Eq.6.33)

$$\tau_{max} = 2.05 * (\text{Bent. \%})^{2.12} + 0.66 * (\text{Bent. \%})^{1.81} * (\Delta T)^{0.12} + 0.38 * (\text{Bent. \%})^{2.6} * (\text{Salt\%})^{0.35} + 0.49 * (\text{Bent. \%})^{2.47} * (\Delta T)^{0.11} * (\text{Salt\%})^{0.22}. \quad (6.33)$$

The least square method has been implemented to obtain the NLM parameters and the model parameters as shown in Table 6.14 was obtained. The relation between predicated and experimental data for yield shear stress (τ_{max}) was shown in Figure 6.37. The relation between predicated and experimental data for maximum shear stress was very good with the coefficient of determination of 0.97. Based on NLM (Table 6.14), bentonite content had the highest effect on increasing the maximum shear stress while the salt contamination and temperature has the tendency to decrease the maximum shear stress. NLM showed that the salt contamination effect was 2.8 times the temperature effect in reducing the maximum shear stress.

Parameter- A_4 of hyperbolic model (Eq.6.16)

Based on 22 data used for parameter- A_4 (Eq.6.16) from the current study and using nonlinear optimization scheme, the following relationship was obtained (Eq.6.34)

$$\text{Parameter-}A_4 = 571.9 * (\text{Bent. \%})^{-3.16} + 642.3 * (\text{Bent. \%})^{-3.84} * (\Delta T)^{0.36} + 541.3 * (\text{Bent. \%})^{-2.4} * (\text{Salt\%})^{0.14} + 567.4 * (\text{Bent. \%})^{-2.72} * (\Delta T)^{0.085} * (\text{Salt\%})^{0.88}. \quad (6.34)$$

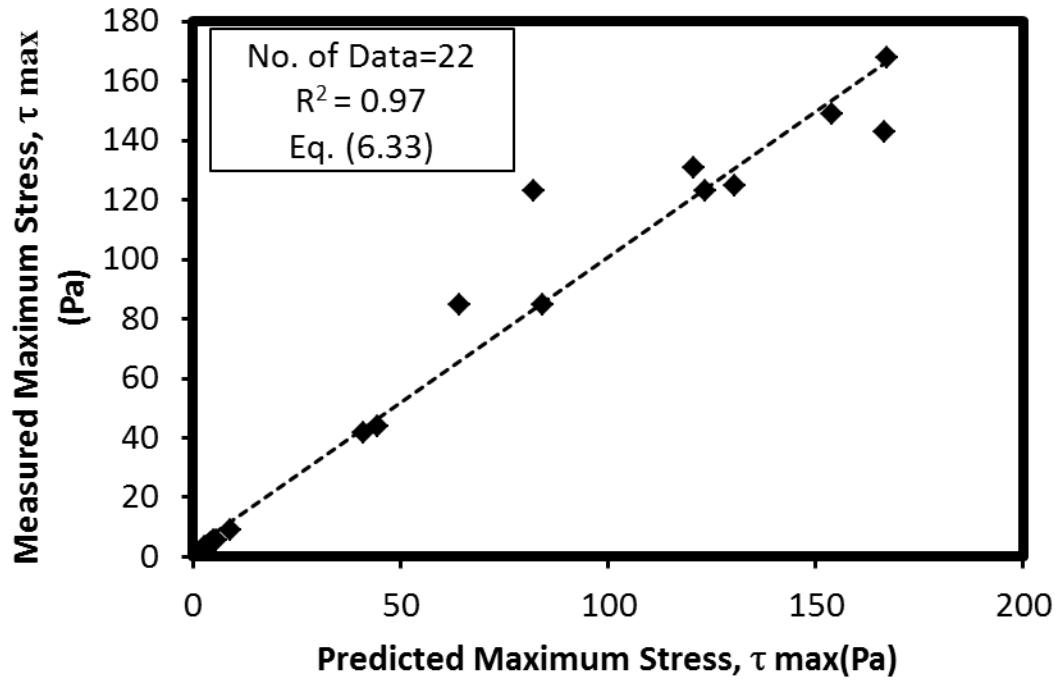


Figure 6.37 Compare the measured and predicated maximum shear stress of bentonite drilling mud contaminated with salt (Eq. 6.33).

The least square method has been implemented to obtain the NLM parameters. The relation between predicated and experimental data for the parameter- A_4 was shown in Figure 6.38. The relation between experimental and predicated data for parameter- A_4 using Eq. (6.34) was very good with the coefficient of determination of 0.94. Based on NLM (Table 6.14), temperature had the highest effect on decreasing the parameter- A_4 compared to the bentonite content and salt contamination. NLM showed that the bentonite content effect was 3.3 times the salt contamination effect in reducing the parameter- A_4 .

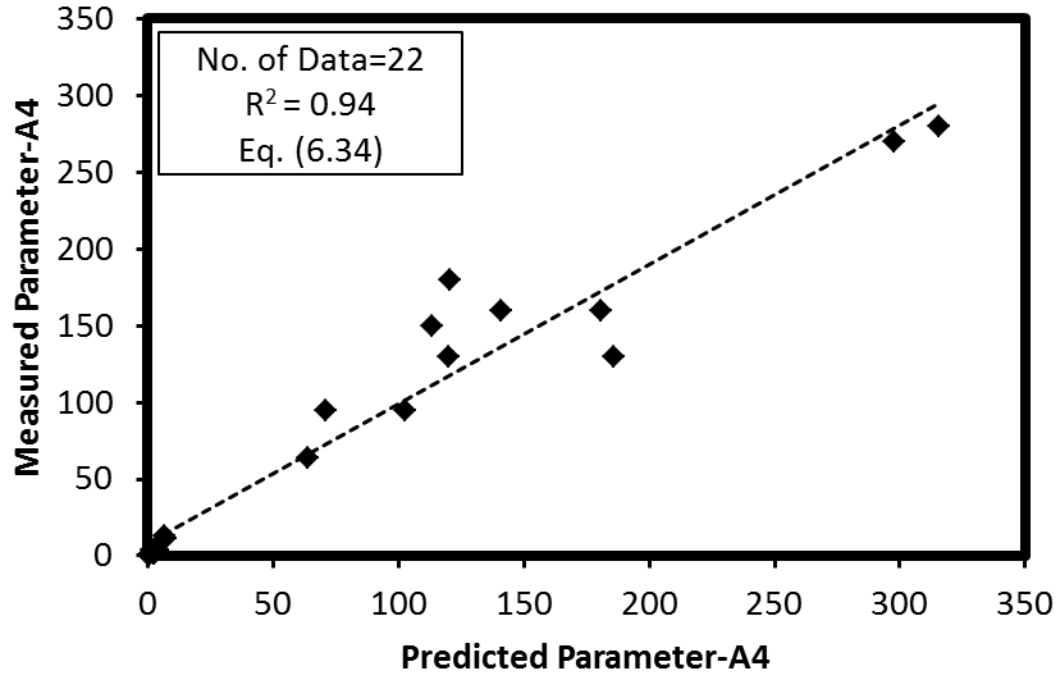


Figure 6.38 Compare the measured and predicated A_4 -parameters of bentonite drilling mud contaminated with salt (Eq. 6.34).

Parameter- B_4 of hyperbolic model (Eq.6.16)

Based on 22 data used for parameter- B_4 (Eq.6.16) from the current study and using nonlinear optimization scheme, the following relationship was obtained (Eq.6.35)

$$\text{Parameter-}B_4 = 0.2 * (\text{Bent. \%})^{-1.62} + 0.09 * (\text{Bent. \%})^{-1.3} * (\Delta T)^{0.12} + 0.12 * (\text{Bent. \%})^{-1.14} * (\text{Salt\%})^{0.38} + 0.078 * (\text{Bent. \%})^{-0.77} * (\Delta T)^{0.11} * (\text{Salt\%})^{0.26}. \quad (6.35)$$

The least square method has been implemented to obtain the NLM parameters. The relation between predicated and experimental data for the parameter- B_4 was shown in Figure 6.39. The relation between experimental and predicated data for parameter- B_4 using Eq. (6.35) was good with the coefficient of determination of 0.81. Based on NLM (Table 6.14), bentonite content had the highest effect on decreasing the parameter- B_4

compared to the salt contamination and temperature. NLM showed that the temperature effect was 1.74 times the salt contamination effect in reducing the parameter-B₄.

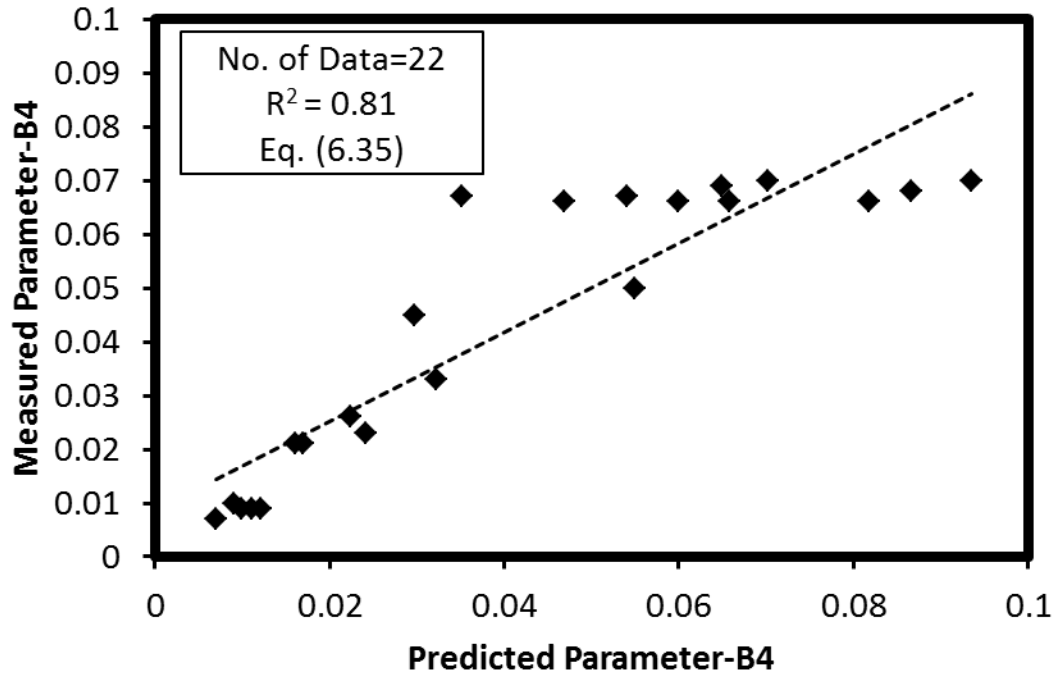


Figure 6.39 Compare the measured and predicated B₄-parameters of bentonite drilling mud contaminated with salt (Eq. 6.35).

(ii) Resistivity

Yield shear stress (τ_o)

Based on 18 data used for yield shear stress from the current study and using nonlinear optimization scheme, the following relationship was obtained (6.36)

$$\tau_o = 0.46 * (\rho)^{0.2} + (3.81 * \text{Bent. \%} - 7.62) * (\rho)^{0.17}. \quad (6.36)$$

The least square method has been implemented through multiple regression analysis and the model parameters as shown in Table 6.15 was obtained. The relation between predicated and experimental data for yield shear stress was shown in Figure 6.40. The

comparison between predicted and experimental yield shear stress versus electrical resistivity for both 2% and 8% bentonite drilling mud can be identified in Figure 6.41. The coefficient of correlation (R^2) of the yield shear stress with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.89 and 0.97 respectively (Table 6.15). The RMSE of the yield shear stress with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.38 Pa and 0.772 Pa respectively (Table 6.15). In Table 6.15, it is clearly indicated the dependency of yield shear stress on bentonite content and electrical resistivity. Yield shear stress increased as the bentonite content and resistivity increased. It should be known that electrical resistivity can be used as non-destructive measurement to estimate drilling mud rheological properties, which can be used for deep well drilling mud monitoring.

Maximum shear stress (τ_{max})

Based on 18 data used for maximum shear stress from the current study and using nonlinear optimization scheme, the following relationship was obtained (6.37)

$$\tau_{max} = 4.36 * (\rho)^{0.18} + (13.32 * \text{Bent\%} - 26.6) * (\rho)^{0.37}. \quad (6.37)$$

The least square method has been implemented through multiple regression analysis and the model parameters as shown in Table 6.15 was obtained. The relation between predicated and experimental data for maximum shear stress was shown in Figure 6.42. The comparison between predicted and experimental maximum shear stress versus electrical resistivity for both 2% and 8% bentonite drilling mud can be identified in Figure 6.43. The coefficient of correlation (R^2) of the maximum shear stress with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.97 and

0.92 respectively (Table 6.15). The RMSE of the maximum shear stress with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.205 Pa and 11.7 Pa respectively (Table 6.15). In Table 6.15, it is clearly indicated the dependency of maximum shear stress on bentonite content and electrical resistivity. Maximum shear stress increased as the bentonite content and resistivity increased. It is proven that electrical resistivity can be used as monitoring tool to check the quality of the drilling mud.

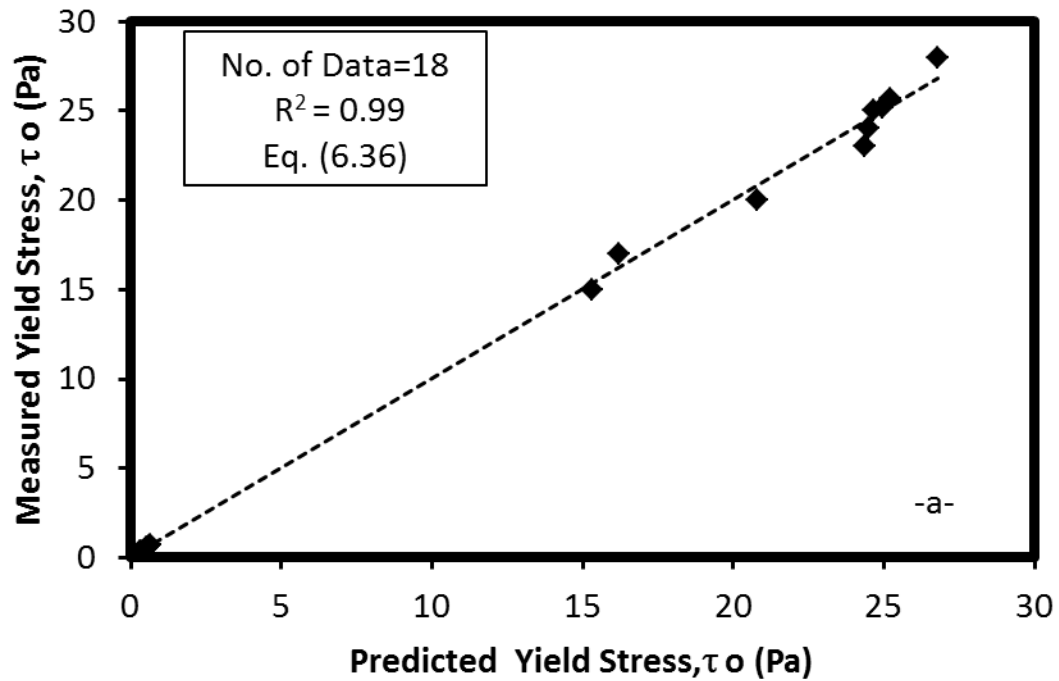


Figure 6.40 Compare the measured and predicated yield shear stress of bentonite drilling mud contaminated with salt (Eq. 6.36).

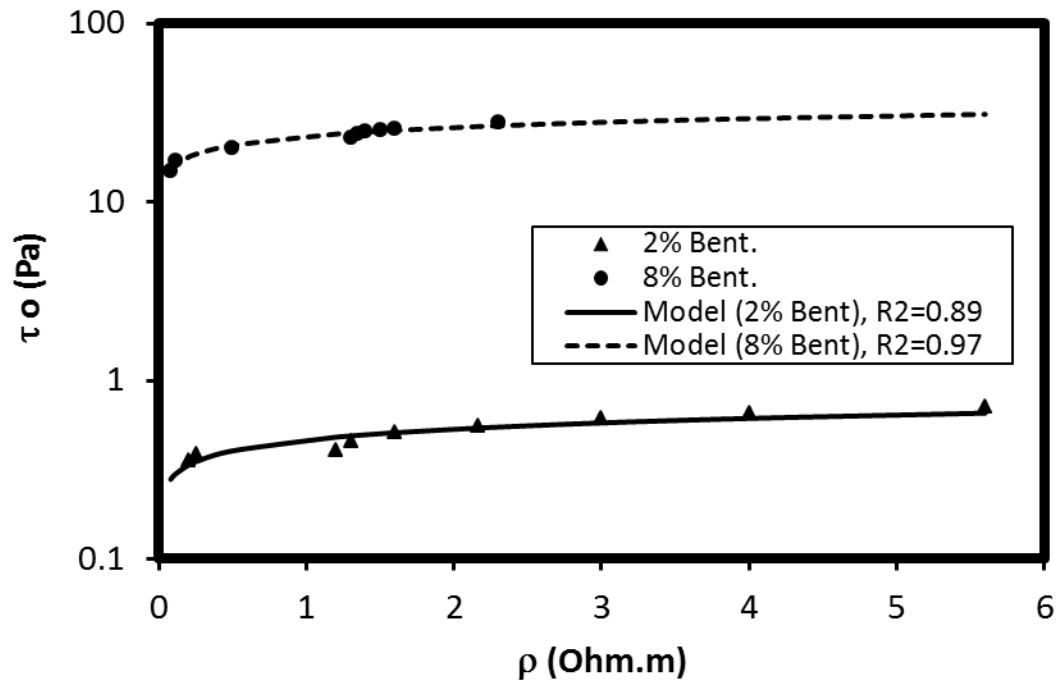


Figure 6.41 The variation between yield shear stress and electrical resistivity of bentonite drilling mud.

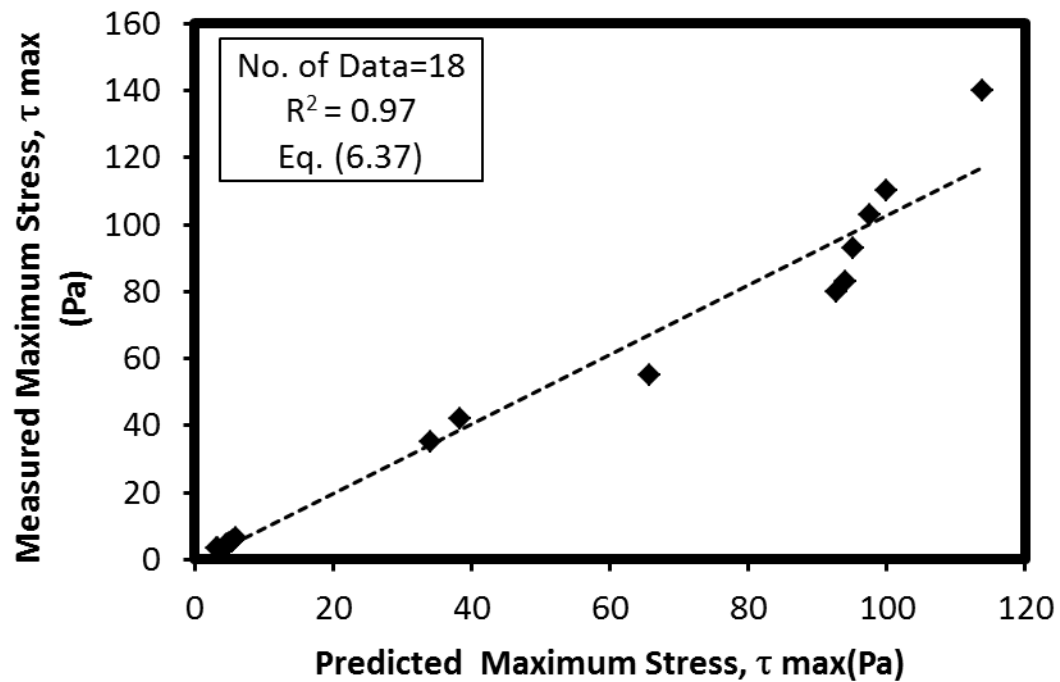


Figure 6.42 Compare the measured and predicated maximum shear stress of bentonite drilling mud contaminated with salt (Eq. 6.36).

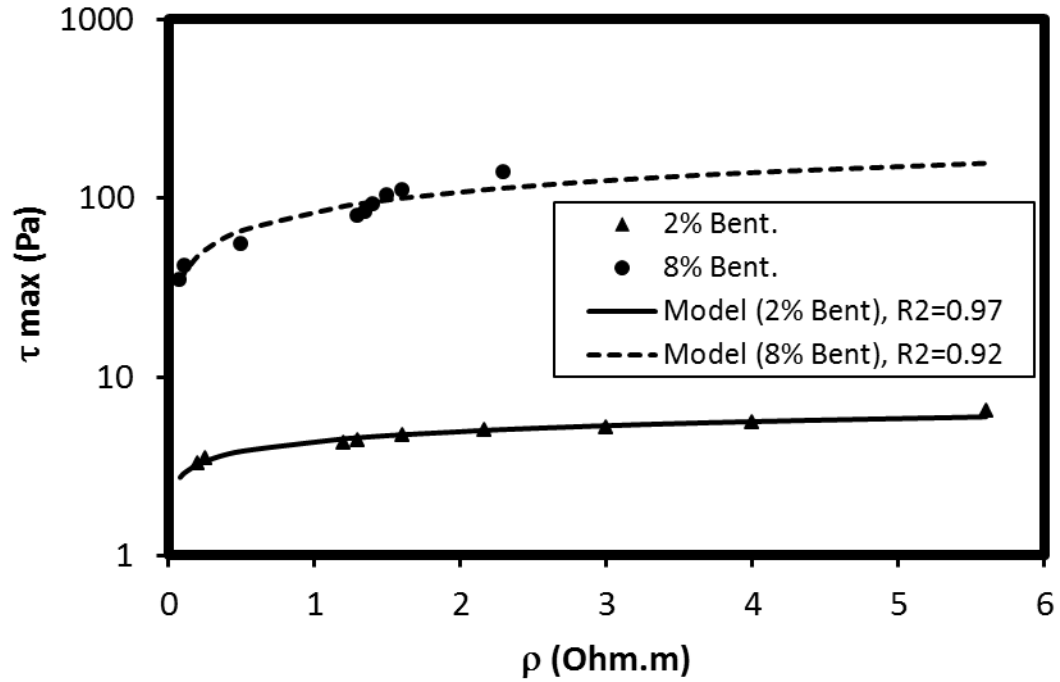


Figure 6.43 The variation between maximum shear stress and electrical resistivity of bentonite drilling mud.

Parameter-A₄ (Eq.6.16)

Based on 18 data used for parameter-A₄ (Eq.6.16) from the current study and using nonlinear optimization scheme, the following relationship was obtained (6.38)

$$\text{Parameter} - A_4 = 1.68 * (\rho)^{-0.86} + (216.2 - 27.02 * \text{Bent. \%}) * (\rho)^{-0.34}. \quad (6.38)$$

The least square method has been implemented through multiple regression analysis and the model parameters as shown in Table 6.15 was obtained. The relation between predicated and experimental data for parameter-A₄ was shown in Figure 6.44. The comparison between predicted and experimental parameter-A₄ versus electrical resistivity for both 2% and 8% bentonite drilling mud can be identified in Figure 6.45. The coefficient of correlation (R^2) of the parameter-A₄ with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.91 and 0.88 respectively

(Table 6.15). The RMSE of the parameter- A_4 with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were $20.55 \text{ (Pa.s)}^{-1}$ and 1.49 (Pa.s)^{-1} respectively (Table 6.15). In Eq. (6.38), it is clearly indicated the dependency of parameter- A_4 on bentonite content and electrical resistivity. Parameter- A_4 decreased as the bentonite content and resistivity increased. It is shown that parameter- A_4 can be captured by electrical resistivity evaluation where it can be used to predict the rheological behavior of the drilling mud.

Parameter-B (Eq.6.16)

Based on 18 data used for parameter- B_4 (Eq.6.16) from the current study and using nonlinear optimization scheme, the following relationship was obtained (6.39)

$$\text{Parameter} - B_4 = 0.015 * (\rho)^{-0.41} + (0.07 - 0.0083 * \text{Bent. \%}) * (\rho)^{0.17}. \quad (6.39)$$

The least square method has been implemented through multiple regression analysis and the model parameters as shown in Table 6.15 was obtained. The relation between predicated and experimental data for parameter- B_4 was shown in Figure 6.46. The comparison between predicted and experimental parameter- B_4 versus electrical resistivity for both 2% and 8% bentonite drilling mud can be identified in Figure 6.47. The coefficient of correlation (R^2) of the parameter- B_4 with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.81 and 0.82 respectively (Table 6.15). The RMSE of the parameter- B_4 with the electrical resistivity relationship for 2% and 8% bentonite drilling mud were 0.0005 (Pa)^{-1} and 0.0048 (Pa)^{-1} respectively (Table 6.15). In Table 6.15, it is clearly indicated the dependency of parameter- B_4 on bentonite content and electrical resistivity. Parameter- B_4 decreased as the bentonite

content and resistivity increased. It is shown that parameter- B_4 can be captured by electrical resistivity evaluation where it can be used to predict the rheological behavior of the drilling mud.

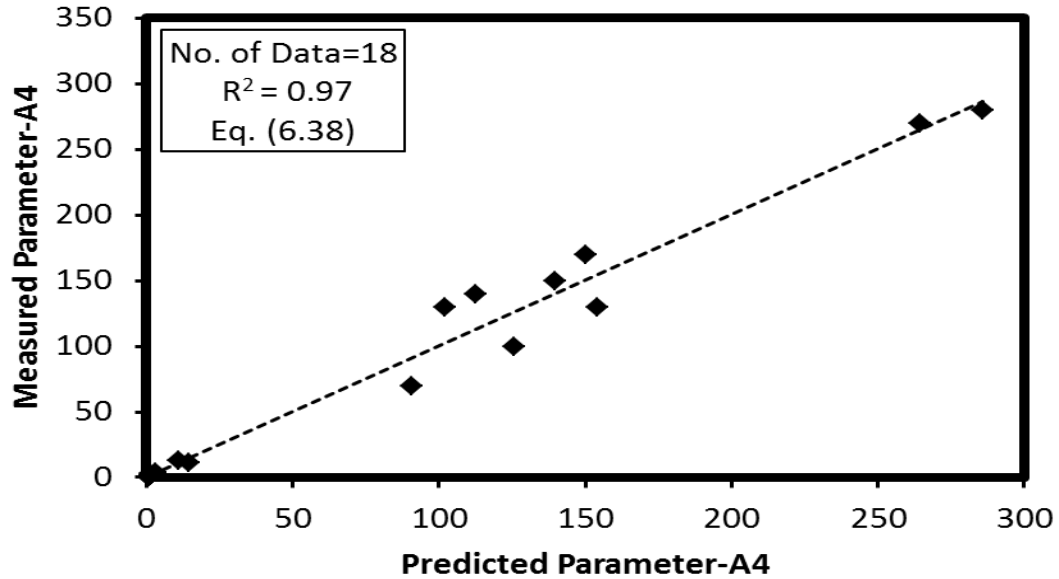


Figure 6.44 Compare the measured and predicated A_4 -parameters of bentonite drilling mud contaminated with salt (Eq. 6.38).

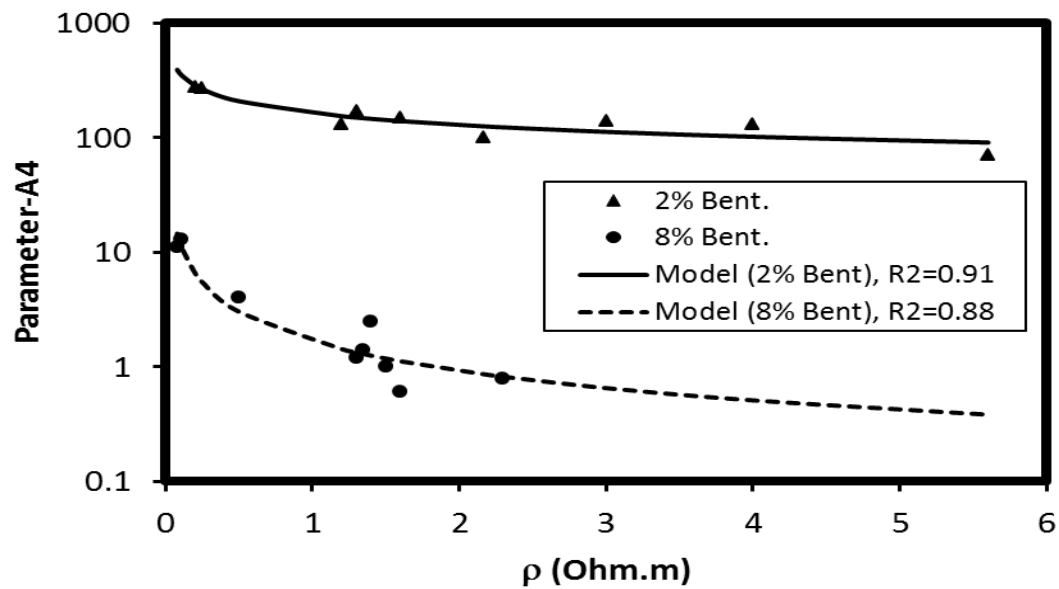


Figure 6.45 The variation between parameter- A_4 and electrical resistivity of bentonite drilling mud.

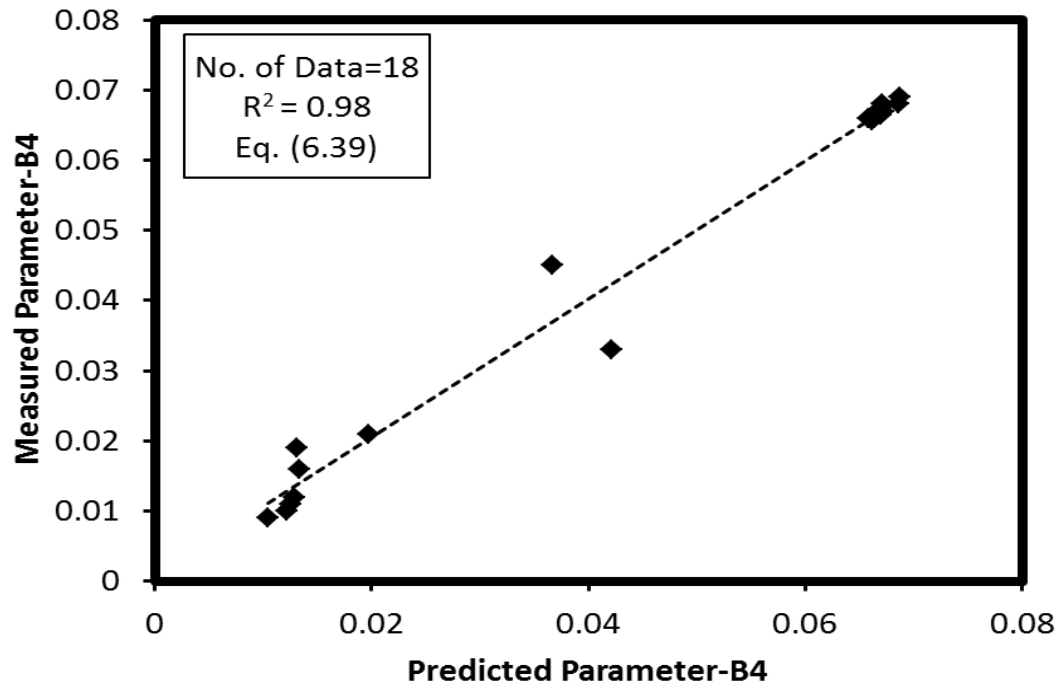


Figure 6.46 Compare the measured and predicated B₄-parameters of bentonite drilling mud contaminated with salt (Eq. 6.39).

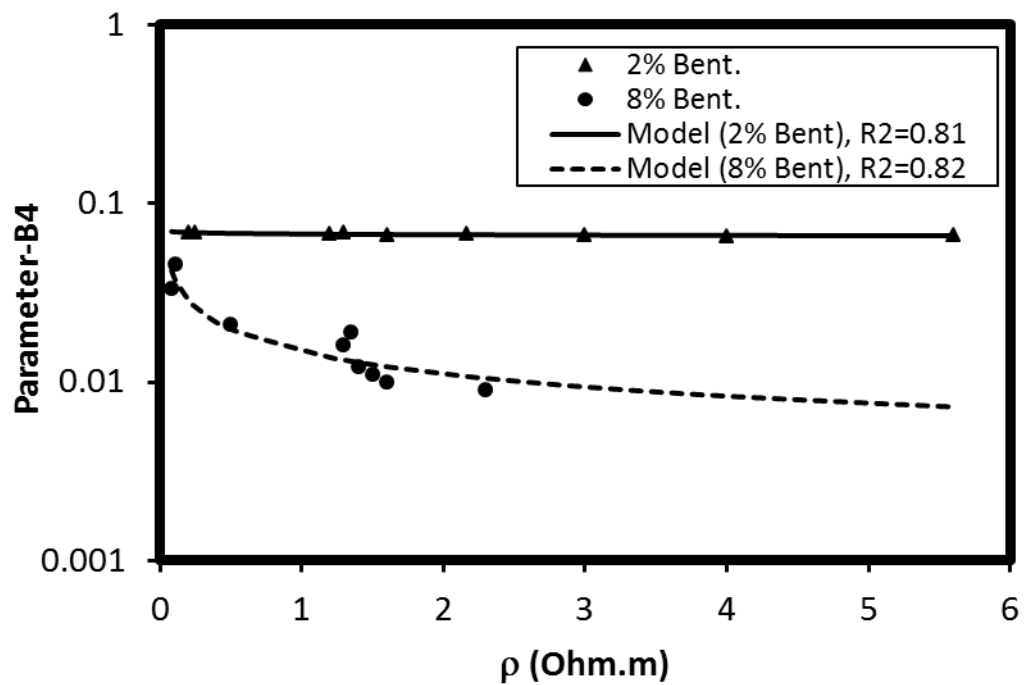


Figure 6.47 The variation between parameter-B₄ and electrical resistivity of bentonite drilling mud.

Table 6.14 Nonlinear model parameters for material and rheological properties (Eqs. 6.31 to 6.35)

Model Parameters	y	z	c	d	e	f	g	h	i	j	k	l	No. of data	RMSE	R ²
ρ (Ohm.m)	42.1 (Ohm.m)	-1.45	0.354 (Ohm.m)/C	-0.3	0.5	0.948 (Ohm.m)	-0.65	-0.75	6.277 (Ohm.m)/C	-0.74	0.094	0.092	36	0.76	0.93
τ_o (Pa)	0.079 (Pa)	2.93	0.0043 (Pa)/C	3.14	0.33	0.073 (Pa)	2.87	0.18	0.083 (Pa)/C	2.79	0.04	0.16	22	1.9	0.99
τ_{max} (Pa)	2.05 (Pa)	2.12	0.66 (Pa)/C	1.81	0.12	0.38 (Pa)	2.6	0.35	0.49 (Pa)/C	2.47	0.11	0.22	22	11.36	0.97
Parameter-A ₄ (Pa s) ⁻¹	571.9 (Pa s) ⁻¹	-3.16	642.3 (Pa s) ⁻¹ /C	-3.84	0.36	541.3 (Pa s) ⁻¹	-2.4	0.14	567.4 (Pa s) ⁻¹ /C	-2.72	0.085	0.88	22	22.97	0.94
Parameter-B ₄ (Pa) ⁻¹	0.2 (Pa) ⁻¹	-1.62	0.09 (Pa) ⁻¹ /C	-1.3	0.12	0.12 (Pa) ⁻¹	-1.14	0.38	0.078 (Pa) ⁻¹ /C	-0.77	0.11	0.26	22	0.012	0.81

Table 6.15 Nonlinear model parameters for resistivity (Eqs. 6.36 to 6.39)

Model Parameters	P	M	Q	N	No. of data	Bent.(%)	RMSE	R ²
τ_o (Pa)	0.46 (Pa/(Ohm.m))	0.2	3.81*Bent.%-7.62 (Pa/(Ohm.m))	0.17	9	2	0.038	0.89
					9	8	0.772	0.97
τ_{max} (Pa)	4.36 (Pa/(Ohm.m))	0.18	13.32*Bent.%-26.6 (Pa/(Ohm.m))	0.37	9	2	0.205	0.97
					9	8	11.7	0.92
Parameter-A (Pa s) ⁻¹	1.68 ((Pa s) ⁻¹ /(Ohm.m))	-0.86	216.2-27.02*Bent.% ((Pa s) ⁻¹ /(Ohm.m))	-0.34	9	2	20.55	0.91
					9	8	1.49	0.88
Parameter-B (Pa) ⁻¹	0.015 ((Pa s) ⁻¹ /(Ohm.m))	-0.41	0.07-0.0083*Bent.% ((Pa s) ⁻¹ /(Ohm.m))	0.17	9	2	0.0005	0.81
					9	8	0.0048	0.82

6.7. Finite element analysis (FEA)

(i) Governing equations

The governing equation for contaminant transport is generally known as the transport equation or the advection-dispersion equation. For easy illustration, one-dimensional transport is presented. The solute transport equation can be derived by considering the mass flux q in an elemental volume of porous material, as illustrated in Figure 6.48. The absolute net mass flux across the element is

$$\text{net mass flux} = \frac{\partial q}{\partial x} dx. \quad (6.40)$$

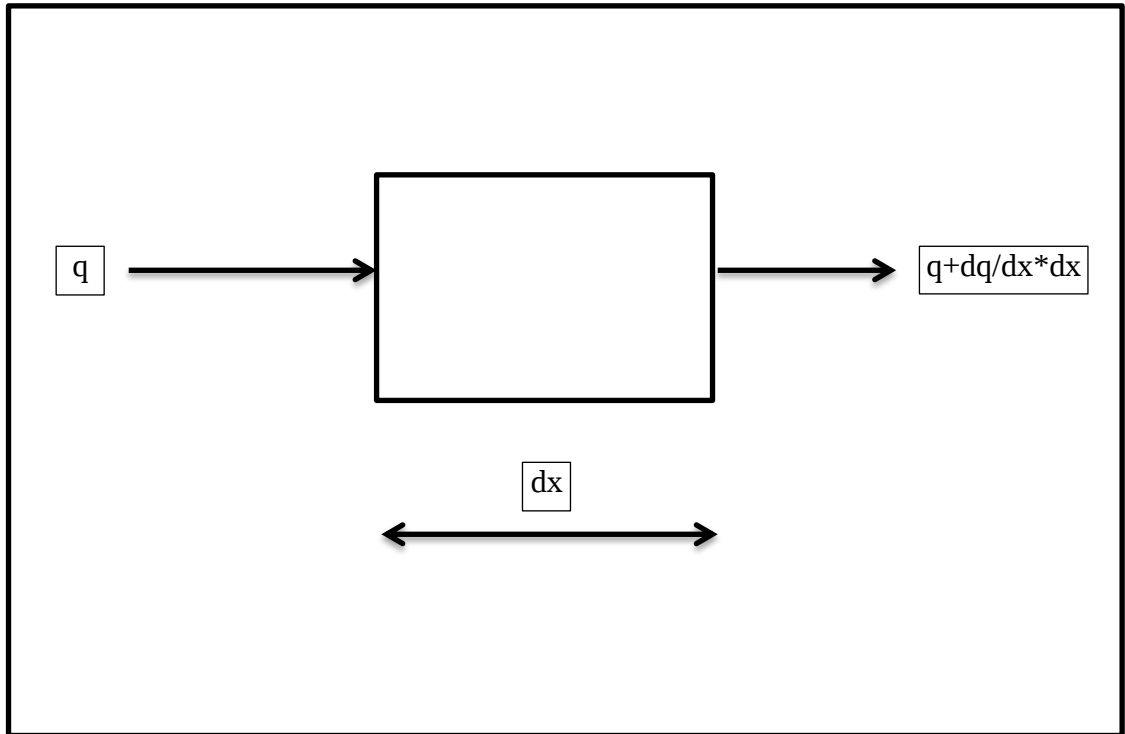


Figure 6.48 Mass balance in a one dimensional element.

To conserve mass, the time rate of change of the total mass M in the element must be equal to the net mass flux. In equation form,

$$\frac{\partial M}{\partial t} dx = - \frac{\partial q}{\partial x} dx. \quad (6.41)$$

By definition, the concentration C is the mass M of dissolved solute in a unit volume of water (solution). In equation form,

$$C = \frac{M}{V_w}, \text{ or } M = C * V_w. \quad (6.42)$$

The volume of water per unit volume of the element is the volumetric water content θ . The mass M per total unit volume then is

$$M = C * \theta. \quad (6.43)$$

Substituting for M in Eq. (6.41) and dividing by dx leads to

$$\theta \frac{\partial C}{\partial t} = - \frac{\partial q}{\partial x}. \quad (6.44)$$

The mass flux through the element arises from both advection and dispersion processes.

In equation form, these two mechanisms are

$$\text{Advection} = v * \theta * C = U * C. \quad (6.45)$$

And,

$$\text{Dispersion} = -\theta * D * \frac{\partial C}{\partial x}, \quad (6.46)$$

where v = average linear velocity, θ = volumetric water content, C = concentration, D = hydrodynamic dispersion coefficient, and U = Darcy velocity (specific discharge).

The negative sign in the dispersion term indicates that the direction of the mass flow is from a high concentration to a low concentration (that is, a negative gradient).

Substituting the previous two terms into Eq. (6.44) leads to the basic transport equation:

$$\theta \frac{\partial C}{\partial t} = - \frac{\partial}{\partial x} \left(-\theta * D * \frac{\partial C}{\partial x} + U * C \right) = \theta * D \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x}. \quad (6.47)$$

This Eq. can be divided by θ , which leads to:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} - v \frac{\partial C}{\partial x}, \quad (6.48)$$

where v is the average linear velocity. This is the form of the equation often seen in publications (Freeze and Cherry 1979). The hydrodynamic dispersion coefficient D is related to the dispersivity, average linear velocity and diffusion coefficient D^* by

$$D = \alpha v + D^*. \quad (6.49)$$

Eq. (6.47) represents the basic transport of a non-reactive and nonradioactive substance; that is, there is no loss of mass due to adsorption or radioactive decay. For general formulation, the loss of mass due to adsorption and radioactive decay must be added to the transport equation. For the transport of a radioactive substance, mass may be lost during the transport process due to radioactive decay of ions in the pore fluid and decay of ions attached to the soil particles. The reduced concentration resulting from radioactive decay, in terms of the initial concentration, is

$$C = C_0 * e^{-\lambda t}, \quad (6.50)$$

where t is the elapsed time and λ is the decay coefficient. The decay coefficient λ can be related to the half-life T of a decaying material. By definition, the half-life T is the elapsed time when the concentration of $C/C_0 = 1/2$. Therefore,

$$\frac{C}{C_0} = \frac{1}{2} = e^{-\lambda T}, \quad (6.51)$$

which can also be written as

$$\lambda = \frac{\ln 2}{T} = \frac{0.693}{T}. \quad (6.52)$$

Differentiating Eq. (6.50) with respect to time leads to

$$\frac{\partial C}{\partial t} = -\lambda C. \quad (6.50)$$

The amount of radioactive mass in the pore-water M_w in an elemental unit volume is $\theta * C$, or

$$M_w = \theta * C_o * e^{-\lambda t}. \quad (6.53)$$

The adsorption S is the amount of mass attached to the soil particles divided by the mass of the solids. In equation form,

$$S = \frac{\text{mass of solute attached to the solids}}{\text{mass of the solids}}. \quad (6.54)$$

The mass per unit volume of the soil (solid) particles can be defined in terms of the bulk (dry) mass density ρ_d of the soil. The parameter S is then defined as

$$S = \frac{M_s}{\rho_d}, \quad (6.55)$$

or

$$M_s = S * \rho_d, \quad (6.56)$$

where M_s is the amount of mass attached to a unit mass of soil particles. In terms of radioactive decay,

$$M_s = S * \rho_d = \rho_d * S_o * e^{-\lambda t}. \quad (6.57)$$

Therefore, the total radioactive mass M in both the fluid and solid phases is

$$M = M_w + M_s = \theta * C_o * e^{-\lambda t} + \rho_d * S_o * e^{-\lambda t}. \quad (6.58)$$

The rate of change of mass due to decay is then

$$\frac{\partial M}{\partial t} = -\lambda * \theta * C - \lambda * S * \rho_d. \quad (6.59)$$

The transport equation (Eq.6.47) can now be modified to include radioactive decay. The result is

$$\theta \frac{\partial C}{\partial t} = \theta D \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} - \lambda * \theta * C - \lambda * S * \rho_d. \quad (6.60)$$

For the transport of a reactive substance, the movement of the mass is also affected by the adsorption of the solute by the soil particles. As discussed above, the amount of mass

adsorbed can be defined in terms of the mass density of the soil particles. From Eq.6.57, the rate of change of the adsorbed mass is

$$\frac{\partial M_s}{\partial t} = \rho_d \frac{\partial S}{\partial t}. \quad (6.61)$$

The adsorption S is a function of concentration C . Experimental results are usually plotted as S versus C . The slope of the S versus C relationship is $\partial S / \partial C$. In the case of a linear relationship, the slope is usually referred to as the distribution coefficient K_d . Then, Eq. (6.61) can then be written as

$$\frac{\partial M_s}{\partial t} = \rho_d \frac{\partial S}{\partial C} \frac{\partial C}{\partial t}. \quad (6.62)$$

Adding the adsorption term to Eq.6.47 gives the following governing differential equation

$$\theta \frac{\partial C}{\partial t} + \rho_d \frac{\partial S}{\partial C} \frac{\partial C}{\partial t} = \theta D \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} - \lambda * \theta * C - \lambda * S * \rho_d, \quad (6.63)$$

or

$$\left(\theta + \rho_d \frac{\partial S}{\partial C} \right) \frac{\partial C}{\partial t} = \theta D \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} - \lambda * \theta * C - \lambda * S * \rho_d. \quad (6.64)$$

Eq. 6.64 can be re-written in matrix form after applying Galerkin approach for finite element analysis as

$$[K]\{C\} = \{Q\}, \quad (6.65)$$

where $[K]$ = a matrix of coefficients related to geometry and material properties, $\{C\}$ = a vector of the concentration at the nodes, and $\{Q\}$ = a vector of the contaminant flux quantities at the node.

(ii) Case study

The downward flow of salt contamination movement to 6% bentonite drilling has been modeled through experimental and numerical finite element study. Initially, an

experimental study has been done to investigate the relationship between the salt concentration and electrical resistivity in 6% bentonite drilling mud as shown in Figure 6.49. The variation of salt concentration with electrical resistivity is modeled as follows:

$$\text{Concentration (\%)} = \frac{\rho}{A+B*\rho}, \quad (6.66)$$

where ρ is the electrical resistivity, A and B are model parameters.

The proposed model (Eq.6.66) agreed very well with the experimental model with R^2 of 0.98. Then, the variation of the electrical resistivity with the time for 6% bentonite drilling mud under the effect of the downward flow of salt contamination was studied as shown Figure 6.50. The variation of the electrical resistivity was monitored up to 48 hours. From the concentration-electrical resistivity and electrical resistivity–time relationships, the salt concentration at any time period in 6% bentonite drilling mud can be obtained as shown in Figure 6.50. Finally, finite element analysis using Eq. (6.65) was to used compare the variation of salt contamination with time with the experimental results as shown in Figure 6.51. FEM analysis included 663 triangular elements with 728 nodes of 2 mm average element size. Finite element result agreed very well with the experimental results for the prediction of the salt concentration over the time.

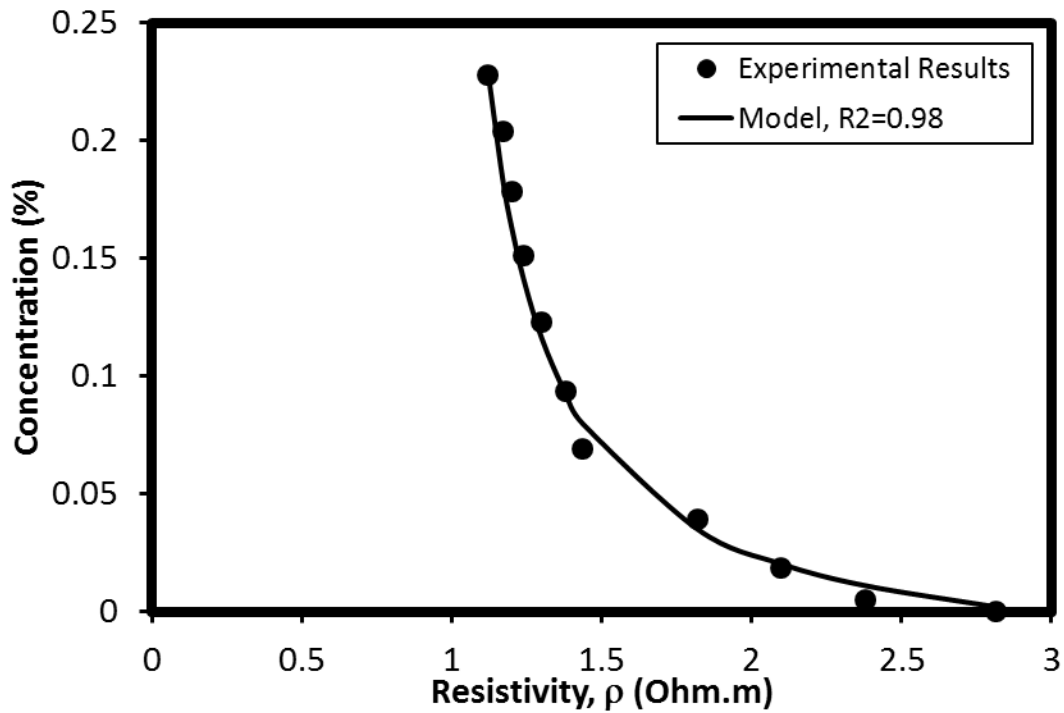


Figure 6.49 Concentration versus resistivity relationship.

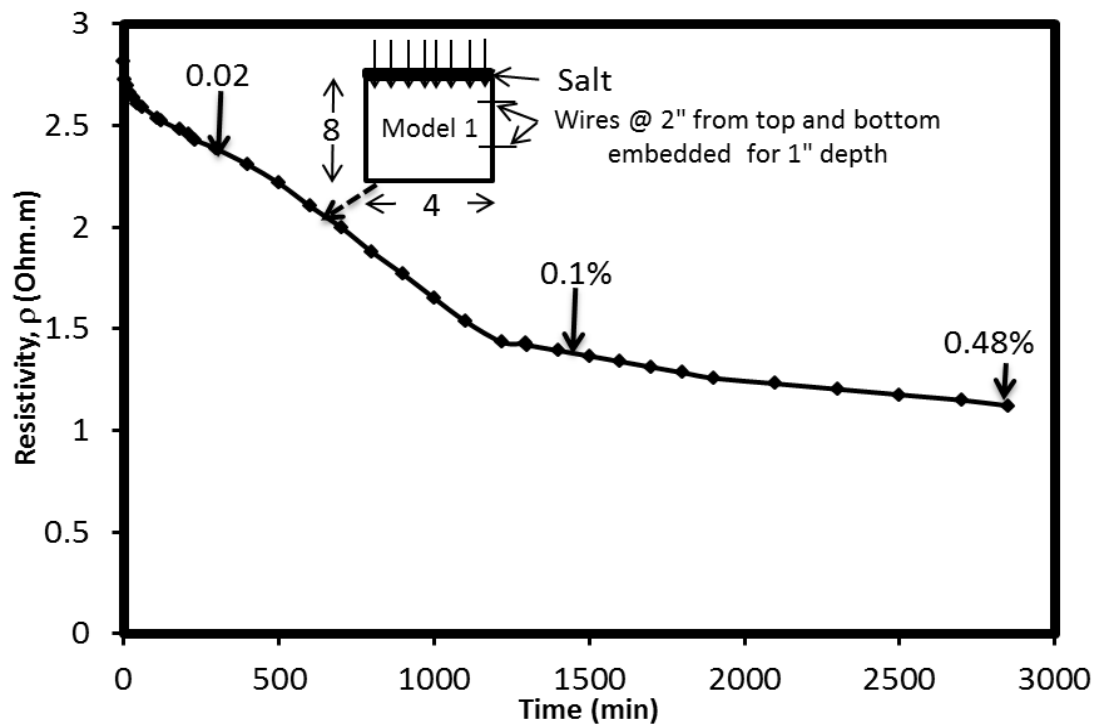


Figure 6.50 The variation of the electrical resistivity with the time for 6% bentonite drilling mud under the effect of downward salt contamination.

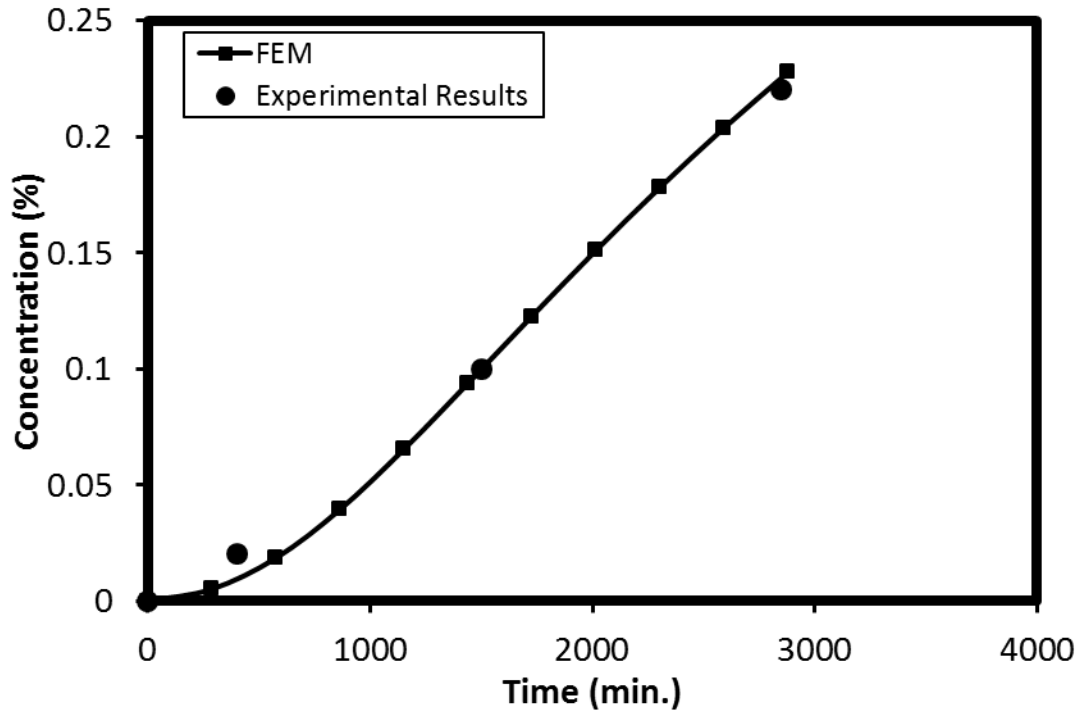


Figure 6.51 Variation of salt concentration with the time in 6% bentonite drilling mud using experimental results and finite element analysis.

6.8.Summary

In this chapter, the following points can be summarized:

- 1.As the bentonite content increased, the density increased linearly. The density increased by 1.65% as the salt content increased from 0% to 3%.
- 2.When the temperature changed from 5°C to 85°C, the density changed by 0.4% and 3% for 0% and 3% salt contents respectively.
- 3.As the salt content was increased to 3%, the pH decreased, possibly due to the interaction between bentonite and salt. The pH reduced from 8.8 to 8.2 with 3% salt contamination with a change of 7 %.
- 4.When the salt content increased from 0.1% to 3%, the average decrease in the electrical resistivities were 1600% and 918% for 2% and 10% bentonite content respectively.

5. When the temperature increased from 5° C to 85° C, the electrical resistivity decreased by 130% and 354% for 0% salt content and 870% and 1328% for 3% salt content of 2% and 10% bentonite content respectively.
6. All tested bentonite drilling mud with and without salt contamination showed pure resistance behavior at high measured frequency (300 kHz) where the effect of contact resistance is minor and negligible.
7. The downward movement of salt was higher than the upward which clearly captured by the change in electrical resistivity of the drilling mud.
8. As the salt increased from 0% to 3%, the CIGMAT penetration of 2% and 10% drilling mud increased by 35% to 70% respectively.
9. As the salt content increased from 0% to 3%, the changes in the electrical resistivity decreased by 98%, and 94% for 2% and 10% bentonite drilling mud respectively.
10. As the salt content increased from 0% to 3%, the shear strength decreased by 75%, and 22% for 2% and 10% bentonite drilling mud respectively.
11. Based on the statistical analysis results, the shear strength-salt model had best prediction for 6% and 10% bentonite content and lowest prediction for 2% bentonite content.
12. Based on the statistical analysis results, the electrical resistivity-salt model had best prediction for 10% bentonite content and lowest prediction for 2% bentonite content.
13. Based on the statistical analysis results and for 2% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 1% salt content and lowest prediction for 0% salt content.

14. Based on the statistical analysis results and for 10% bentonite content, the CIGMAT penetrometer penetration-weight model had the best prediction for 0.1% salt content and lowest prediction for 1% salt content.
15. A bulk resistivity decreased by 46% when the bentonite content increased from 2% to 8% with 3% salt contamination.
16. When the temperature increased from 25° C to 75° C in 2% bentonite drilling mud with 0% salt contamination, the yield stress was the same (0.6 Pa) while the parameters k_1 and n values decreased from 0.2 (Pa.s) to 0.125 (Pa.s) and 0.52 to 0.5 respectively.
17. When the temperature increased from 25° C to 75° C in 2% bentonite drilling mud with 1% salt contamination, the yield stress increased from 0.1 Pa to 0.2 Pa while the parameters k_1 and n values reduced from 0.2 (Pa.s) to 0.15 (Pa.s) and 0.48 to 0.46 respectively.
18. When the temperature increased from 25° C to 75° C in 2% bentonite drilling mud with 3% salt contamination, the yield stress increased from 0.2 Pa to 0.5 Pa while the parameters k_1 and n values reduced from 0.19 (Pa.s) to 0.10 (Pa.s) and 0.46 to 0.45 respectively.
19. When the temperature increased from 25° C to 75° C in 8% bentonite drilling mud with 0% salt contamination, the yield stress was the same (35 Pa) while the parameters k_1 and n values reduced from 7 (Pa.s) to 5 (Pa.s) and 0.44 to 0.43 respectively.
20. When the temperature increased from 25° C to 75° C in 8% bentonite drilling mud with 1% salt contamination, the yield stress decreased from 4.4 Pa to 1.2 Pa while the parameter k_1 value increased from 5 (Pa.s) to 7 (Pa.s) and the parameter n value decreased from 0.40 to 0.27 respectively.

21. When the temperature increased from 25° C to 75° C in 8% bentonite drilling mud with 3% salt contamination, the yield stress increased from 1.2 Pa to 1.3 Pa while the parameters k_1 and n values reduced from 5 (Pa.s) to 3.2 (Pa.s) and 0.31 to 0.27 respectively.
22. The maximum shear stress ($\tau_{\max}$) decreased from 8.9 Pa to 5.2 Pa when the temperature was increased from 25° C to 75° C for 2% bentonite drilling mud with 0% salt contamination respectively.
23. Adding 1% salt to the 2% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 26% and 15% at 25° C and 75° C respectively.
24. Adding 3% salt to the 2% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 38% and 33% at 25° C and 75° C respectively.
25. The maximum shear stress ($\tau_{\max}$) decreased from 167.7 Pa to 124 Pa when the temperature was increased from 25° C to 75° C for 8% bentonite drilling mud with 0% salt contamination respectively.
26. Adding 1% salt to the 8% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 50% and 65% at 25° C and 75° C respectively.
27. Adding 3% salt to the 8% bentonite drilling mud decreased the maximum shear stress ($\tau_{\max}$) by 73% and 80% at 25° C and 75° C respectively.
28. It is clearly indicated the contribution of bentonite content to have the highest impact on the $(\tau_Y)_0$ and R and S parameters while the contribution of the combined effect of bentonite content and temperature was the minimal.

29. It is clearly indicated the contribution of bentonite content to have the highest impact on the $(\tau_{\max})_o$ and G and H parameters while the contribution of the combined effect of bentonite content and temperature was the minimal.

30. When the temperature increased from 25° C to 75° C for 2% bentonite drilling mud, the electrical resistivity decreased from 5.6 Ohm.m to 3 Ohm.m, 2.21 Ohm.m to 0.55 Ohm.m and 1.73 Ohm.m to 0.2 Ohm.m at 0%, 1% and 3% salt contamination contents respectively.

31. When the temperature increased from 25° C to 75° C for 8% bentonite drilling mud, the electrical resistivity decreased from 2.3 Ohm.m to 0.75 Ohm.m, 1.43 Ohm.m to 0.22 Ohm.m and 1.12 Ohm.m to 0.083 Ohm.m at 0%, 1% and 3% salt contamination contents respectively.

32. Based on NLM, bentonite content had the highest effect on decreasing the electrical resistivity compared to salt contamination and temperature. NLM showed that the salt contamination effect was 2.66 times the temperature effect in reducing the electrical resistivity.

33. Based on NLM, bentonite content had the highest effect on increasing the yield shear stress while the salt contamination and temperature has the tendency to decrease the yield shear stress. NLM showed that the salt contamination effect was 3 times the temperature effect in reducing the yield shear stress.

34. Based on NLM, bentonite content had the highest effect on increasing the maximum shear stress while the salt contamination and temperature has the tendency to decrease the maximum shear stress. NLM showed that the salt contamination effect was 2.8 times the temperature effect in reducing the maximum shear stress.

35. Based on NLM, temperature had the highest effect on decreasing the parameter- A_4 compared to the bentonite content and salt contamination. NLM showed that the bentonite content effect was 3.3 times the salt contamination effect in reducing the parameter- A_4 .
36. Based on NLM, bentonite content had the highest effect on decreasing the parameter- B_4 compared to the salt contamination and temperature. NLM showed that the temperature effect was 1.74 times the salt contamination effect in reducing the parameter- B_4 .
37. Generally, yield shear stress increased as the bentonite content and resistivity increased.
38. Generally, maximum shear stress increased as the bentonite content and resistivity increased.
39. Parameter- A_4 decreased as the bentonite content and resistivity increased.
40. Parameter- B_4 decreased as the bentonite content and resistivity increased.
41. Finite element result agreed very well with the experimental results for the prediction of the salt concentration over the time.

Chapter 7 - Conclusions and Recommendations

In this study ultra-soft soils representing the deepwater seabed offshore, coastal soils, and onshore soils with filter cake and drilling mud were characterized using new non-destructive in-situ test methods and modeling of the behavior. The new test methods include the two-probe electrical method and CIGMAT miniature penetrometer. The clay content in the ultra-soft soil, filter cake and drilling mud varied from 2% to 10% by weight. The type of clays investigated include montmorillonite (bentonite) and kaolinite. The shear strength of the ultra-soft soil varied from 0.01 ± 0.0001 kPa to 0.30 ± 0.0002 kPa using the modified vane shear test. Lime, polymer, cement, sand, and kaolinite were used to check their effectiveness in improving the properties of the bentonite ultra-soft soil. A CIGMAT miniature penetrometer was developed for ultra-soft soil characterization. In addition, this research focused on modeling the behavior of filter cake for deep oil well characterization. Finally, a combined procedure for both experimental and nonlinear models were developed to characterize the behavior of the water based drilling mud under the effect of both contamination and temperature.

Based on the experimental and analytical results of this study, the following conclusions are advanced

1. Based on the electrical characterization of impedance versus frequency, the ultra-soft soil and drilling mud were characterized as resistive materials. For the bentonite based ultra-soft soil, when the bentonite content was increased from 2% to 10%, the electrical resistivity decreased from 6.29 ± 0.001 Ohm.m to 2.56 ± 0.0005 Ohm.m respectively, a 59% decrease in resistivity. The shear strength increased from 0.01 ± 0.0001 kPa to

0.17±0.0002 kPa as the bentonite contents increased from 2% to 10%, respectively, a 1600% increase in the shear strength.

2.The ultra-soft soil with 10% bentonite was treated with different modifiers such as lime, polymer, cement, sand, and kaolinite. Based on the electrical characterization of impedance versus frequency, the ultra-soft soil after treatment is also behaved as resistive materials. Polymer had the highest impact compared to other modifiers in increasing the shear strength of the ultra-soft soil. The shear strength of the 10% bentonite ultra-soft soil treated with 10% polymer increased from 0.17 kPa to 6.8 kPa, a 3900% increase in the shear strength.

3.Relative electrical resistivity decreased by 246% when the bentonite content was increased from 2% to 10% in the ultra-soft soil. Addition of 10% of lime to the ultra-soft soil with 10% of bentonite content decreased the relative electrical resistivity by 171%. Addition of 10% of polymer to the ultra-soft soil with 10% of bentonite content reduced the relative electrical resistivity by 545%.

4.Based on the analytical results, electrical resistivity was correlated strongly with different properties of untreated and treated ultra-soft soil such as shear strength, water content, and solid content. Power law, linear and hyperbolic models were used to predict the shear strength-resistivity relationship for the untreated, lime-treated and polymer-treated ultra-soft soils respectively. In addition, linear model was used to predict the water content relationship with electrical resistivity for the untreated and treated ultra-soft soils.

5.The CIGMAT miniature penetrometer penetration varied linearly with the shear strength of the untreated and treated soft soils with 10% bentonite. Increasing the

weight from 7 gm to 28 gm increased the penetration from 20 mm to 35 mm, a 75% increase in penetration depth for ultra soft soil of 0.17 kPa in shear strength (10% bentonite content). Increasing the weight from 7 gm to 28 gm increased the penetration from 15 mm to 25 mm, a 67% increase in penetration depth for ultra soft soil of 0.27 kPa in shear strength (2% lime treatment). Increasing the weight from 7 gm to 28 gm increased the penetration from 1 mm to 6 mm, a 500% increase in penetration depth for ultra soft soil of 6.8 kPa in shear strength (10% polymer treatment).

6. A new coupling continuous time and depth function is developed to represent the seepage-consolidation phenomenon during filter cake formation under different pressure and temperature. The new continuous function solution is compared with Terzaghi discrete consolidation solution and both solutions were verified using several experimental results.

7. A new kinetic model was established to address the API-model shortcomings. The proposed kinetic model has a limit on the total amount of the fluid loss, whereas the API-model has no limit on the fluid loss and the fluid loss increased proportionally to the squared root of the time. In the new kinetic model, the variations of the fluid loss, porosity, permeability, and solid content in the cake to the solid content in the mud with time have been included.

8. Increasing the salt contamination and temperature of the drilling mud decreased the electrical resistivity. When the salt content increased from 0.1% to 3%, the average decrease in the electrical resistivities were 1600% and 918% for 2% and 10% bentonite content respectively. When the temperature increased from 5° C to 85° C, the electrical

resistivity decreased by 130% and 354% for 0% salt content and 870% and 1328% for 3% salt content of 2% and 10% bentonite content respectively.

9. Based on NLM, bentonite content had the highest effect on decreasing the electrical resistivity, yield shear stress, and maximum shear stress compared to salt contamination and temperature. NLM showed that the salt contamination effects were 2.66, 3, and 2.8 times the temperature effect in reducing the electrical resistivity, yield shear stress, and maximum shear stress, respectively.

The recommendations for the future work of this study are as

1. Perform field study for soft soil characterization on onshore and offshore soil conditions. Apply CIGMAT penetrometer for soft soil characterization in the field to quantify the shear strength using shear-penetration correlation.
2. Monitor the soft soil condition in the field under different temperature changes using electrical resistivity method for short and long term measurement.
3. Apply lime, polymer, and cement as improving agents to study their effectiveness for long-term condition. Test the improving agents for interface conditions, especially for pipe soil interaction in horizontal and vertical situations.
4. Prepare a setup for two and three dimensional consolidation condition during filter cake formation. Test the filter cake formation in the three dimensional setup under different pressures and temperatures. Analyze the problem using finite element scheme to model the stress distribution and fluid loss during filter cake formation.
5. Test the drilling mud under high pressure, high temperature condition with the effect of different contaminations such as oil, cement and salt.

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