

Comparative Evaluation of Saturated and Partially Saturated Clay Soils in Hilla City, Babylon Governorate: Implications on Primary Consolidation Behavior

Samhar A. Rida^{1*} , Raid A. Mahmood¹  and Amer A. Al-Khalidy² 

¹ Department of Geology, College of Science, University of Basrah, Basrah, Iraq

² Department of Applied Geology, College of Science, University of Babylon, Babylon, Iraq

³ Correspondence: Pgs.samhar.amer@uobasrah.edu.iq

Received:

21 November 2025

Accepted:

10 May 2026

Published:

30 June 2026

Abstract

The velocity of settlement can lead to excessive deformations, cracking, or even structural collapse, making it a crucial consideration in geotechnical design. Settlement can continue for years or even decades in clayey soils. Assuming constant permeability, Terzaghi's one-dimensional consolidation theory explains the time-dependent settling behavior. The impact of partial saturation on this behavior, however, has not gotten as much attention. This study compares fully saturated clayey soils with partially saturated soils at a 50% degree of saturation to examine the impact of soil saturation on consolidation characteristics. Five boreholes were selected in Hilla City, Babylon Governorate. Both boreholes are used for laboratory and field research, as well as for physical, geotechnical, and chemical testing, in compliance with British and ASTM standards. The results show that the consolidation settling range for fully saturated clayey soils is 0.75 to 3.26 cm, while the range for moderately saturated soils ($S = 50\%$) is 0.68 to 2.59 cm. Reduced permeability and increased matric suction are the reasons for the observed reduction in settlement during partial saturation. These findings show how crucial it is to account for saturation conditions when evaluating consolidation behavior and planning ground improvement to ensure the long-term stability and functionality of structures constructed on clayey deposits.

Keywords: Consolidation; Clayey Soils; Permeability; Settlement; Fully Saturated; Partially Saturated

1. Introduction

Soils are frequently idealized in traditional soil mechanics as either totally dry or entirely moist. However, a lot of real geotechnical problems include partially saturated soils, in which the spaces are occupied by both water and air. This dual-phase state has a major effect on soil permeability, compressibility, and general deformation behavior. As a result, there are substantial differences in the mechanical reactivity of partly and fully damp soils. This study investigates these differences by examining the consolidation behavior of fully and partially saturated clayey soils at 50% degree of saturation. To ensure accurate structural design and minimize long-term settlement-related geotechnical problems, the settlement analysis accounts for changes in the permeability coefficient during partial saturation. A soil mass undergoes volumetric compression when subjected to additional stress. This can happen when pore water and/or pore air are expelled. Consolidation, according to Terzaghi (1943), is the decrease in a saturated soil's water content without the water being replaced by air. When an external

force is applied to saturated clays, the pore water pressure rises instantly. Settlement results from excess pore pressures gradually dissipating over time if drainage is allowed.

The poor permeability of fine-grained soils is responsible for the time-dependent character of consolidation. On the other hand, because of their high permeability, coarse-grained soils settle quickly and have fast pore pressure dissipation (Das, 2006; Karkush and Ali, 2023). Babylon Governorate's summers are marked by high temperatures and high evaporation rates, which lower groundwater levels and increase salinity. In winter, increased rainfall elevates soil saturation and groundwater levels, potentially leading to greater settlement. Thus, the region's alternating wet and dry seasons directly influence groundwater fluctuations, soil saturation, and, consequently, the consolidation behavior of Hilla City clays, Babylon Governorate.

Several researchers modified a traditional oedometer and performed consolidation tests on soil samples under various suctions, including Barden and Sides (1970), Fredlund and Morgenstern (1977), Fredlund and Rahardjo (1986), Escario and Juca (1989), Rahardjo and Fredlund (1995), and Rahardjo and Fredlund (1996). They created the desired suction in the sample using the axis-translation technique; the sample's lateral deformation was limited in this test, and the volume change was measured solely vertically. Al-Khakani (2006) studied swelling problems in clay soils in Hilla city based on results from laboratory and field tests of samples collected from 15 locations. The results indicate a low-to-high degree of swelling for Hilla City. Namah (2014) studied the prediction of consolidation settlement in saturated and unsaturated soils from laboratory tests in the Village of Musaiab, City of Babylon, Babylon Governorate, and noted that soil settlement is strongly influenced by the degree of saturation. Settlement was studied at several soil saturation levels (100%, 91%, 85%, 75%, and 60%). Mathematical relationships were found linking these parameters (compressibility coefficient C_c , swelling coefficient C_s , volumetric compressibility m_v , and coefficient of consolidation C_v) with the degree of saturation. Salih et al. (2022) described soil composition in Hilla City, Babylon Governorate, by collecting geotechnical reports that focused on physical and mechanical properties. The regression results showed that the soil is cohesive to a depth of 15 meters, and below that it is non-cohesive. There is a direct correlation between the characteristics.

Al-Amar and Hadi (2018) assessed the soil characteristics of the banks along a portion of the Hilla River, Babylon Governorate, to determine their stability and susceptibility to erosion. To describe soil behavior under hydrological and environmental conditions, a number of geological and geotechnical tests were carried out and compared with engineering standards. The findings demonstrate the necessity of technical solutions that pinpoint areas of mechanical property fluctuations and relative susceptibility in order to preserve bank stability. Al-Amar (2024) provided a thorough geological and geotechnical description of the soils of the Babylon Governorate's Hilla District. Soil categorization, strength characteristics, compaction behavior, and engineering compatibility were all determined through laboratory and field investigations. The results encourage safer, more effective construction in the area by offering crucial information for infrastructure planning and foundation design. Al-Khalidi (2018) used the downhole seismic technique to study subsurface rock and soil properties in the South Baghdad region. To assess soil behavior under seismic loading, dynamic soil metrics such as elastic moduli, shear wave velocity, and site categorization were calculated. The findings offer vital information for seismic hazard assessment, geotechnical engineering design, and foundation suggestions for upcoming building projects in the area. Shakir et al. (2013) created a more accurate shear-wave velocity model for subsurface profiling in Najaf City, Iraq, using laterally constrained inversion (LCI) on surface-wave data. The technique reduces artifacts and enhances the resolution of lateral soil changes by improving continuity between neighboring 1D inversions. The findings support enhanced geotechnical evaluation and seismic site categorization for engineering and earthquake-related applications. Al-Khalidy (2025) described the underlying soil characteristics at the Al-Debony Power Electrical Station in Iraq using Vertical Electrical Sounding (VES). In order to determine soil layers, moisture zones, and weak

strata—all of which are essential for foundation planning—electrical resistivity data were analyzed. The findings offer crucial geotechnical information to improve foundation depth, lower construction hazards, and build safe structures in order to construct and optimize grounding systems at the Diwaniyah electrical power station site in Iraq, Kadhim et al. (2025) looked into soil electrical resistivity. To categorize soil types, evaluate the impacts of moisture, and detect conductivity changes that impact electrical safety and grounding performance, field resistivity tests were utilized. The findings offer helpful suggestions for improving grounding effectiveness, lowering fault risks, and guaranteeing the dependability of the site's electrical infrastructure.

This study aims to compare fully saturated clayey soils with partially saturated soils at 50% degree of saturation to examine the impact of soil saturation on consolidation characteristics in Hilla City, Babylon Governorate, applying physical, geotechnical, and chemical testing, in compliance with British and ASTM standards.

The City of Hilla is located in the Babylon governorate, central Iraq, and is bounded by latitudes 32°26'32.0-32°28'55.9 N and longitudes 44°24'00.2- 44°24'07.8 E (Fig. 1). Five boreholes, each three meters deep, were drilled. Borehole coordinates are shown in Table 1.

Table 1. Coordinates of boreholes in the study area

Boreholes No. and name of districts	Latitude (N)	Longitude (E)
Aleaskari District	32°26'32.0	44°24'00.2
Almuhandisin District	32°27'16.1	44°24'07.1
Al-Akramin District	32°27'25.8	44°23'49.8
Muealimin District	32°28'01.7	44°23'18.5
Alshuhada' District	32°28'55.9	44°24'07.8

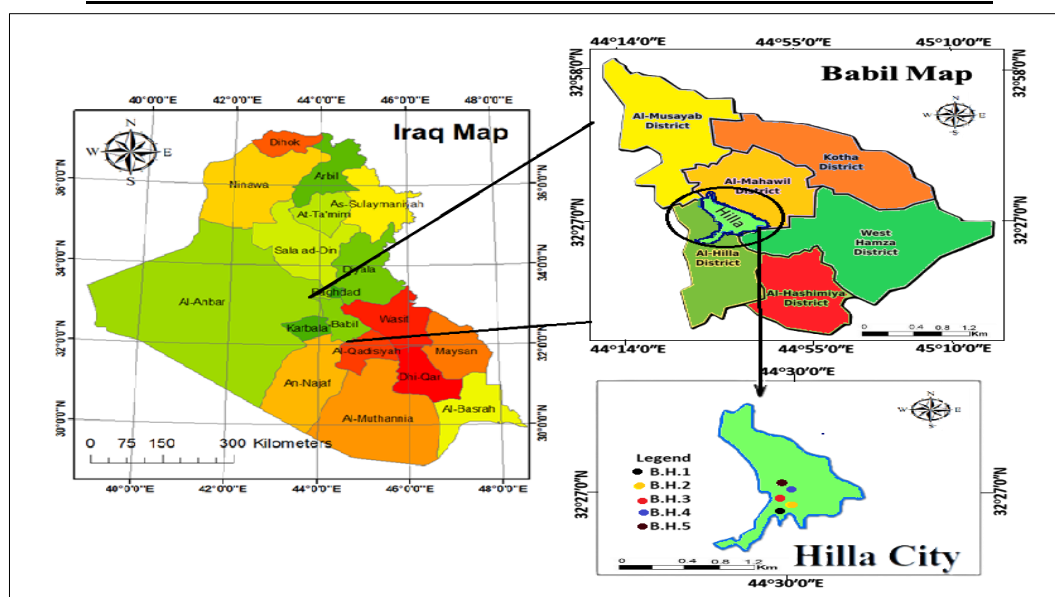


Fig. 1. Location of the study area

Geologically, the city of Hilla lies on the Unstable Shelf of the tectonic subdivision of Iraq, forming part of the Mesopotamian foredeep basin (Buday and Jassim, 1984). This zone, known as the Mesopotamian Zone, lies between the Zagros Fold-Thrust Belt to the northeast and the Stable Arabian Platform to the southwest (Al-Hiti, 1985). The study area is mainly covered by Quaternary deposits ranging in age from the Pleistocene to the Recent. The floodplain sediments are predominantly derived from the Tigris and Euphrates Rivers and their distributaries. The area is not affected by direct erosion of older bedrock units; rather, it is a wide depression infilled with alluvial deposits from repeated flooding

events. These deposits are typically composed of soft, fine-grained materials such as fine sand, silt, clay, and silty clay (Al-Rubaiee and Al-Owaidi, 2024).

2. Materials and Methods

Five rotary-drilled boreholes (BH1 to BH5) to 3 m depth were executed across Hilla City. Elevations were leveled at each site. Groundwater levels were measured per ASTM D4750 after 24 h. Sampling followed ASTM D1452/D5783, with 0.5 m intervals. Disturbed samples (ASTM D1586) and undisturbed samples (ASTM D1587) were sealed and transported under controlled conditions. Plate 1 shows field photographs.



Plate 1. Photos from the fieldwork stage

Upon receipt, Shelby tubes were extruded using a mechanical extruder. Tests were conducted at the University of Babylon (Applied Geology) and Al-Ma'awal Soil Testing Division in accordance with ASTM and BS 1377 standards (Standard, B., 1990-1377).

2.1. Physical Tests

The conducted physical tests are for water content (w) & degree of saturation (S_r) (ASTM D2216), specific gravity (G_s) (ASTM D854), Atterberg's limits (ASTM D4318), grain size (ASTM D6913/D7928), unit weight (ASTM D7263); void ratio (e) and porosity (n) computed from γ_d , specific gravity (G_s), and water content (w).

2.2. Engineering Tests

The conducted engineering tests are Oedometer consolidation per ASTM D2435 on saturated specimens and $S_r \approx 50\%$ reconditioned specimens (Smith₇ and Smith., 1998); parameters: e_0 , p_c , C_c , C_s , a_v , m_v , E_{oed} , E , C_v , and k . UU triaxial (ASTM D2850), Modified Proctor (ASTM D1557), permeability per ASTM D2434/D5084. Note: Terzaghi consolidation applies strictly to saturated soils; partially saturated soils are interpreted comparatively (Saba, 2023).

2.3. Chemical Tests

The conducted chemical tests include organic matter, sulfates, chlorides, pH, total soluble salts, gypsum, and carbonate content (Standard, B., 1990-1377). All the tests depend on American Society for Testing and Materials (ASTM) standards and British Standards (ASTM, 1985).

3. Results

3.1. Soil Physical Test Results

Physical examinations were conducted for all the disturbed and undisturbed models, and the results are shown in Tables 2, 3, 4, 5, and 6.

Table 2. Physical soil test results for BH1

BH No.	Aleaskari District					
Depth(m)	0-0.5	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3
Type of samples	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed
m.c%	21	22	21	22	24	26
Sand%	38.3	38.8	31.6	28.8	21.2	18.1
Silt%	31.4	29.6	31.3	33.2	23.3	20.3
Clay%	30.3	31.6	37.1	38	55.5	61.6
L.L%	36	44	37	46	37	53
P.L%	22	23	26	25	20	23
P.I%	14	21	11	21	17	30
I.C%	1.07	1.05	1.45	1.14	0.76	0.9
Consistency	Solid	Solid	Solid	Solid	plasticity	plasticity
A%	0.46	0.66	0.3	0.55	0.31	0.49
S%	75	71	65	82	70	81
e	0.77	0.85	0.88	0.74	0.94	0.89
n%	43	46	47	43	49	47
γ_{wet} (gm/cm ³)	1.85	1.78	1.76	1.88	1.72	1.76
γ_{dry} (gm/cm ³)	1.55	1.48	1.46	1.58	1.42	1.46
Gs	2.74	2.74	2.75	2.75	2.76	2.76
USCS	CL	CL	ML	CL	CL	CH

Table 3. Physical soil test results for BH2

BH No.	Almuhandisin District					
Depth(m)	0-0.5	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3
Type of samples	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed
m.c%	18	15	16	19	19	19
Sand%	49.8	41.5	38.9	34.2	27.7	22.1
Silt%	12.8	13.2	20.9	18.9	25.3	23.8
Clay%	35.4	45.3	40.2	46.9	47	54.1
L.L%	36	42	32	51	37	52
P.L%	18	19	18	22	18	20
P.I%	18	23	14	29	19	30
I.C%	1	1.17	1.14	1.1	0.95	1.1
Consistency	Solid	Solid	Solid	Solid	plasticity	Solid
A%	0.51	0.51	0.35	0.62	0.4	0.55
S%	61	54	58	64	59	66
e	0.81	0.76	0.76	0.81	0.89	0.79
n%	45	43	43	45	47	44
γ_{wet} (gm/cm ³)	1.81	1.85	1.86	1.81	1.75	1.83
γ_{dry} (gm/cm ³)	1.51	1.55	1.56	1.51	1.45	1.53
Gs	2.73	2.73	2.74	2.74	2.74	2.74
USCS	CL	CL	CL	CH	CL	CH

Table 4. Physical soil test results for BH3

BH No.	3- Al-Akramin District					
Depth(m)	0-0.5	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3
Type of samples	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed
m.c%	26	27	15	16	18	19
Sand%	24.9	23.1	15	11	8.9	6.8
Silt%	38	32	32.7	34.2	40.8	41
Clay%	37.1	44.9	52.3	54.8	50.3	52.2
L.L%	33	40	32	52	30	53
P.L%	24	24	22	22	21	21
P.I%	9	16	10	30	9	32
I.C%	0.78	0.81	1.7	1.2	1.33	1.06
Consistency	plasticity	plasticity	Solid	Solid	Solid	Solid
A%	0.24	0.36	0.19	0.55	0.18	0.61
S%	71	79	59	50	55	62
e	0.99	0.93	0.7	0.87	0.9	0.84
n%	50	48	41	47	47	46
ywet (gm/cm ³)	1.66	1.71	1.91	1.76	1.74	1.79
ydry (gm/cm ³)	1.36	1.41	1.61	1.46	1.44	1.49
Gs	2.71	2.72	2.73	2.73	2.73	2.74
USCS	ML	CL	CL	CH	CL	CH

Table 5. Physical soil test results for BH4

BH No.	4- Muealimin District					
Depth(m)	0-0.5	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3
Type of samples	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed
m.c%	21	21	14	15	16	16
Sand%	18.7	15.8	15.4	13.3	11.7	7.3
Silt%	32.7	33.2	34.5	31.9	39.7	29.7
Clay%	49	51	50.3	54.8	48.6	63
L.L%	45	52	43	53	38	55
P.L%	23	23	20	20	19	19
P.I%	22	29	23	33	19	36
I.C%	1.09	1.07	1.26	1.15	1.16	1.08
Consistency	Solid	Solid	Solid	Solid	Solid	Solid
A%	0.45	0.57	0.46	0.6	0.39	0.57
S%	82	82	50	55	67	64
e	0.7	0.7	0.76	0.75	0.66	0.69
n%	41	41	43	43	40	41
ywet (gm/cm ³)	1.91	1.91	1.86	1.87	1.96	1.93
ydry (gm/cm ³)	1.61	1.61	1.56	1.57	1.66	1.63
Gs	2.74	2.74	2.75	2.75	2.75	2.75
USCS	CL	CH	CL	CH	CL	CH

Table 6. Physical soil test results for BH5

BH No.	5 - Alshuhada' District					
Depth(m)	0-0.5	0.5-1	1-1.5	1.5-2	2-2.5	2.5-3
Type of samples	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed
m.c%	18	20	22	23	22	23
Sand%	17.3	16.2	7.8	5.3	1.4	-
Silt%	30	27.9	25.4	26.2	35.7	36.6
Clay%	52.7	55.9	66.8	68.5	62.9	63.4
L.L%	58	55	51	56	53	56
P.L%	24	23	23	26	25	25
P.I%	34	32	28	30	28	31
I.C%	1.18	1.09	1.04	1.1	1.11	1.06
Consistency	Solid	Solid	Solid	Solid	Solid	Solid
A%	0.65	0.57	0.42	0.44	0.45	0.49
S%	76	64	62	89	69	79
e	0.65	0.86	0.98	0.71	0.88	0.8
n%	39	46	49	42	47	45
γ _{wet} (gm/cm ³)	1.96	1.78	1.69	1.91	1.77	1.83
γ _{dry} (gm/cm ³)	1.66	1.48	1.39	1.61	1.47	1.53
G _s	2.74	2.75	2.75	2.76	2.76	2.76
USCS	CH	CH	CH	CH	CH	CH

3.2. Soil Engineering Test Results

Engineering tests were performed on all borehole samples obtained from the study area, with detailed results summarized in Tables 7, 8, and 9.

Table 7. Soil Engineering test results (consolidation (fully saturated soil))

B H N o.	Dep th (m)	e ₀	p _c (kN/ m ²)	P ₀ (kN/ m ²)	OC R	Ty pe of OC R	C _c	C _s	av *10 ⁻³ (kN/ m ²)	E (kN/ m ²)	mv*1 0 ⁻³ (kN/ m ²)	E _o (kN/ m ²)	√t 90 (mi n)	C _v *10 ⁻³ (Cm ² /s ec)	K*10 ⁻⁸ (cm/s ec)
Aleaskari district	0.5-1	0.86	90	-	-	-	0.214	0.024	7.3	136	3.9	254	3.8	0.98	38.7
	1.5-2	0.98	88	-	-	-	0.239	0.029	7.9	127	4.0	251	3.4	1.22	48.7
	2.5-3	0.86	105	158.8	0.66	UC	0.276	0.027	8.7	115	4.7	213	3.3	1.3	60.9
Almuhandisin District	0.5-1	0.76	105	-	-	-	0.184	0.045	8.0	125	4.6	220	8.7	0.19	8.51
	1.5-2	0.54	95	-	-	-	0.354	0.023	9.3	107	6.1	164	8.5	0.2	11.9
	2.5-3	0.63	150	167.1	0.90	UC	0.115	0.022	3.3	305	2.0	497	6.5	0.34	6.74
Alakirmin District	0.5-1	0.72	150	-	-	-	0.106	0.088	2.0	500	1.3	861	8	0.22	2.87
	1.5-2	0.67	200	-	-	-	0.122	0.091	6.5	154	3.9	258	8.4	0.20	7.78
	2.5-3	0.76	160	158.4	1.01	OC	0.172	0.018	2.4	417	1.4	735	7.6	0.25	3.33
Almuealimin District	0.5-1	0.79	180	-	-	-	0.127	0.066	7.2	138	4.0	248	6	0.39	15.9
	1.5-2	0.86	185	-	-	-	0.137	0.034	8.1	123	4.4	230	5.2	0.52	22.8
	2.5-3	0.89	125	172.5	0.72	UC	0.167	0.006	1.5	667	0.8	1262	5	0.57	4.48
Alshuhada' District	0.5-1	0.87	90	-	-	-	0.261	0.046	3.4	294	1.8	549	7	3.53	5.26
	1.5-2	0.91	95	-	-	-	0.304	0.038	2.3	435	1.2	830	6	0.39	4.73
	2.5-3	1.01	110	167.1	0.66	UC	0.283	0.030	3.1	323	1.5	649	5.5	0.47	7.20

Table 8. Soil Engineering test results (Consolidation (Partially saturated soil 50%))

BH No.	Depth (m)	e_0	p_c (kN/m ²)	P_0 (kN/m ²)	OCR	Type of OCR	C_c	C_s	$a_v \cdot 10^{-3}$ (kN/m ²)	E (kN/m ²)	$mv \cdot 10^{-3}$ (kN/m ²)	E_0 (kN/m ²)	$\sqrt{t_{90}}$ (min)	$C_v \cdot 10^{-3}$ (cm ² /s)	$K \cdot 10^{-7}$ (cm/s)
Aleaskari District	0.5-1	0.415	70	-	-	-	0.109	0.016	4.30	232	3.04	329	3.3	1.30	3.95
	1.5-2	0.770	62	-	-	-	0.206	0.021	5.52	181	3.12	321	3	1.57	4.90
	2.5-3	0.407	70	158.77	0.44	UC	0.144	0.022	6.10	164	4.34	231	3	1.57	6.81
Almuhan disin District	0.5-1	0.468	65	-	-	-	0.120	0.015	4.58	218	3.12	320	4.9	0.75	2.34
	1.5-2	0.453	70	-	-	-	0.182	0.021	8.00	125	5.51	182	3.3	1.30	7.15
	2.5-3	0.434	80	167.06	0.48	UC	0.093	0.004	1.68	594	1.17	852	2.2	2.92	3.43
Alakirmin District	0.5-1	0.618	90	-	-	-	0.092	0.073	2.00	500	1.24	890	4.8	0.61	0.76
	1.5-2	0.623	95	-	-	-	0.103	0.069	5.20	192	3.20	312	5	0.90	2.88
	2.5-3	0.605	80	158.39	0.51	OC	0.122	0.015	2.10	476	1.31	764	5.1	0.55	0.72
Almuealimin District	0.5-1	0.559	70	-	-	-	0.111	0.033	6.32	158	3.95	247	6	2.40	9.73
	1.5-2	0.255	80	-	-	-	0.125	0.025	3.10	323	2.47	405	3.3	1.30	3.21
	2.5-3	0.635	85	172.51	0.49	UC	0.145	0.005	1.20	834	0.73	1364	3.4	1.22	0.89
Ishuhada' District	0.5-1	0.494	70	-	-	-	0.060	0.005	1.10	909	0.74	1358	2	3.60	2.60
	1.5-2	0.520	70	-	-	-	0.260	0.010	1.70	588	1.12	894	4	1.05	1.17
	2.5-3	0.526	80	167.06	0.48	UC	0.273	0.019	1.60	625	1.05	954	4	0.88	0.93

Table 9. Soil Engineering test results

BH.No.	Depth (m)	K (cm/sec)	C kN/m ²	ϕ	Depth (m)	Max ydry (gm/cm ³)	O.M. C %	P ₀ kN/m ²	u _w kN/m ²	P kN/m ²	(S) cm	(S) for 50% Sr(cm)
Aleaskari District	0.5-1	4.41*10	-	-	0-1	-	-	-	-	-	-	-
	1.5-2	7.66*10	-	-	1-2	1.75	16	-	-	-	-	-
	2.5-3	3.96*10	56.9	15.	2-3	-	-	158.	24.52	183.3	3.2	2.18
Almuhandisi District	0.5-1	9.8*10 ⁻⁵	-	-	0-1	-	-	-	-	-	-	-
	1.5-2	4.37*10	51.5	5.6	1-2	-	-	-	-	-	-	-
	2.5-3	9.61*10	-	-	2-3	1.74	18	167.	19.62	186.6	1.4	1.30
Alakirmi District	0.5-1	9.03*10	-	-	0-1	-	-	-	-	-	-	-
	1.5-2	9.51*10	-	-	1-2	1.83	15	-	-	-	-	-
	2.5-3	9.61*10	30.5	5.8	2-3	-	-	158.	24.52	182.9	0.7	0.68
Almuealimi n District	0.5-1	9.7*10 ⁻⁶	-	-	0-1	-	-	-	-	-	-	-
	1.5-2	2.94*10	52	2.7	1-2	1.87	17	-	-	-	-	-
	2.5-3	1.98*10	-	-	2-3	-	-	172.	24.52	197.0	1.5	1.18
Alshuhada' District	0.5-1	9.9*10 ⁻⁵	-	-	0-1	-	-	-	-	-	-	-
	1.5-2	6.3*10 ⁻⁵	-	-	1-2	1.82	18	-	-	-	-	-
	2.5-3	6.44*10	26	16.	2-3	-	-	167.	19.62	186.6	2.9	2.59

3.3. Soil Chemical Test Results

Chemical tests were conducted on 5 boreholes in the study area, with a total of 15 samples collected at a rate of 3 per meter depth per borehole. The results were evaluated in accordance with the British Standard (B.S.1377–1990). The detailed outcomes of the chemical analyses are presented in Table 10.

Table 10. Soil chemical test results.

B.H.	depth	Org.%	SO ₃ %	CL%	PH	T.S.S %	Gyp. %	CaCO ₃ %
Aleaskari District	0-1	0.07	0.9	0.035	8.03	1.39	1.94	4.5
	1-2	0.12	0.39	0.031	8.4	1.79	0.84	4.8
	2-3	0.53	1.103	0.028	8.12	3.02	2.37	4.85
Almuhandisin District	0-1	0.07	1.88	0.071	7.82	2.42	4.04	7.01
	1-2	0.16	0.77	0.068	8	1.31	1.66	12.1
	2-3	0.07	0.35	0.051	8.1	0.98	0.75	11.3
Alakirmin District	0-1	0.25	0.72	0.042	8.1	1.08	1.55	6.3
	1-2	0.05	0.56	0.04	8.12	0.7	1.20	6.4
	2-3	0.13	0.31	0.037	8.11	0.64	0.67	6.3
Almuealimin District	0-1	0.02	1.4	0.032	7.82	2.64	3.01	4.6
	1-2	0.04	1.12	0.026	8	2.15	2.41	4.3
	2-3	0.08	0.32	0.024	8.65	0.7	0.69	4.1
Alshuhada' District	0-1	0.015	0.66	0.088	8.07	1.2	1.42	7
	1-2	0.02	0.53	0.075	8.04	1.85	1.14	8.12
	2-3	0.023	0.33	0.064	8.05	1.53	0.71	8.4

4. Discussion

4.1. Discussion of Physical Soil Properties

Two important markers of the physical behavior of soil that differentiate between dry, partially saturated, and fully saturated situations are moisture content (m.c%) . and degree of saturation (S%). To differentiate between dry, moderately saturated, and fully saturated conditions, moisture content (m.c.%) and degree of saturation (Sr.%) are considered essential indicators of soil behavior. The moisture content (m.c.%) and degree of saturation (Sr.%) in the study area range from 14% to 27% and 50% to 89%, respectively. The high groundwater table and the predominance of clayey soils with limited permeability and high water-retention capacity are responsible for the rise in water content with depth. According to the grain-size study, the sand percentage ranges from 0 to 49.8%, the silt fraction from 12.8 to 41%, and the clay fraction from 30.3 to 68.5%. These ratios verify that the soil is fine-grained. The kind of soil and its water content determine the Atterberg's limits. The plastic limit (P.L) is between 18 and 26%, the liquid limit (L.L) is between 30 and 58%, and the plasticity index (P.I) is between 9 and 36%. The subsurface soils (below the fill layer) range from low-plasticity silty clay (CL) to high-plasticity silty clay (CH) according to the Unified Soil Classification System (USCS) and the plasticity chart. Depending on their inherent moisture level, the consistency index (C.I) ranges from 0.76 to 1.7, suggesting that the soils can be plastic, semi-solid, or solid (Bowles, 1984). According to Poland (1984), the specific gravity (G_s) of typical clayey soils ranges from 2.71 to 2.76. Wet unit weight (γ_w) ranges from 1.66 to 1.96 g/cm³, whereas dry unit weight (γ_d) ranges from 1.36 to 1.66 g/cm³. The specific gravity (G_s) and void ratio (e), which range from 0.65 to 0.99 and exhibit a porosity of 39 to 50%, are strongly related to these characteristics. According to Al-Ashu (1991), a soil is categorized as typically active clay if its clay mineral activity falls between 0.18 and 0.66, showing consistent geotechnical behavior and a small tendency for swelling.

4.2. Discussion of Soil Engineering Test Results

4.2.1. Soil Consolidation of fully saturated

All undisturbed soil samples taken using a Shelby tube were subjected to a one-dimensional consolidation test created especially for fine-grained, undisturbed silty clay soils. The initial void ratio (e_0) data showed variations in soil density and structure with depth, ranging from 0.54 to 1.01. The void ratio is highly dependent on soil type; sandy soils often have higher void ratios and faster consolidation, while clayey soils have lower void ratios and slower consolidation due to their greater compressibility and lower permeability. Pre-consolidation pressure (P_c) values ranged from 88 to 200 kN/m², whereas compression index (C_c) values ranged from 0.106 to 0.354. exhibiting a moderate to high level of compressibility. Clay minerals have higher C_c values, indicating a higher tendency to settle under applied pressures.

On the other hand, higher sand concentrations or lower vacancy ratios result in lower C_c values, indicating lower compressibility. Depending on the kind, quantity, and swelling ability of clay minerals, the swelling index (C_s) varied from 0.006 to 0.091. According to Al-Ashu (1991), the over-consolidation ratio ($OCR = \sigma'_p / \sigma'_{v0}$) ranged from 0.66 to 1.01, suggesting that the soils in the research region range from unconsolidated (UC) to slightly over-consolidated (OC). While the soil elastic modulus (E) varied from 107 to 667 kN/m², the coefficient of compressibility (a_v) ranged from 1.5×10^{-3} to 9.3×10^{-3} kN/m². The range of the volumetric compressibility coefficient (m_v) was 0.8×10^{-3} to 6.1×10^{-3} kN/m². whereas the oedometer elastic modulus (E_o) varied from 164 to 1262 kN/m². Because of differences in the soil's mechanical, physical, and consistency characteristics, these measures show considerable fluctuation with depth. The coefficient of consolidation (C_v) varied from 1.87×10^{-3} to

$12.9 \times 10^{-3} \text{ cm}^2/\text{s}$, whereas the $\sqrt{t_{90}}$ values ranged from 3.3 to 8.7 min. These results are in line with the low consolidation rates typical of cohesive soils with fine grains (Head and Epps, 1982). According to Al-Ashu (1991), the soil's coefficient of permeability (k) varied from 60.9×10^{-8} to $2.57 \times 10^{-8} \text{ cm/s}$, indicating very low permeability.

The computed settlement (S) values ranged from 0.75 to 3.26 cm. It was found that settlement increased with clay content and varied with depth; higher clay content resulted in longer consolidation durations, lower C_v and k values, and ultimately larger settlement magnitudes.

4.2.2. Consolidation of partially saturated soil (50% saturated)

From the results of the tests for consolidation partially saturated soils ($S=50\%$), the values of the consolidation parameters (e_o , p_c , C_c , C_s , a_v , m_v) are higher in fully saturated soils than in partially saturated soils (Fig. 2). The ratio of initial voids (e_o) had the highest value (0.77) at BH1 in depth 1.5-2m and the lowest value (0.26) at BH4 in depth 1.5-2m; the highest value was (C_c) 0.273 at BH5 in depth 2.5-3m and the lowest value (0.06) at BH5 in depth 0.5-1m; and the highest values were (C_s) 0.07 at BH2 in depth (2.5-3m).

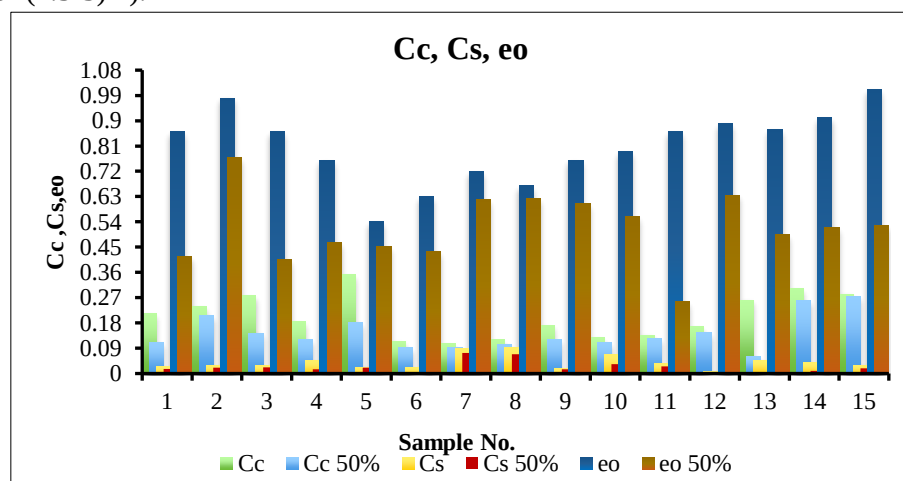


Fig. 2. Initial void ratio(e_o), Compression index (C_c), and swelling index (C_s) Comparison between total and partial saturation results for all soil samples

Pre-consolidation pressure (P_c) at BH3 depth 1.5-2 m had the maximum value (95 kN/m^2), while the lowest value was 62 kN/m^2 at BH1 depth 1.5-2 m (Fig. 3).

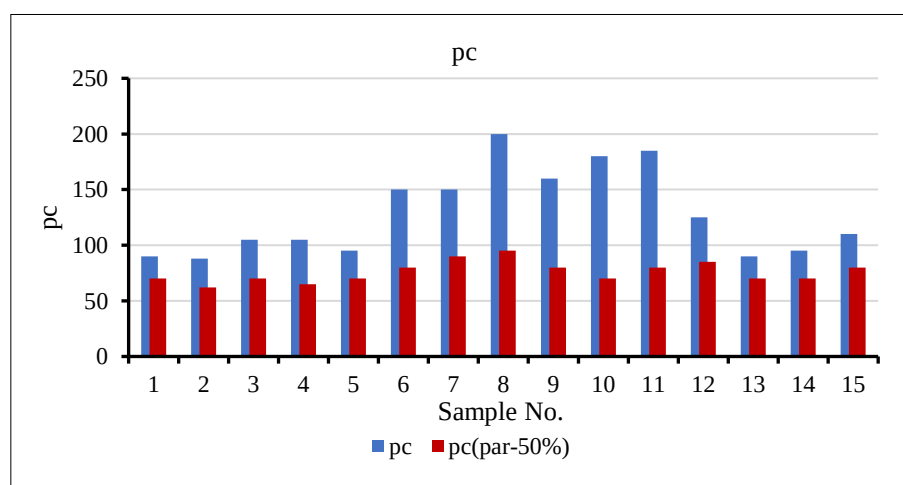


Fig. 3. Pre-consolidation pressure(p_c) Comparison between total and partial saturation results for all soil samples

The coefficients of compressibility (a_v) ranged from 8×10^{-3} kN/m² at BH2 depth 1.5-2m to 1.1×10^{-3} kN/m² at BH5 depth 0.5-1m, while volumetric compressibility (m_v) ranged from 5.51×10^{-3} kN/m² at BH2 depth 1.5-2m to 0.73×10^{-3} kN/m² at BH4 depth 2.5-3m (Fig. 4).

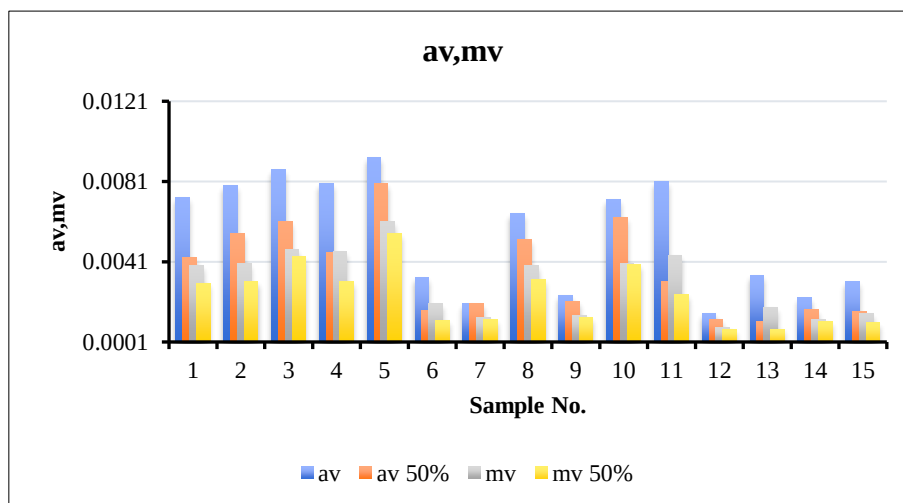


Fig. 4. Coefficients of compressibility (a_v) and Volumetric compressibility (m_v) Comparison between total and partial saturation results for all soil samples

The consolidation parameters (E , E_o , c_v , and k) are higher in partially saturated soils than in fully saturated soils (Fig. 5). Soil elastic modulus (E) ranged from 909 kN/m² at BH5 depth 0.5-1 m to 125 kN/m² at BH2 depth 1.5-2 m, while oedometer elastic modulus (E_o) ranged from 182 kN/m² at BH2 depth 1.5-2 m to 1364 kN/m² at BH4 depth 2.5-3 m.

Comparing the permeability coefficient (k), the highest value was 9.73×10^{-7} cm/sec at BH4 depth 2.5-3m, and the lowest value was 0.72×10^{-7} cm/sec at BH3 depth 2.5-3m, (Fig. 6-a). The greatest coefficient of consolidation (C_v) was 3.6×10^{-3} cm²/sec at BH5 depth 0.5-1m and the lowest value (0.55×10^{-3} cm²/sec) at a depth of 2.5–3 m (BH3) (Fig. 6-b).

In comparison to fully saturated soils, the consolidation test results for partially saturated soils showed a discernible decrease in the values of the initial void ratio (e_0), compression index (C_c), swelling index (C_s), pre-consolidation pressure (P_c), coefficients of compressibility (a_v and m_v), time factor ($\sqrt{t_{90}}$), coefficient of permeability (k), and settlement (S).

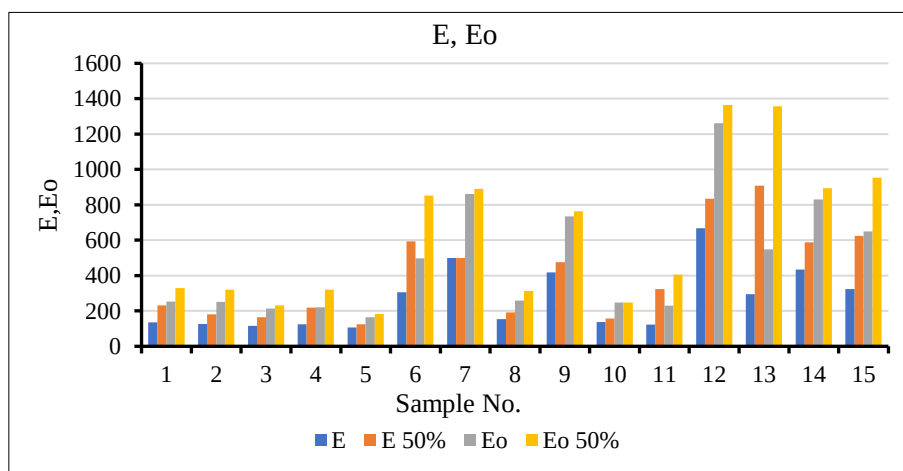


Fig. 5. Soil elastic modulus (E) and Oedometer elastic modulus (E_o) Comparison between total and partial saturation results for all soil samples

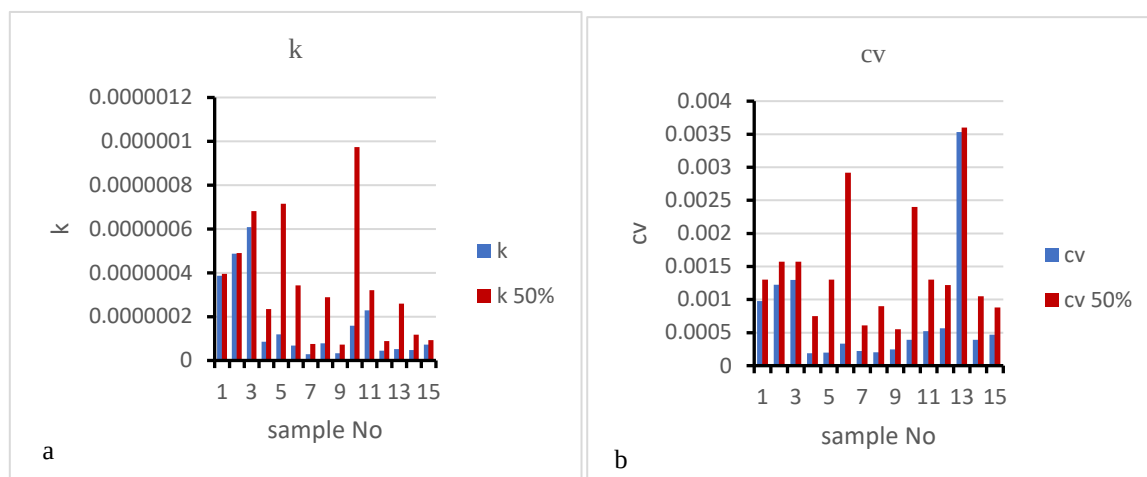


Fig. 6. a) Permeability coefficient (k) Comparison; b) Coefficient of consolidation (Cv) between total and partial saturation results for all soil samples

On the other hand, soil elastic modulus (E), oedometer modulus (E_o), and coefficient of consolidation (Cv) all increased. The mechanical and physical characteristics of the soil, especially its clay content and compressibility, directly affect this behavior. As saturation increases, more water penetrates the pore spaces, reducing interparticle friction and increasing settling values. Conversely, decreased pore-water content at lower saturation levels increases frictional resistance and suction, which lessens settling and limits deformation. The compaction curve, which shows that soils on the wet side of the ideal moisture content are more compressible while those on the dry side are stiffer and more resistant to deformation, is comparable to this pattern of reactivity (Namah, 2014).

4.2.3. The discussion of the results of other engineering tests

Based on permeability coefficient (k) values between 9.7×10^{-6} and 9.9×10^{-4} cm/s, the soils in the research area were classified as clayey, low-permeability (Al-Ashu, 1991). The main reason for differences in permeability throughout the site is variations in the clay mineral composition, namely the swelling-shrinkage behavior and compressibility of the soils. These factors affect the size, shape, and continuity of pore spaces, which causes regional variations in hydraulic conductivity. The findings of the shear strength parameters show that the cohesiveness (c) and angle of internal friction (ϕ) range from 2.7° to 16.4° and 26 to 56.9 kN/m², respectively. The low friction angles indicate that the soil is cohesive and fine-grained, while the relatively high cohesion values indicate the presence of clay minerals that offer substantial shear resistance.

The optimal moisture content (OMC) values vary from 15% to 18%, whereas the maximum dry density (MDD) varies from 1.74 to 1.87 g/cm³, according to the modified proctor compaction test findings. While consolidation is the slow removal of water from wet soils under continuous stress, compaction is the intentional rearrangement of soil particles to improve dry density by pushing air out of spaces. Table 6 illustrates how effective stress rises as the groundwater table declines. The pore-water pressure drops as the water table falls, increasing the effective stresses in the soil layers. The effective stress and total stress are equal since there is no pore pressure in regions above groundwater. This connection emphasizes how groundwater fluctuations have a significant impact on soil strength and settlement behavior, highlighting the necessity of taking seasonal or human changes in groundwater conditions into consideration during design and analysis.

4.2.4. Soil chemical test results discussion

The results of the chemical analysis are important for soil, especially soil under engineering structures, as they affect foundations. The values of pH between 7.82 and 8.65 and the values of SO_3 ranged between 0.31 and 1.88 and the values of CL ranged between (0.024-0.088) and the values of T.S.S ranged between 0.64 and 3.02 and the values of gypsum ranged between 0.67 and 4.04 and it was shown that all chemical analyses are in accordance with the specification (B.S.1377:1990 part 3).

4.3. Evaluation of Relationships between Soil Properties

Several studies have examined the interrelationships among soil geotechnical properties. For instance, Ali (2006) analyzed these relationships using the correlation coefficient (r), which measures the degree of linear association between two variables and quantifies the extent to which a change in one variable is associated with a proportional change in another. Kong (1969) noted binary correlation coefficients are among the most widely applied statistical measures in geotechnical research. The correlation coefficient (r) ranges between -1 and +1. Values approaching +1 indicate a strong direct relationship; values close to -1 indicate an inverse relationship, while values near zero signify the absence of a meaningful linear correlation (Jawad and Mazen, 2011).

The relationship between the liquid limit and the percentage of clay is a direct relationship of positive correlation coefficient values ($R^2=0.4173$ or $r = 0.65$), as shown in Fig. 7. An increase in the liquid limit is noted with an increase in the clay content because the relationships are direct.

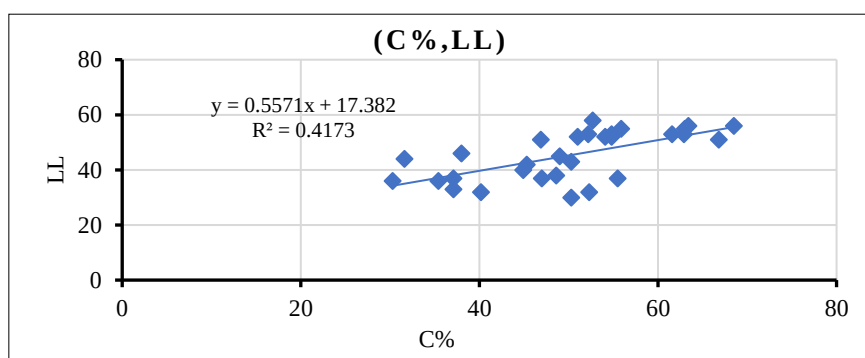


Fig. 7. The relationship between LL and C% in BH1 and BH5

The relationships shown in the curves between LL and C_c indicate that the correlation coefficient ($r=0.32$ for fully saturated, higher than $r=0.29$ for partially saturated) shows a direct relationship shown in Fig. 8. Soil compressibility increases with increasing clay content, and since the relationship between clay content and liquid limit (LL) is direct, compressibility increases with increasing liquid limit (LL).

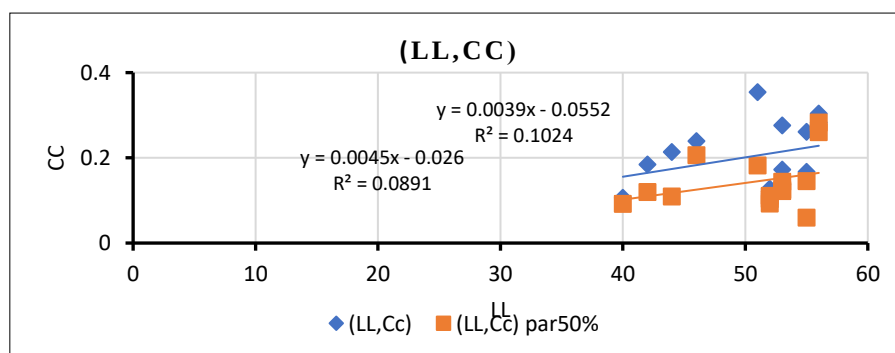


Fig. 8. The relationship between LL and C_c in B.H.1 and B.H.5

The relationships between the liquid limit (LL) and both the consolidation coefficient (cv) and the permeability coefficient (k) showed inverse relationships ($r(cv) = -0.29$ to fully saturated. Less than $r(cv) = -7.7 \times 10^{-3}$ to partially saturated and $r(k) = -0.85$ to fully saturated. Less than $r(k) = -0.57$ to partially saturated, while the relationship between the liquid limit and the settlement (S) was direct ($r = 0.58$ to fully saturated, higher than $r = 0.17$ to partially saturated), as shown in Figs. 9, 10, and 11. This is attributed to the increase in the proportion of clay with a liquid limit, which leads to decreases in the consolidation and permeability coefficients and an increase in the time required for settlement.

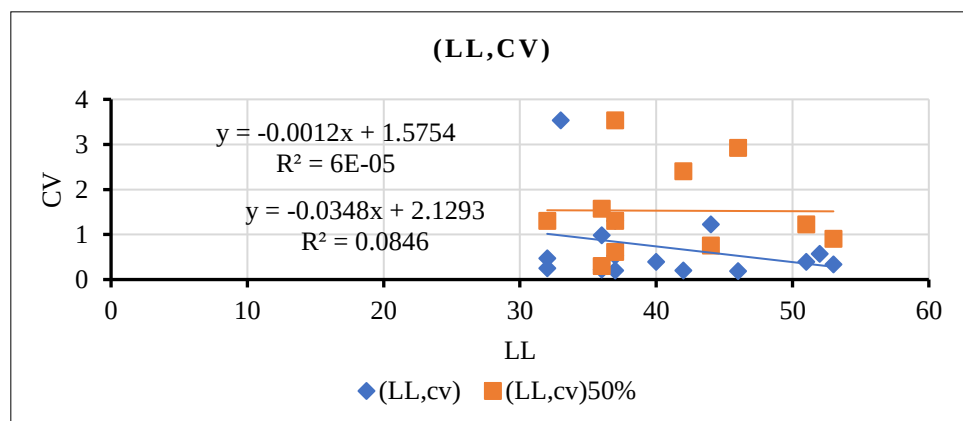


Fig. 9. The relationship between LL and cv in BH1 and BH5

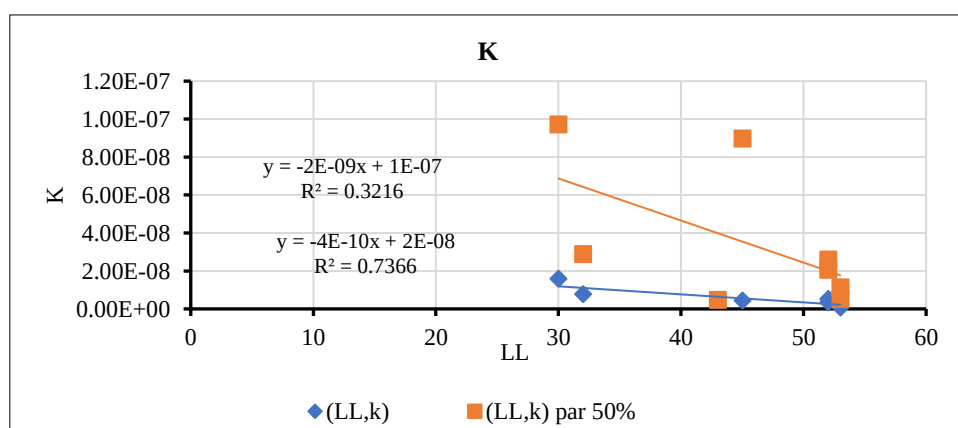


Fig. 10. The relationship between LL and k in BH1 and BH5

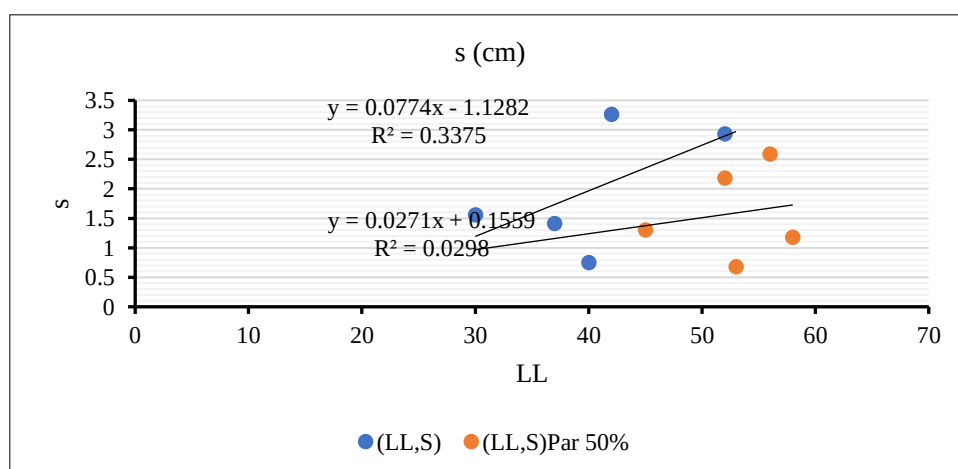


Fig.11. The relationship between LL and S (cm) in BH1 and BH5

5. Conclusions

According to the study, the soils in the region under investigation may be divided into two primary sections: a coarser portion composed mostly of sand, and a soft, fine-grained layer composed mostly of clay and silty clay. The soils are categorized as low-plasticity clay (CL) to high-plasticity clay (CH) according to the Casagrande plasticity chart. The engineering and consolidation test results show that the parameters e_0 , P_c , C_c , C_s , a_v , m_v , and $\sqrt{90}$ is higher in fully saturated soils than in half-saturated (50%) soils. This suggests greater compressibility and deformation potential under full-saturation circumstances. Compressibility (C_c), liquid limit, and clay concentration are all clearly correlated with one another.

However, partially saturated soils have greater C_v and k values due to the combined effects of air and water in the pore spaces, which improve drainage and reduce compressibility. As a result, in partially saturated soils, settlement happens more slowly but to a lesser extent, while in fully saturated clays, the consolidation process takes longer and results in more settlement. These results emphasize how crucial it is to take saturation conditions into consideration when designing engineering structures and foundations, since ignoring changes in soil moisture can result in erroneous settlement estimates and possible structural instability.

Author contributions: CRediT

- Conceptualization: Samhar A. Rida, Raid A. Mahmood, Amer A. AL-Khalidy
- Data curation: Samhar A. Rida, Amer A. AL-Khalidy, Raid A. Mahmood.
- Formal analysis: Samhar A. Rida, Raid A. Mahmood, Amer A. AL-Khalidy.
- Funding acquisition: Samhar A. Rida
- Investigation: Samhar A. Rida
- Methodology: Samhar A. Rida
- Project administration: Amer A. AL-Khalidy, Samhar A. Rida
- Resources: Amer A. AL-Khalidy, Samhar A. Rida
- Software: Amer A. AL-Khalidy, Samhar A. Rida,
- Supervision: Raid A. Mahmood, Amer A. AL-Khalidy
- Validation: Raid A. Mahmood, Amer A. AL-Khalidy
- Visualization: Samhar A. Rida
- Writing – original draft: Samhar A. Rida, Amer A. AL-Khalidy, Raid A. Mahmood.
- Writing – review and editing: Amer A. AL-Khalidy, Raid A. Mahmood

Declaration of Competing Interest

The authors declare that they have no competing financial interests or personal relationships that could have influenced the work reported in this study.

Funding Sources

This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sector.

Data Availability

Data are included in the article or supplementary material. All data generated or analyzed during this study are included in this published article and/or its supplementary materials.

Acknowledgements

The authors thank the professors of the University of Babylon's Department of Applied Geology and the employees of Al-Ma'awal Company for their crucial help and support during this project.

References

- Al-Amar, H.A.S., Hadi, A.H. 2018. Assessment of soil banks of part from Shatt al-Hilla in Al Hilla city/Babylon Governorate. *International Journal of Civil Engineering and Technology*, 9(11), 2306-2315.
- Al-Amar, H.A.S., 2024. Geotechnical engineering and geological study for soil of Hilla District in Babylon Governorate in Iraq. *Institute of Physics Publishing Conference Series: Earth and Environmental Science*, 1374(1), 012007. <https://doi.org/10.1088/1755-1315/1374/1/012007>
- Al-Ashu, M.O., 1991. *Principles of Soil Mechanics*. Dar Al-Kutub for Printing and Publishing, University of Mosul.
- Al-Hiti, B.M., 1985. Groundwater quality within the Baghdad area. MSc. Thesis, University of Baghdad College of Science.
- Ali, W.A., 2006. Some geotechnical properties of the soils of the middle of the Tun Kupri basin and their relationship to groundwater investment. PhD. thesis, College of Science, University of Baghdad.
- Al-Khakani, M.O.K., 2006. Study of Swelling for Clay Soils in Hilla City. MSc. Thesis, University of Baghdad College of Science.
- Al-Khalidi, A.A.L., 2018. Study of engineering properties by using the downhole method for the South Baghdad area in Iraq. *International Journal of Civil Engineering and Technology*, 9, 508–520.
- Al-Khalidy, A.A., 2025. Determining Soil Properties using Vertical Electrical Sounding: The Case Study of the Power Electrical Station Project in Al-Debony, Iraq. *Engineering, Technology and Applied Science Research* 15(3):23514-23521. <https://doi.org/10.48084/etasr.10751>.
- Al-Rubaiee, A.K.H., Al-Owaidi, M.R., 2024. Initial assessment of soil liquefaction potential for selected sites in Hilla city, Babylon Governorate, Iraq. *Quarterly Journal of Engineering Geology and Hydrogeology*, 57(1), qjegh2023-076. <https://doi.org/10.1144/qjegh2023-076>
- Al-Zubaydi, J.H.A., Al-Kaily, A.A.L., Al-Rubaey, M.Q.K., 2016. Engineering properties of soft clay stabilized with lime materials, emulsified asphalt, and bentonite sodium for the subgrade of the road under construction in Hilla City, Babylon Governorate. *International Journal of Civil Engineering and Technology*, 7(5), 347–367.
- ASTM (American Society for Testing and Materials), 1985. Classification of soils for engineering purposes: annual book of ASTM standards, D 2487-83, 04.08. In American Society for Testing and Materials. 395-408.
- Barden, L., Sides, G.R., 1970. Engineering behavior and structure of compacted clay. *Journal of the Soil Mechanics and Foundations Division*, 96(4), 1171-1200. <https://doi.org/10.1061/JSFEAQ.0001434>
- Bowles, J.E., 1984. *Foundation Analysis and Design* (4th Ed.). McGraw-Hill, USA.
- Buday, T., Jassim, S.Z. 1984. Geological Map of Iraq, 1:1,000.00 scale series. Sheet No. 1, Tectonic Map, Publication of Geosurv, Baghdad.
- Das, B.M., 2006, *Principles of Geotechnical Engineering* (5th Ed.). Brooks Cole/Thompson Learning, USA.
- Escario, V.G., Juca, A.C., 1989. Strength and deformation of partly saturated soils. In *Proc. 12th Int. Conf. on SMFE*, 1, 43-46.
- Fredlund, D.G., Rahardjo, H., 1986. Unsaturated soil consolidation theory and laboratory experimental data (154-154). ASTM International.
- Fredlund, D.G., Morgenstern, N.R., 1977. Stress state variables for unsaturated soils. *Journal of the geotechnical engineering division*, 103(5), 447-466. <https://doi.org/10.1061/AJGEB6.0000423>
- Head, K.H., Epps, R.J., 1982. *Manual of Soil Laboratory Testing Volume 2: Permeability. Shear Strength and Compressibility Tests*. Pentech Pres, London.
- Jawad, A.S, Mazen, H.J., 2011. Source of Correlation Coefficient. Scientific Research, Baghdad, Dar Al-Zia Printing and Design.
- Kadhim N.K., Qassim A.A., Amer J.K., 2025. Applications of electrical resistivity of soil in design of electrical power station: Case study in Diwaniyah. *Institute of Physics Publishing Conf. Series: Earth and*

- Environmental Science 1545 ,012006, Institute of Physics Publishing Publishing, <https://doi.org/10.1088/1755-1315/1545/1/012006>.
- Karkush M., Ali, N.A., 2023. Electrokinetic enhancement of microbial induced calcite precipitation used in improving the shear strength of soft clay. In Smart Geotechnics for Smart Societies, (155-163). CRC Press. <https://doi.org/10.1201/9781003299127-14>
- Kong, C., 1969. Quantitative Methods for Business and Economic Analysis, International Text Books company, Pennsylvania.
- Namah, M.Y., 2014. Prediction of consolidation settlement of unsaturated soil from laboratory tests. M.Sc. Thesis, College of Science, University of Baghdad.
- Poland, W.S., 1984. On the analyst's neutrality. Journal of the American Psychoanalytic Association, 32(2), 283-299. <https://doi.org/10.1177/000306518403200203>
- Rahardjo, H., Fredlund, D.G., 1995. Experimental verification of the theory of consolidation for unsaturated soils. Canadian Geotechnical Journal, 32(5), 749-766. <https://doi.org/10.1139/t95-074>
- Rahardjo, H., Fredlund, D.G., 1996. Consolidation apparatus for testing unsaturated soils. Geotechnical testing journal, 19(4), 341-353. <https://doi.org/10.1520/GTJ10712J>
- Saba J.M.K., 2023. Shear strength and chemical properties of soft clayey soil treated with magnetized water. Magazine of Civil Engineering, 124(8), 12406. <https://doi.org/10.34910/MCE.124.6>
- Salih, M.M., Jawad, F.W., Al-Ameri, A.F.I., Abdulhameed, A.A., 2022. Geotechnical correlations of soil properties in Hilla City–Iraq. Open Engineering, 12(1), 729-742. <https://doi.org/10.1515/eng-2022-0335>
- Shakir, A.M., Foti, S., Garofalo, F., Hijab, B.R., 2013, Laterally constrained inversion of surface wave data at Najaf City (Iraq). Soil Dynamics and Earthquake Engineering, 45, 89–95. <https://doi.org/10.1016/j.soildyn.2012.11.003>
- Smith, G.N., Smith, I.G.N., 1998. Elements of soil mechanics. 7th ed. Blackwell Science Inc., Oxford, UK.
- Standard, B., 1990. 1377, Methods of Test for Soils for Civil Engineering Purposes. British Standard Institution, London.
- Terzaghi, K., 1943. Liner-plate tunnels on the Chicago (Il) subway. Transactions of the American Society of Civil Engineers, 108(1), 970-1007. <https://doi.org/10.1061/TACEAT.0005664>