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Effect of degree of saturation on consolidation parameters of fine-grained soils

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Research Article

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ABSTRACT

Terzaghi (1943)'s theory of consolidation phenomenon has some limitations for unsaturated geological environments in nature. Additionally, soils are subjected to consolidation tests under partially saturation conditions. The scope of this study is to illustrate the effect of degree of saturation (S_r) on the consolidation parameters using remolded soil samples. For this purpose, swelling pressure, compression index (c_c), re-compression index (c_r), coefficient of consolidation (c_v), and pre-consolidation stress (σ_p') were determined for the samples having different degrees of saturation and plasticity (PI). The consolidation parameters obtained from "undersaturated" remolded samples with varying degrees of saturation were compared with the consolidation parameters obtained from "fully saturated" remolded samples to illustrate which parameters of consolidation are affected most by the degree of saturation. The effect of swelling of partially saturated samples was eliminated prior to conducting oedometer tests. Results indicated a meaningful relationship between the compression index and the degree of saturation, though the relationship with the recompression index was less certain. Pre-consolidation stress generally decreased as saturation increased, while no definitive trend was observed between the degree of saturation and the consolidation coefficient. Overall, most consolidation parameters were found to be somewhat affected by the degree of saturation.

1. Introduction

Terzaghi (1943)'s one dimensional consolidation theory the following assumptions are made: the soil is homogeneous and fully saturated, soil particles and pore water are incompressible, and water flow during pore water dissipation is only in the vertical direction, the consolidation is one-dimensional (1D). Additionally, the change in soil volume due to consolidation is negligible compared to the initial soil volume, flow is laminar and follows Darcy's law, and permeability remains uniform throughout the compressible soil layer. The coefficient of volume compressibility remains constant over time, and compression occurs solely from pore water expulsion, so initial and secondary consolidation are neglected. Finally, the coefficient of consolidation for a specific soil and site conditions is assumed to be constant.

Terzaghi (1943)'s consolidation theory postulates that soil specimen subjected to oedometer tests are fully saturated, a postulation is valid only for samples collected from the capillary fringe or below the water table (ASTM 2435/D2435M-11, 2020). Thereby, the theory of consolidation has some limitations for unsaturated geological environments in nature, which are sited most of engineering projects and/

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or foundations above fully saturated zones where the soil is partially saturated. Additionally, soil samples taken from boreholes will be unsaturated due to volume expansion as overburden pressure is removed. And in practice, test specimens are directly subjected to consolidation test regardless of the zone where the soil sample collected from. That means no saturation procedure is applied prior to consolidation test. Although the soil specimen is inundated at the beginning of the test, this procedure is only for preventing of the specimen against drying during the test (ASTM 2435/D2435M-11, 2020). In other words, inundation of the test specimen prior to the consolidation test does not promote saturation of soil specimen at all. Eventually, it turns out that all soils are subjected to consolidation tests under partially saturation conditions.

No earlier study was found investigating the effect of saturation degree on consolidation parameters for remolded soils by performing conventional consolidation experiments. Some researchers investigated the effects of soil index properties on certain consolidation-related parameters such as the pre-consolidation pressure, compression index. Some previous studies investigating consolidation properties in natural and/or remolded soils are summarized below. Several studies have been carried out to date on the investigation of the consolidation properties of soils., recompression index and swelling pressure by employing methods such as fuzzy logic, finite elements, certain empirical relationships as well as computer simulations (e.g., Dipova and Cangir, 2010; Naser, 2013; Kayabalı and Yaldız, 2014; Sünnetçi and Ersoy, 2016; Yurtçu and Özocak, 2016). These researchers revealed in general that the index properties of soils and the consolidation parameters can be calculated. Kayabalı and Özdemir (2010), Asadi (2017) and Balcı et al. (2018) tried to find the consolidation parameters in natural and remolded soils by using centrifugal consolidation technique, which can reduce the duration of experimental studies that take more than a week in the traditional consolidation method to a few hours. For this purpose, the consolidation parameters obtained from the centrifugal consolidation method and the consolidation parameters obtained from the one-dimensional consolidation experiments were compared. Gibson et al. (1967), Yong et al. (1983) and Fox and Berles (1997) explain high strains in soils. Biot (1941) developed Terzaghi (1925)'s one-dimensional consolidation theory in three dimensions and used this theory (Biot, 1956) in solving dynamic problems. Working on one-dimensional consolidation theory, Davis and Raymond (1965), Xie et al. (2002), Geng et al. (2006) and Cai et al. (2007) argued that as a result of loads applied on the sample during consolidation test, the compressibility and permeability decrease with the shrinking of the pores in the soil. Hansbo (1960), Davis and Raymond (1965) and Geng (2008) developed nonlinear solutions of consolidation problems by using compression and permeability coefficients that decrease as a result of loading. Working on remolded clays prepared by considering the degree of saturation, Barden (1965) explained the stress state on unsaturated soils using Bishop (1959)'s relationship. Fredlund (1977), Fredlund and Morgenstern (1976) developed mathematical relations for the solution of consolidation problems in unsaturated soils. Casagrande and Fadum (1940) and Taylor (1942) developed methods for finding the consolidation coefficient, and Cortellazzo (2002), Muntohar (2009) compared the methods developed by different researchers to determine the consolidation coefficient. Boone (2010), who worked on the calculation of the pre-consolidation stress, also presented the work of many researchers in summary form. Phanikumar and Amrutha (2014) aim that the roles of overburden pressure and the degree of saturation in changing the void ratio and compressibility of clays. For this purpose, conducted a series of consolidation tests on soil samples with a wide range of saturation degrees (25, 50, 75 and 100%) to ensure pre-excavation, post-excavation and post-construction conditions in a laboratory environment. However, the main purpose of their study is not to compare the effect of saturated and unsaturated soils on consolidation parameters. To realize the pre-excavation and post-excavation conditions in the laboratory environment, they applied and removed 25, 50 and 100 kPa stress to the samples with different degrees of saturation and applied a stress up to 400 kPa to achieve the post-construction condition.

2. Materials

A total of 8 soil samples with different plasticities were used in this investigation. Samples were collected from the vicinity of Ankara, Türkiye. They were dried and sieved through the #40 sieve prior to the compaction and consolidation tests. Phase relationships of solid-water-air system (e.g., Holtz and Kovacs, 1981) were used to prepare identical samples. For this, the initial void ratios and dry unit weights of the samples were kept constant. The index and physical properties of the samples were determined, and the samples were compacted using static methods. The permeability-consolidation cell (Figure 1) was used to saturate the partially saturated samples and to perform consolidation experiments on these samples. Swelling pressure value of partially saturated samples was measured in the setup developed by Kayabalı and Yaldız (2014) (Figure 2).

Figure 1- Permeability-consolidation test apparatus and its accessories (from Asadi, 2017).

Figure 2- Swelling pressure test apparatus (from Kayabalı and Yaldız, 2014).

3. Methods

To determine the physical and index properties of samples, sieve analysis, hydrometer test, Atterberg limits tests and pycnometer test were performed, and soil classes were determined according to Unified Soil Classification System (USCS). For the plastic limit and liquid limit tests, the part of the samples that passed #40 sieve was used. Plastic limit tests were carried out using the hand-rolling method according to ASTM D 4318-05 (American Society of Testing Materials, 2005) with 5 replicates of each sample and 50 g of the sample amount in the replicate. Liquid limit experiments were performed using the falling cone method according to BSI 1377 (British Standards Institution, 1990) at 5 points corresponding to the penetration amounts between 10-30 mm for each sample. The solid particle densities of the samples were calculated by applying the pycnometer test according to ASTM D 854-06 (American Society of Testing Materials, 2006*a*), using 25 g dry sample in 50 ml pycnometers.

The nature of the investigation requires preparation of soil samples with certain physical properties. To investigate the effect of various degrees of saturation on specific consolidation parameters the void ratio, the dry mass of soil sample and the total volume of the compacted soil sample need to be kept constant. The only variable is the amount of water for the preparation of test specimens. From each soil sample, 4 identical specimens were prepared which are homogeneously mixed at 70, 80 and 90 percent degrees of saturation. During the sample preparation stage, 2 samples at 70% saturation and the other 2 samples at 80% and 90% degrees of saturation, respectively, were prepared using the static compaction method. One of the samples compacted at 70% degree of saturation was used to obtain a fully saturated sample in the permeability-consolidation setup.

Initial trial tests showed that respectively at void ratios of 0.7 and 0.9 to CL and CH types of soils are suitable to prepare the consolidation test specimens to result in total wet densities recorded in most fine-grained soils. All necessary computations were made to find the proper amounts of dry soil mass and water for the initial degree of saturation of first 0.7 to fill a consolidation ring with the diameter of 50 mm and a height of 20 mm. The soil and water were mixed and poured into a hollow cylinder with 50 mm diameter. Compressive load was incrementally applied up to 1 kN onto the wet mixture using a piston and load frame. After this, the specimen was turned upside down and the same loading procedure was applied from the reverse side and the resulting specimen height was measured. This procedure was repeated until the specimen height is reduced to 20 mm. At the end, a test specimen with diameter of 50 mm, a height of 20 mm, a void ratio of 0.7 and a degree of saturation of 0.7 was obtained. The statically compacted soil specimen was then transferred to a consolidation ring. The same specimen preparation procedure was adopted to obtain specimens with degrees of saturation of 0.8 and 0.9 keeping the void ratio of 0.7, the amount of dry soil mass and the total volume constant. All procedures outlined above both for CL and CH type of soils were repeated for using the void ratio of 0.7.

It is well known that a compaction procedure never yields a final product with the "zero air void". Therefore, to prepare a fully saturated specimen (S_r=1.0) a different scheme was employed. For this, the spare specimen with the degree of saturation of 0.7 was prepared and placed into a specific cell shown in Figure 1. This is a two-stage procedure to obtain a fully saturated test specimen. The specimen in this cell first is allowed to saturate under a hydraulic head of about 50. After the water appears at the bottom outlet the specimen is left under hydraulic pressure for an additional period so that at least one pore-volume of water passes through the specimen. During the saturation stage, a load equaling to swell force determined from the procedure outlined below is kept on top of the test specimen to prevent the swelling of specimen during saturation. The second stage involves the routine one-dimensional consolidation procedure.

Swell pressure of soils is known to be controlled by dry density and water content. Therefore, artificially prepared soil specimens are expected to exhibit some degree of expansion when subjected to inundation prior to consolidation test. To obtain reliable data free of swelling effect from the consolidation tests, certain precautions need to be taken to eliminate such effects. For this, a methodology was adopted in which the swell pressure of the compacted soil sample under certain void ratio and degree of saturation was measured on another identical sample. The consolidation specimen was placed in a consolidation cell. The loading piston attached to a load cell was lowered gradually on top of the specimen (Kayabalı and Yaldız, 2014, Figure 2). An initial seating load of 10N was applied to ensure that there is no gap between the top of the loading piston and the top cap laying on top of the specimen. The specimen was inundated and left so for a period of 24 hours. At the end of 1 day the total swell force was read from the digital display from which the initial seating load was deducted and then the remaining value was divided by the surface area of the test specimen to obtain the swell pressure.

Consolidation tests was carried out starting by eliminating the swell pressure recorded earlier (American Society of Testing Materials, 2006*b*). For this, the specimen was loaded to the measured swell force and then it was inundated so that the additional height coming from the swelling process is cancelled out. Vertical loads applied such that the resulting effective stresses are 25, 50, 100, 200, and 400 kPa. Unloading of the specimens was rendered as the loading sequence was reversed; namely from 400 kPa to 200 kPa, from 200 kPa to 100 kPa and so on. Vertical displacements were recorded immediately after placing the consolidation load. The readings were taken at times of 5 sec., 10 sec., 30 sec., 60 sec., 2 min., 4 min., 10 min., 30 min., 1 hr., 2 hrs., 4 hrs., 8 hrs., and 24 hrs. Vertical displacement versus square root of times (in minutes) curve were constructed to determine t_{90} values of each stage of consolidation test per specimen. t_{90} values were used in computing the coefficient of consolidation, c_v.

4. Experimental Results

4.1. Physical and Index Properties of Soil Samples

The physical and index properties of 8 clay samples were determined initially. After that, swelling pressures, compression indices, re-compression indices, pre-consolidation stresses and consolidation coefficients for 32 homogeneous specimens (8 samples multiplied by 4 different levels of saturation) obtained from these samples were determined through the swelling and consolidation tests. Particle size distribution, liquid limit, plastic limit, solid particle density and soil classes according to the USCS for the investigated soil samples are given in Table 1 and Figure 3.

The liquid limit of the samples varies between 24.4 and 98%, the plastic limits are between 14.8 and 31.4% and thus the plasticity indices range from 9.6 to 66.6%. N1, N2 and N3 samples were classified as low plasticity clay (CL) and N4, N5, N6, N7 and N8 samples as high plasticity clay (CH). The specific gravity of the samples ranges from 2.68 to 2.78. As a result of sieve and hydrometer analyzes the percentages of sand, silt and clay of the samples vary between 16-80%, 18-64% and 3-37%, respectively.

Table 1- Index properties of soil samples employed in the investigation.

Sample No.	N1	N2	N3	N4	N5	N6	N7	N8
LL (%)	24.4	35.7	43.1	57.0	70.5	77	84.5	98
PL (%)	14.8	18.9	22.9	24.3	25.9	21.5	28.3	31.4
PI (%)	9.6	16.8	20.2	32.7	44.6	55.5	56.2	66.6
Soil class (USGS)	CL	CL	CL	CH	CH	CH	CH	CH
Specific Gravity (G_s)	2.70	2.74	2.76	2.76	2.75	2.68	2.74	2.78
Sand (%)	70	23	23	20	19	51	18	16
Silt (%)	18	64	63	60	56	30	57	47
Clay (%)	3	13	14	20	25	19	25	37

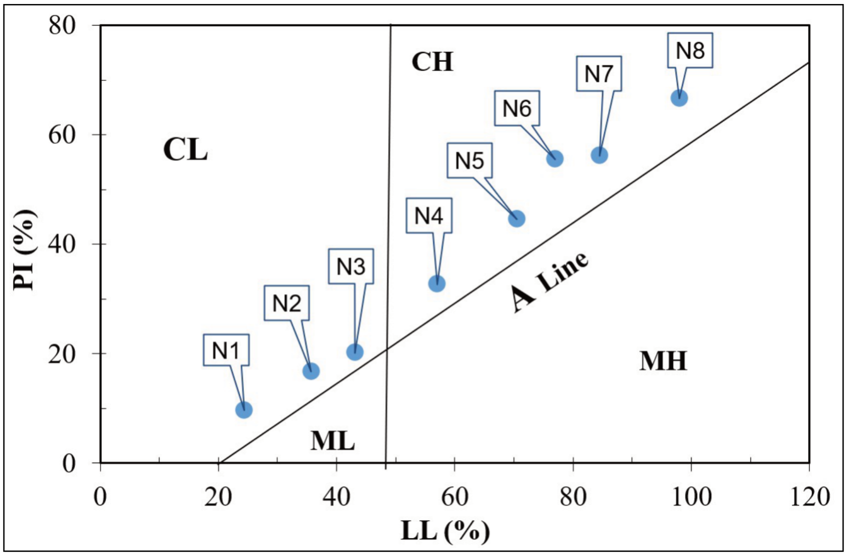

Figure 3- Distribution of soil samples according to Unified Soil Classification System (USCS).

4.2. Swelling Pressure

Swell tests were performed on partially saturated remolded soil specimens prior to the consolidation tests. The amount of swelling measured on those samples was used as the pre-load for the samples subjected to oedometer tests prior to inundation. Swelling pressures measured for all samples at different degrees of saturation are presented in Table 2.

The measured swelling pressures varied between 3 and 100 kPa. For the partially saturated samples, the relationships between swelling pressure and degree of saturation and plastic limit were also evaluated. It was observed that the swelling pressure values decreased with increasing saturation degrees. This result is due to the high-water content of soils with high saturation degree and the low tendency of these soils to absorb water (Villar and Lloret, 2008, Elsharief et al., 2014, Keskin et al., 2023).

It was observed that the swelling pressure values increased with increasing plasticity. In addition, swelling behavior was not observed in the low plasticity samples (N1 and N2), while a small amount of swelling occurred in the low plasticity N3 sample.

Table 2- Swelling pressures for all soil samples (kPa).

No.	Degrees of Saturation, S_r			
	70%	80%	90%	100%
N1	-	-	-	-
N2	-	-	-	-
N3	10	3	-	-
N4	10	8	3	-
N5	25	20	10	-
N6	28	25	18	-
N7	65	35	30	-
N8	100	80	65	-

4.3. Void Ratio - Effective Stress Relationships

As a result of the consolidation experiments, the void ratio-effective stress graphs were plotted, and the consolidation parameters of all the samples were determined using the Casagrande method (the details of this method may be found in Holtz and Kovacs, 1981). To compare the changes in consolidation parameters like that compression stress, re-compression stress and pre-consolidation stress, the void ratio-effective stress plots which show 4 different degrees of saturation (70%, 80%, 90%, and 100%) for each sample are presented in Figure 4. Detailed evaluations of these parameters are presented in the following sections.

4.4. Compression Index, c_c

Compression index values vary between 0.15-0.39 (Table 3). The degree of saturation versus the

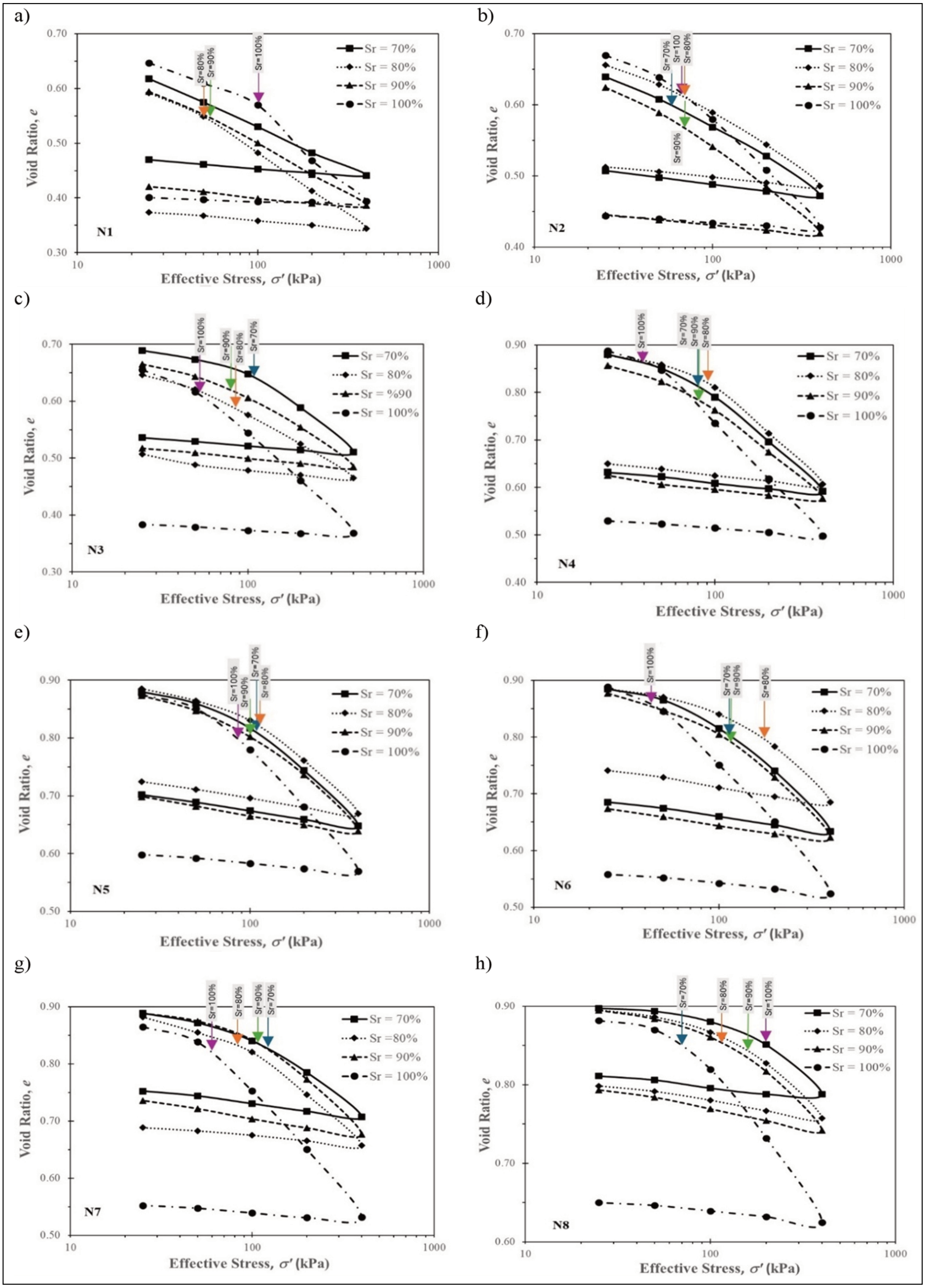

Figure 4- Oedometer test results for 8 soil samples conducted at 4 different saturation ratios for a) Sample N1, b) Sample N2, c) Sample N3, d) Sample N4, e) Sample N5, f) Sample N6, g) Sample N7 and h) Sample N8.

Table 3- The list of compression indices (c_c), recompression indices (c_r) and preconsolidation pressures (σ_p') obtained from oedometer tests conducted on specimens of varying degrees of saturation (S_r).

No.	LL (%)	S_r	c_c	c_r	σ'_p (kPa)
N1	24	0.7	0.15	0.023	-
N1	24	0.8	0.23	0.024	50
N1	24	0.9	0.19	0.028	55
N1	24	1.0	0.29	0.006	100
N2	36	0.7	0.16	0.029	60
N2	36	0.8	0.17	0.022	70
N2	36	0.9	0.20	0.021	68
N2	36	1.0	0.25	0.013	70
N3	43	0.7	0.23	0.021	110
N3	43	0.8	0.18	0.035	85
N3	43	0.9	0.20	0.027	80
N3	43	1.0	0.29	0.012	56
N4	57	0.7	0.33	0.033	81
N4	57	0.8	0.34	0.036	90
N4	57	0.9	0.31	0.040	82
N4	57	1.0	0.39	0.026	44
N5	71	0.7	0.28	0.045	110
N5	71	0.8	0.27	0.046	115
N5	71	0.9	0.27	0.049	100
N5	71	1.0	0.35	0.024	82
N6	77	0.7	0.30	0.043	122
N6	77	0.8	0.26	0.047	180
N6	77	0.9	0.30	0.042	120
N6	77	1.0	0.38	0.028	57
N7	85	0.7	0.22	0.038	130
N7	85	0.8	0.27	0.026	82
N7	85	0.9	0.27	0.049	120
N7	85	1.0	0.37	0.017	58
N8	98	0.7	0.15	0.019	200
N8	98	0.8	0.18	0.034	170
N8	98	0.9	0.20	0.043	140
N8	98	1.0	0.32	0.021	98

compression index graphs of all the samples are presented in Figure 5.

It was determined that there were positive correlations between increasing degrees of saturation and compression index, with coefficients of determination ranging from R^2=0.32 to R^2=0.94. While evaluating the compression index versus degree of saturation (S_r - c_c) relationships of the samples, plasticity was also considered. It was observed that the coefficients of determination of the relationships between S_r and c_c were different in each sample. The reason for this difference is due to the different PI values of the samples. Since the density values are varying in a very narrow range, the relation between the cc and the density has not been evaluated. Azzouz et al. (1976), Herrero (1983) and Kalantary and Kordnaeij (2012) revealed that LL and e_0 are the most effective parameters on c_c. Wroth and Wood (1978) and Mesri et al. (1975) revealed that the compressibility and compression index of clays are mainly affected by type of clay minerals, clay content, plastic limit and liquid limit. When the S_r - c_c relationships are evaluated by considering the PI the determination coefficients of c_c – S_r relationships for N1 and N2 samples with PI values of 9.6% and 16.8% are quite high, and there are positive linear relationships between c_c and S_r. However, it is seen that the c_c - S_r relationship for the sample N3 with a PI value of 20.2% is still positive, but the coefficient of determination is quite low (R^2 = 0.32). Thus, although the PI for these three samples increased, the correlation coefficients of the S_r - c_c relationships first increased and then decreased. When the S_r - c_c relationships of other five samples with higher PI values were examined, it was observed that the determination coefficients of the S_r - c_c relationships increased from 0.34 to 0.81, as the PI values increased from 32.7% to 66.6%. It was thought that the soil group (CL and CH) should also be considered while evaluating the samples with these different PI values. Accordingly, while there was inconsistency in the S_r - c_c relationships for the first three samples in the CL soil group, the significance of the relations increased as the PI increased for the five samples in the other CH soil group. In addition to considering the PI effect when examining the S_r - c_c relationships, differences between the partially saturated samples (S_r = 70%, 80% and 90%) and the fully saturated samples were also observed. When the graphs in Figure 5 are examined, it is observed that the c_c values for all PI values also increased as the saturation increased until it reached 90%. However, c_c increased quite significantly in 100% saturated samples. These results clearly show that the c_c values of fully saturated and partially saturated samples can be quite different from each other.

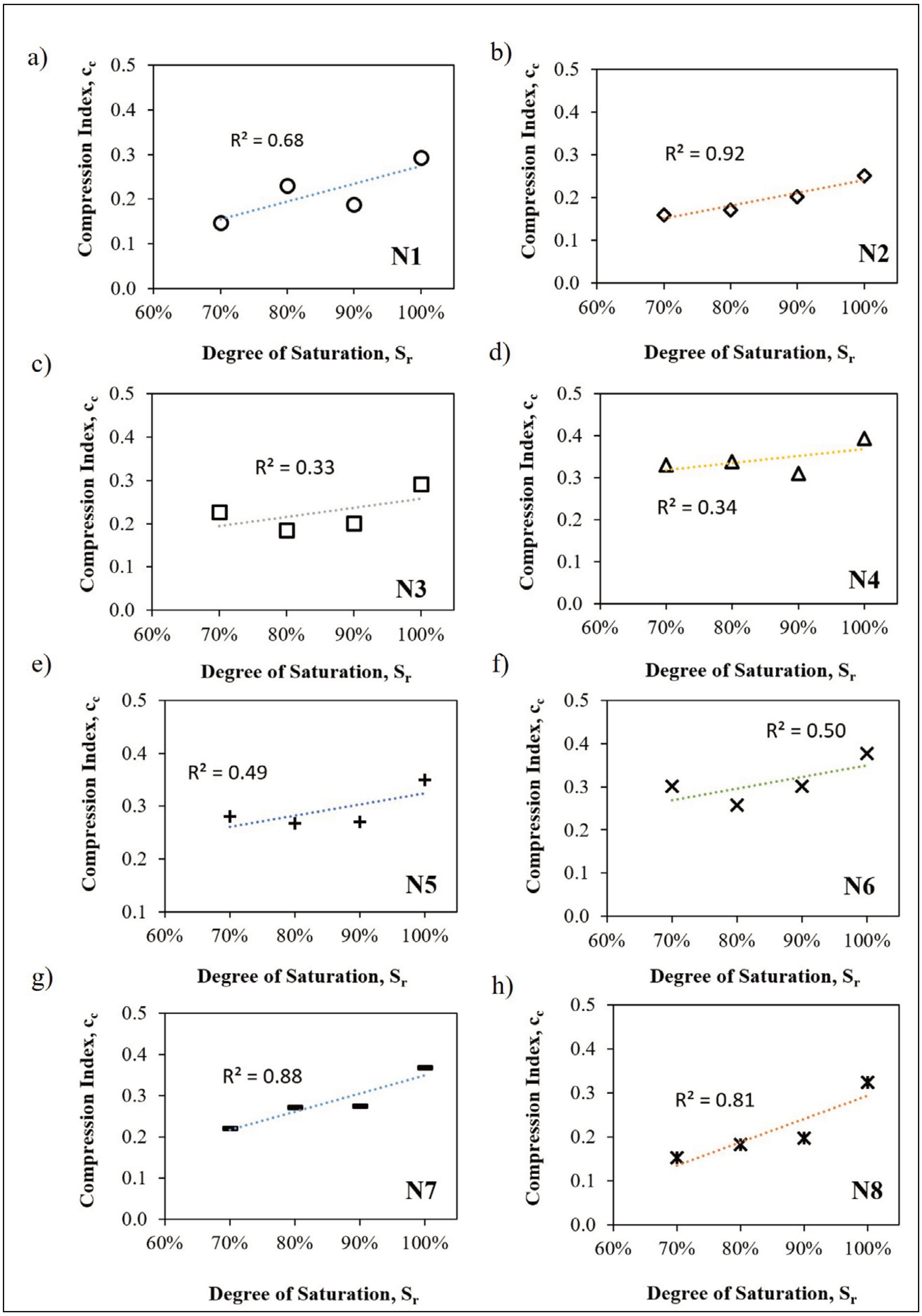

Figure 5- Graphs illustrating the relationship between the compression index and the degree of saturation for a) Sample N1, b) Sample N2, c) Sample N3, d) Sample N4, e) Sample N5, f) Sample N6, g) Sample N7 and h) Sample N8.

4.5. Re-compression Index, c_r

Recompression index values vary between 0.01 and 0.05 (Table 3). The saturation degree-recompression index graphs of all samples are presented in Figure 6.

It can be clearly seen from the graphs in Figure 6 that fully saturated (S_r = 100%) and partially saturated (S_r = 70%, 80% and 90%) samples significantly affect the c_r values in the recompression index-degree of saturation relationships (c_r - S_r). Accordingly, when

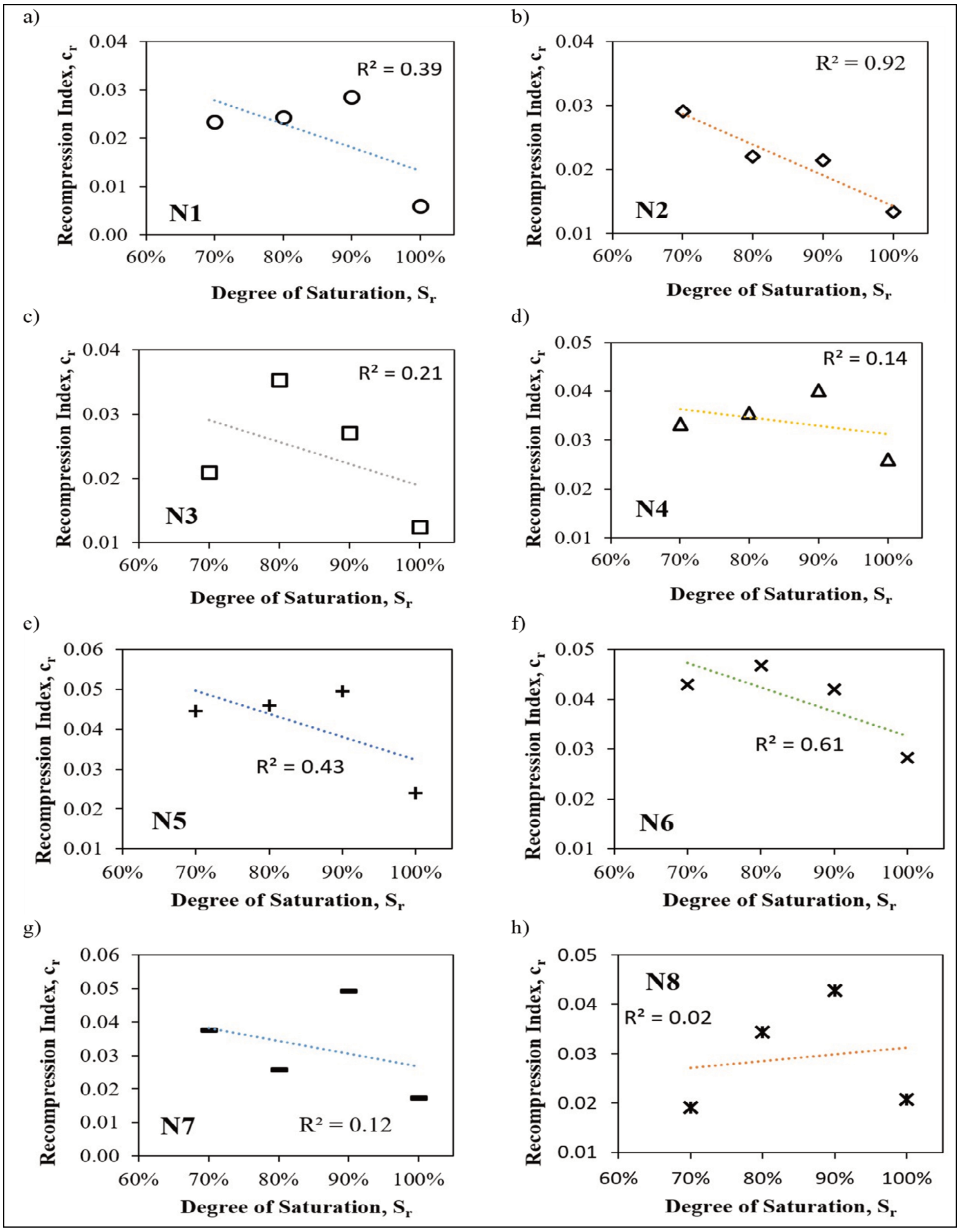

Figure 6- Graphs illustrate the relationship between the recompression index and the degree of saturation for a) Sample N1, b) Sample N2, c) Sample N3, d) Sample N4, e) Sample N5, f) Sample N6, g) Sample N7 and h) Sample N8.

the relationships of N1, N3, N4, N5 and N8 samples were examined, it was observed that the c_r values of the fully saturated sample were inconsistent with the c_r values of partially saturated samples. It was determined that the c_r value of the fully saturated sample decreased abruptly in comparison with the c_r value of the partially saturated samples in almost all cases. Thus, it has been understood that the c_r value of the saturated sample deteriorates the general trend of the c_r - S_r relationships and affects the R^2 values by significantly decreasing them. When c_r - S_r relations were evaluated according to increasing PI values and soil group, it was determined that these two parameters did not have a significant effect on the relationships.

4.6. Pre-consolidation Stress, σ_p'

The pre-consolidation stress values of the samples vary between 44 and 200 kPa. The saturation degree

- pre-consolidation stress graphs of all samples are presented in Figure 7. Since the graph of e-log-σ_p' at 70% saturation was linear in the N1 sample, the pre-consolidation stress could not be calculated. It was observed that the pre-consolidation stress of the saturated samples, especially those with high plasticity, was lower than the values of the partially saturated samples.

Except for samples N1, N2 and N6, the increasing degree of saturation causes the pre-consolidation stress to decrease. While the pre-consolidation stress values of partially saturated high plasticity samples (N5, N7 and N8) were very close to each other, a significant decrease was observed in the pre-consolidation stress of the fully saturated high plasticity samples (Figure 7).

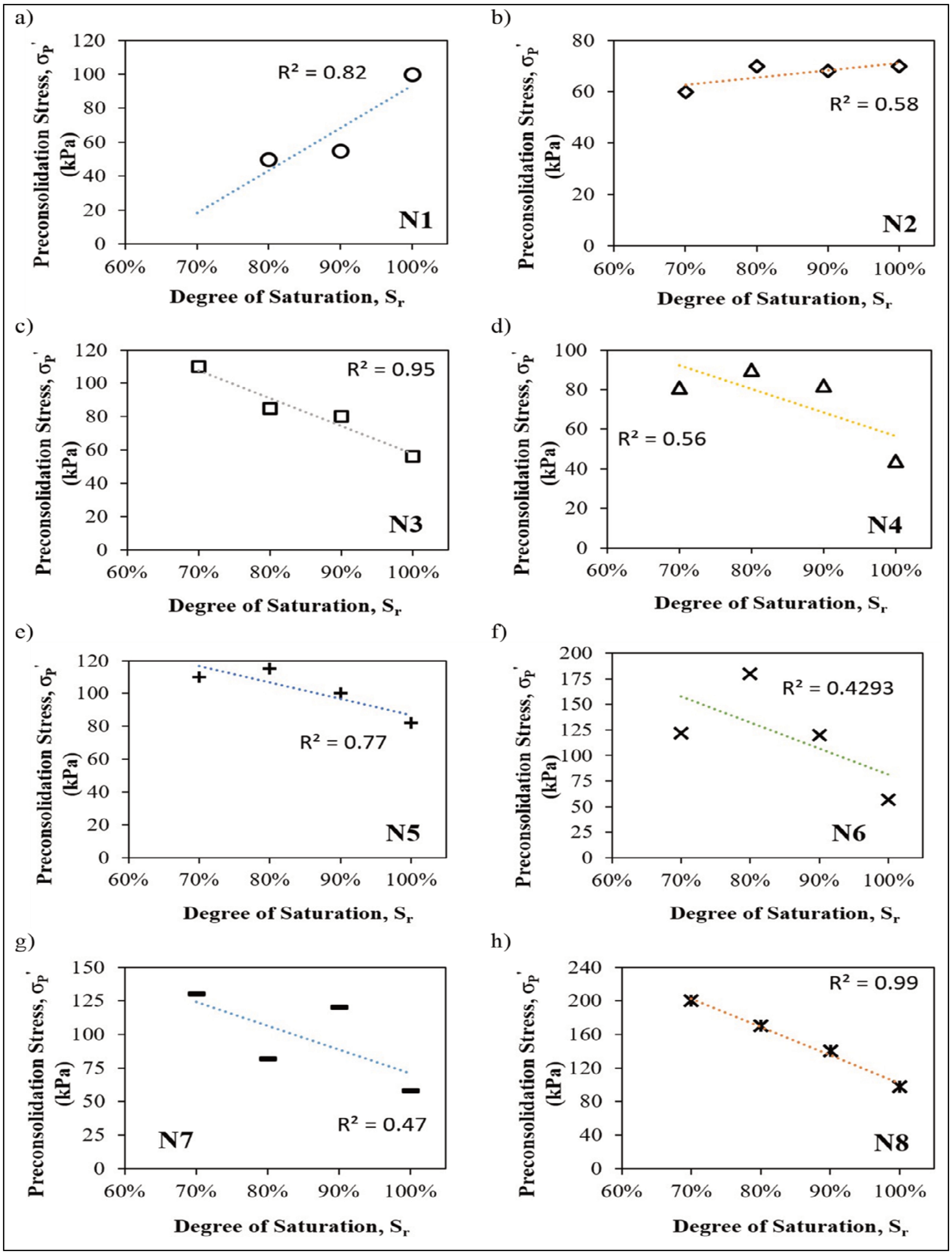

Figure 7- Graphs illustrating the relationship between the pre-consolidation pressure and the degree of saturation for a) Sample N1, b) Sample N2, c) Sample N3, d) Sample N4, e) Sample N5, f) Sample N6, g) Sample N7 and h) Sample N8.

4.7. Coefficient of Consolidation, c_v

The consolidation coefficient values of the samples vary between $1x10^{-4}$ – $14x10^{-4}$ cm^2/s (Table 4). Saturation degree-consolidation coefficient graphs for 25, 50, 100, 200 and 400 kPa stresses applied at 70%, 80%, 90% and 100% saturation degrees of all the samples are presented in Figure 8.

When the relationship between the consolidation coefficient values and the effective stresses was evaluated it was observed that the coefficients of determination of this relationship are quite low, and it is concluded that there is no significant relationship between the saturation degree and the coefficient of consolidation. In addition, the relationship between the consolidation coefficient, the degree of saturation and the plastic limit were also evaluated. It was observed that the consolidation coefficient was not affected by the degree of saturation and the plasticity value of the samples. The relationships between the coefficient of consolidation and the degree of saturation, the effective stress and the plasticity index appear to be rather random.

5. Conclusions

The following conclusions were reached at the end of this investigation. The swelling pressures of all remolded samples at statically compacted at 70, 80, 90 and 100 percent degrees of saturation range from 3-100 kPa. It was observed that the swelling pressure decreases as the degree of saturation increases, as expected. The effect of soil plasticity on the swelling pressure was prominent as well. Swelling behavior was not observed in the partially saturated low plasticity samples in general. The aim of determining the swelling pressures for partially saturated soils was to ensure that swelling did not affect the results of oedometer tests.

The general tendency between the compression index (c_c) and the degree of saturation is that the compression index increases with the increasing degree of saturation. This conclusion is valid for both the low- and high-plasticity samples. The practical meaning of this conclusion is that the total settlement calculated for fine grained soils may differ depending on the degree of saturation.

Table 4- Coefficients of consolidation (x1000) determined through the oedometer tests conducted at different degrees of saturation.

No.	σ' (kPa)	70%	80%	90%	100%
N1	25	9.0	8.7	9.5	0.0
N1	50	8.7	8.4	9.2	0.0
N1	100	9.5	9.1	9.9	13.8
N1	200	10.1	9.7	10.8	11.3
N1	400	10.4	10.2	11.2	11.3
N2	25	9.1	6.5	4.1	0.0
N2	50	7.8	5.5	1.0	11.1
N2	100	7.5	6.0	5.3	11.7
N2	200	8.1	6.6	4.6	10.2
N2	400	8.3	7.6	7.3	9.0
N3	25	5.7	9.7	8.5	0.0
N3	50	9.4	6.9	7.7	5.0
N3	100	7.6	6.7	5.2	8.3
N3	200	9.3	8.1	6.8	6.7
N3	400	7.4	6.8	5.7	6.2
N4	25	4.2	10.3	6.0	0.0
N4	50	1.2	1.7	4.8	5.5
N4	100	6.6	7.8	5.3	11.3
N4	200	7.1	5.3	6.0	5.2
N4	400	3.7	1.8	4.6	3.4
N5	25	4.7	7.3	7.7	0.0
N5	50	5.4	1.1	3.6	1.7
N5	100	5.0	7.5	4.9	8.3
N5	200	6.8	7.3	4.6	6.7
N5	400	6.4	6.2	3.5	5.8
N6	25	0.0	6.4	5.0	0.0
N6	50	0.0		7.8	0.0
N6	100	6.4	6.8	4.2	8.3
N6	200	8.0	7.8	7.0	5.0
N6	400	6.6	7.1	6.2	3.7
N7	25	7.2	7.9	0.0	0.0
N7	50	0.0	7.6	0.0	7.1
N7	100	5.1	4.2	1.1	8.1
N7	200	5.1	5.4	7.5	4.9
N7	400	5.3	4.9	5.5	2.5
N8	25	0.0	3.6	6.0	0.0
N8	50	1.9	7.7	2.2	0.0
N8	100	2.2	0.0	2.6	0.0
N8	200	2.1	1.0	2.5	3.7
N8	400	4.4	7.4	5.3	1.6

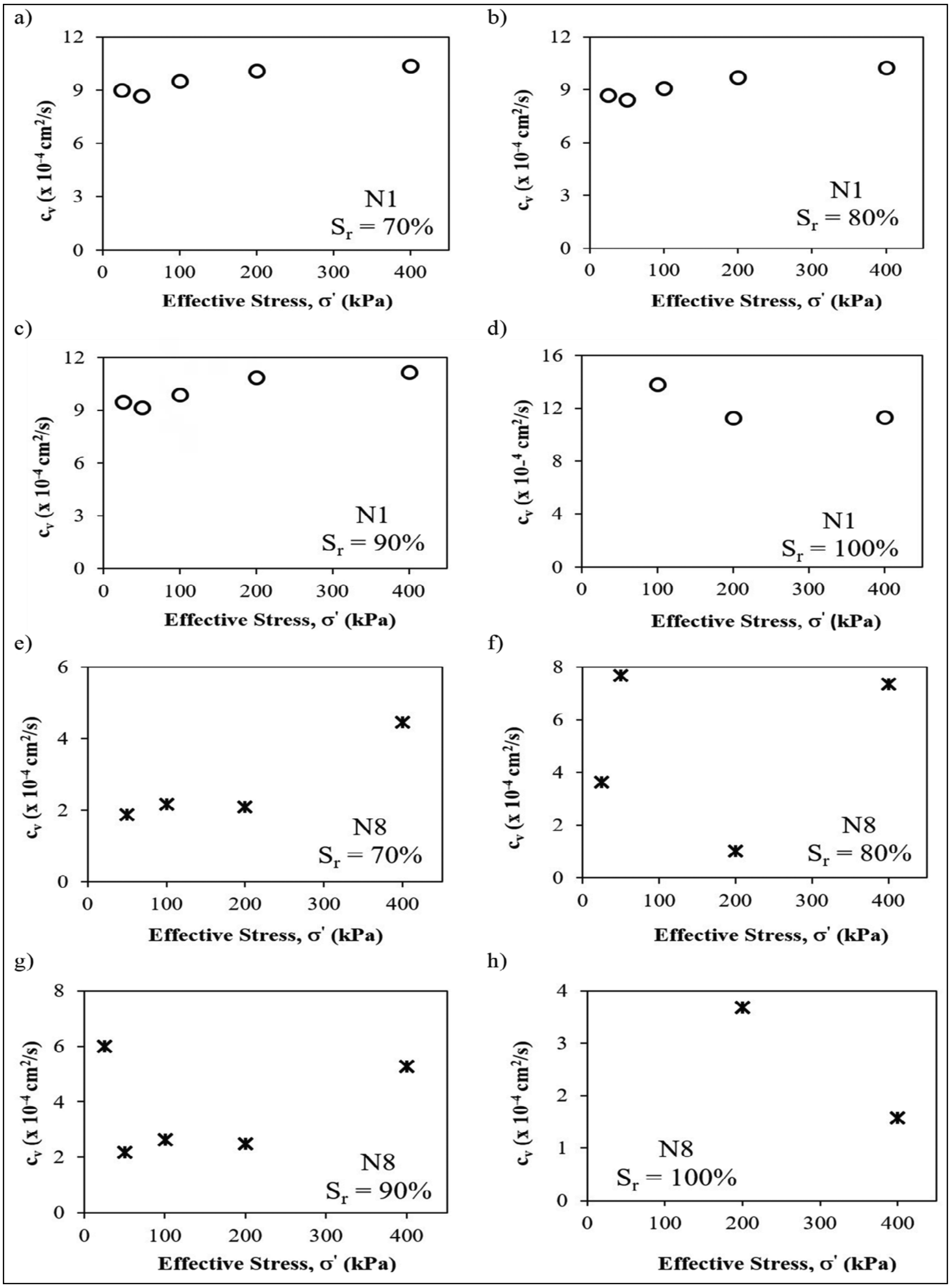

Figure 8- Consolidation coefficient versus effective stress plots for the selected two samples (i.e., N1 and N8).

Regarding the dependence of the re-compression index (c_r) on the degree of saturation, no conclusive statement can be made because of the inconsistent pattern between those two parameters while the general picture portrays that c_r tend to decrease with the increasing degree of saturation in contrary to the case of the compression index. The plasticity index did not appear to affect the c_r in general. Increasing the number of soil samples may help asserting a more conclusive statement for this correlation.

Excluding the two samples with the lowest plasticity (i.e., N1, and N2), the pre-consolidation pressure (σ_p') appears to decrease as a function of the increasing degree of saturation. In general, high-plasticity samples exhibit higher pre-consolidation pressures than do the low-plasticity soils at similar degrees of saturation.

There was no systematical pattern between the coefficient of consolidation at varying effective stresses and the degree of saturation. And alike, no conclusive statement can be asserted for the dependence of the coefficient of consolidation on soil plasticity.

The overall conclusion is that all the consolidation parameters considered herein are affected somehow by the degree of saturation of remolded soil samples. This investigation was performed on laboratory prepared identical samples. Further investigation using natural soil deposits would help better handling the settlements calculations on fine-grained soils.

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