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A Theoretical Approach to Time Rates of Settlements and Pore Water in Clayey Soils

Killi Zeminlerde Oturma ve Boşluk Suyu Hızlarına Kuramsal Bir Yaklaşım

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Abstract

In this study, the phenomenon of hydrodynamic dispersion is considered and the effects of time rates of settlements and pore water to consolidation are discussed. It is expressed that dry unit weight and plasticity of a soil are primary parameters in hydrodynamic dispersion. By means of comparing the theoretical results obtained for hydrodynamic dispersion parameters of clayey soils with the corresponding results of the performed experiments, the reliability of the theory is proven by reaching to values in a very close agreement to each other. On the other hand, it is pointed out in this study that viscous properties of soil grains can also be determined in primary consolidation. This aspect carries a great importance in slope stability problems of soils.

Keywords: Dry unit weight, Hydrodynamic dispersion, Plasticity, Soil skeleton, Time rates of settlement

Öz

Bu çalışmada, hidrodinamik dispersiyon olayı ele alınmış ve oturma hızları ile boşluk suyu hızlarının konsolidasyona olan etkileri incelenmiştir. Kuru birim hacim ağırlık ve plastisitenin hidrodinamik dispersiyonda temel göstergeler olduğu ifade edilmiştir. Killi zeminlerin hidrodinamik dispersiyon parametreleri için teorik olarak bulunan sonuçlar ile karşılıkları olan deney sonuçlarının kıyaslanmaları sonucunda, birbirlerine çok yakın değerlere ulaşılarak kuramın güvenilirliği kanıtlanmıştır. Ayrıca, bu çalışmada, birincil konsolidasyon içinde, zemin danelerine ait viskoz özelliklerin de saptanabildiği gösterilmiştir. Bu konu, zeminlerin şev stabilitesi problemlerinde önemli bir yere sahiptir.

Anahtar Kelimeler: Kuru birim hacim ağırlık, Hidrodinamik dispersiyon, Plastisite, Zemin iskeleti, Oturma hızları

1. Introduction

The first theory on consolidation was given by Terzaghi and the change in pore water pressures as a result of the variations in hydraulic gradient was defined by the following equations given below as Equation 1.a and Equation 1.b (Özaydın 2005):

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \qquad 1.a$$

$$c_v = \frac{K}{m_v \gamma_w} \qquad 1.b$$

The independent variables of the differential equation given as Equation 1.a are the coordinate z and time t. c_v and u represent the coefficient of consolidation and pore water pressure, respectively, K is the permeability of a soil and m_v is the coefficient of volume compressibility (Battaglio et al. 2003).

Terzaghi's theory of consolidation is a linear theory which considers compaction and settlement of saturated and homogeneous soils. In this theory, the coefficient of consolidation was accepted to be a parameter as a result of small deformation amounts, compatible soil water permeability and compressibility values (Geng et al 2006). In addition, compressibility and permeability of a soil were considered to stay constant for a soil layer whereas aforementioned soil parameters differ according to the type and history of a soil (Battaglio et al 2005).

On the other hand, the non-linear nature of a consolidation phenomenon and viscous property of soils cause a creep effect at the upper layers of a soil. Therefore, viscous effect

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is related to the continuous and slow deformations of a soil subjected to an external loading (Asch et al 1989). The aforementioned creep phenomenon is taken into consideration as a diffusion movement throughout a soil mass (Culling 1963).

In all of the consolidation equations, the occurrence of laminar and transient flows and the validity of Darcy's Law have been accepted. It is also known that time rate of settlement and drainage decrease as a result of consolidation. Low time rates of settlement which occur under these conditions bring out the phenomenon of hydrodynamic dispersion along with the viscous nature of fine grained soils. The term hydrodynamic dispersion stands as a process in which both grain diffusion and dispersion of a soil are considered.

Consolidation process also has a profound effect in the transportation of contaminating particles throughout a compressed porous medium. It has been observed that drainage of pore water is highly effective in the transportation of soluble particles inside of a soil mass by means of diffusion. Eventually, the random nature of pore structure of a soil mass and the transportation and spreading of contaminating particles in a soil brings out the diffusion problem. In addition, since the courses and directions of pore water change and as a result the time rates of pore water differ vastly during the application of a loading as a result of random pore geometries, the phenomenon of dispersion also comes into question.

Hydrodynamic dispersion, in general, defines the combined effect of orientation of particles to form a more disperse structure and grain diffusion occurring as a result of very low time rates of settlement. On the other hand, hydrodynamic dispersion is a phenomenon which occurs in micro pores and at the outside of a solid-liquid intersection as a result of low flow velocities (Tekinsoy 2013). Also, according to performed studies, dispersion is a physical phenomenon that occurs between solid and liquid phases as a result of low time rates of settlement. Variations in time rates of settlement occur in relation to the changes in pore water pressures (Nielsen et al. 1972). However, dispersion and grain diffusion were only studied by the writers of this study in an aspect of soil mechanics. Besides of soil mechanics point of view, the only study about the subject was in the transportation of melted particles in homogeneous soils as a result of a concentration gradient (Nielsen et al. 1972).

In this study, the phenomena of dispersion and grain diffusion in the consolidation of clayey soils are studied and effects of time rates of settlement in a consolidation phenomenon are discussed. Obtained data are given in an aspect of hydrodynamic dispersion and some contributions are presented.

2. Material and Methods

2.1. Material

Consolidation tests are generally performed on fine grained and plastic soils. For this reason, more than 200 consolidation tests were conducted in order to analyze and compare the effects of hydrodynamic dispersion in a consolidation phenomenon and the results of 96 of them were given in detail in the PhD thesis of the first writer, Dr. Barış Mahmutluoğlu (Mahmutluoğlu 2014). Among the aforementioned consolidation tests results, the results of the tests in which soil samples taken from different regions of Antalya City in Turkey with various dry unit weights and plasticity properties were used in this study. Since dry unit weight and plasticity of clayey soils are two very significant parameters in the hydrodynamic dispersion phenomenon, by using soil samples with very similar plasticity properties, only the effect of variations in dry unit weights of soil samples throughout a consolidation event are being presented in this study. Results of these experiments are both compared to the theoretical counterparts and to each other and the effect of dry unit weight in a consolidation is expressed in terms of hydrodynamic dispersion.

Physical properties of the soil samples which were chosen among the collected soil samples from Antalya City in Turkey are given in Table 1. As can be seen from Table 1, soil samples which are used in this study both have a plasticity index of 14 and so they would have very similar plasticity properties.

Table 1. Index properties of soil samples.

Sample No.	1	2
Drilling Depth (m)	4.5-4.9	1.5-1.9
Liquid Limit (w_L) (%)	38	36
Plastic Limit (w_p) (%)	24	22
Porosity (n)	0.409	0.359
Void Ratio (e)	0.693	0.560
Natural Water Content (w_n) (%)	27	13
Dry Unit Weight (γ_k) (gr/cm^3)	1.559	1.775
Unit weight of Grains (γ_s) (gr/cm^3)	2.640	2.770
Natural Unit Weight (γ_n) (gr/cm^3)	1.980	2.006
Plasticity İndex (I_p) (%)	14	14

Note: Samp. 1 and Samp. 2 represent the soil samples which were taken from Antalya from depths of 4.5-4.9 m and 1.5-1.9 m, respectively.

2.2. Method

Initially, the phenomenon was studied statistically and it was realized that dry unit weight is the primary variable in hydrodynamic dispersion (Mahmutluoğlu 2014). As a result of the performed regression analyses, the most compatible equations were constructed which were given in detail in the PhD Thesis of Dr. Barış Mahmutluoğlu and eventually Equation 2 was given as the dispersion differential equation (Mahmutluoğlu 2014):

$$\frac{\partial u}{\partial t} = v_z \frac{\partial u}{\partial z} + c_v \frac{\partial^2 u}{\partial z^2} \quad (2)$$

where, c_v and u represent the coefficient of consolidation and pore water pressure, respectively. v_z is the time rate of settlement and z and t represent the coordinate and time, respectively.

The equation of dispersion given as Equation 2 is an equation which includes convective flow relative to time rates of settlement along with the conventional consolidation term. Since dry unit weight is the primary parameter in hydrodynamic dispersion, based on the equation given in Equation 2, the following differential equation can be derived as the dispersion differential equation with respect to dry unit weight (Mahmutluoğlu 2014):

$$\frac{\partial \gamma_k}{\partial t} = v_z \frac{\partial \gamma_k}{\partial z} + D_s \frac{\partial^2 \gamma_k}{\partial z^2} \quad (3)$$

where, γ_k is dry unit weight, v_z is time rate of settlement and D_s is the diffusivity coefficient. z and t represent the coordinate and time, respectively.

Diffusivity coefficient (D_s) and the other hydrodynamic dispersion parameters were obtained by solving the differential equation given as Equation 3 (Mahmutluoğlu 2014). These solutions are used in this study to obtain diffusion and dispersion parameters of a specific region in Antalya Turkey and to show the effect of dry unit weight on the phenomenon by comparing clayey soils of similar plasticity properties.

The equations for diffusion and dispersion parameters which were found by solving the dispersion differential equation are given by Equation 4, Equation 5, Equation 6, Equation 7, Equation 8, Equation 9, Equation 10 and Equation 11 in the following pages of the study (Tekinsoy 2013; Mahmutluoğlu 2014).

3. Results and Discussion

Results of the performed consolidation tests on CL group soil samples named as CL-1 (drilling depth: 4.5-4.9 m) can be seen in Table 2. Based on the results given in Table 2, diffusive and dispersive parameters of the CL-1 sample are obtained and presented in Table 3.

Results of the performed consolidation tests on the second CL group soil samples named as CL-2 (drilling depth: 1.5-1.9 m) can be seen in Table 4. Based on the results given in Table 4, diffusive and dispersive parameters of the CL-2 sample are obtained and presented in Table 5.

Table 2. Effective stress variations relative to dry unit weights for the cl-1 group sample.

Total Press p (kg/cm^2)	Samp. Height H (mm)	Void Ratio e (%)	Pressure Increment Δp (kg/cm^2)	Dry Unit Weight γ_k (gr/cm^3)	Coeff. of Volume Comp. m_v (cm^2/kg)	Efective Stress Increment σ' (kg/cm^2)
0.00	20.000	69.344	69.344	1.559	-	-
0.25	19.734	67.092	67.092	1.580	0.05391	0.25
0.50	19.497	65.085	65.085	1.599	0.04863	0.25
1.00	19.185	62.443	62.443	1.625	0.03253	0.50
2.00	18.820	59.353	59.353	1.657	0.01943	1.01
4.00	18.240	54.442	54.442	1.709	0.01590	1.94
2.00	18.283	54.806	54.806	1.705	0.00118	1.99
1.00	18.348	55.356	55.356	1.699	0.00354	1.00
0.50	18.408	55.864	55.864	1.694	0.00652	0.45
0.25	18.462	56.322	56.322	1.689	0.01172	0.25
0.00	18.630	57.744	57.74	1.674	0.03606	0.25

Table 3. Dispersion results of the CL-1 group soil sample.

Total Press P (kg/cm^2)	Dry Unit Weight γ_k (gr/cm^3)	Dif. Coeff. Ds×10^{-5} (cm^2/dk)	Disp. Vrb. x -	Disp. Flux Js (gr/cm^2dk)	Dispers. Soil Amount ΔWs (gr)	Total Comp. Soil ΔWt (gr)
0.00	1.559	-	-	-	-	-
0.25	1.580	0.905	8.642	3.45×10-39	9.75×10-35	0.010968
0.50	1.599	1.681	6.266	2.17×10-23	6.14×10-19	0.039506
1.00	1.625	2.658	4.903	1.84×10-16	5.20×10-12	0.105616
2.00	1.657	3.739	4.055	6.44×10-13	1.82×10-8	0.227059
4.00	1.709	5.321	3.295	3.13×10-10	8.85×10-6	0.518363
2.00	1.705	5.209	3.338	2.27×10-10	6.42×10-6	0.492213
1.00	1.699	5.039	3.406	1.35×10-10	3.82×10-6	0.454117
0.50	1.694	4.880	3.472	0.82×10-10	2.32×10-6	0.421994
0.25	1.689	4.735	3.535	0.50×10-10	1.41×10-6	0.392581
0.00	1.674	4.276	3.754	0.09×10-10	0.25×10-6	0.309349

Note: *Since consolidation concludes at the end of each pressure increment, the value of time rate of settlement v_z was taken as v_z=0.*

Table 4. Effective stress variations relative to dry unit weights for the CL-2 group sample.

Total Press p (kg/cm^2)	Samp. Height H (mm)	Void Ratio e (%)	Pressure Increment Δp (kg/cm^2)	Dry Unit Weight γ_k(gr/cm^3)	Coeff. Of Volume Comp. m_v (cm^2/kg)	Efective Stress Increment σ' (kg/cm^2)
0.00	20.000	56.043	-	1.775	-	-
0.25	19.885	55.146	0.25	1.785	0.02313	0.24
0.50	19.745	54.054	0.25	1.798	0.02835	0.26
1.00	19.552	52.548	0.50	1.816	0.01974	0.50
2.00	19.268	50.332	1.00	1.843	0.01474	1.00
4.00	18.945	47.812	2.00	1.874	0.00852	1.96
2.00	19.020	48.397	2.00	1.867	0.00197	1.90
1.00	19.102	49.037	1.00	1.859	0.00429	1.00
0.50	19.198	49.786	0.50	1.849	0.01000	0.54
0.25	19.312	50.675	0.25	1.838	0.02360	0.25
0.00	19.578	52.751	0.25	1.813	0.05436	0.25

As can be seen from Table 2 and Table 4, the effective stress increment values in the final columns which were found theoretically (σ') are very close to the experimental counterparts (p) in the first columns both for the CL-1 and CL-2 samples, individually. These theoretical values were reached by using Equation 4 which is given below for effective stresses and this equation, as mentioned previously, was obtained by solving the differential equation in Equation 3 (Tekinsoy 2013; Mahmutluoğlu 2014).

$$\sigma' = \frac{1}{m_v} ln \frac{\gamma_{k2}}{\gamma_{k1}} \tag{4}$$

where, m_v is the coefficient of volume compressibility, γ_{k1} and γ_{k2} are the initial and final dry unit weights for any pressure increment, respectively.

Comparison of effective stress variations obtained both theoretically and experimentally can be seen in the graphics given in Figure 1 and Figure 2 for the CL-1 and CL-2 group soil samples, respectively.

If variations in dry unit weights of the samples named as CL-1 and CL-2 are considered, it can be computed from the consolidation results from Table 2 and Table 3 that the CL-1 sample has a dry unit weight variation of 9.62 % in the loading phase and 7.38 % throughout the whole consolidation test whereas the CL-2 sample has a dry unit

Table 5. Dispersion results of the CL-2 group soil sample.

Total Press P (kg/cm²)	Dry Unit Weight γ_k (gr/cm³)	Dif. Coeff. Ds×10^{-5} (cm²/dk)	Disp. Vrb. x-	Disp. Flux Js (gr/cm²dk)	Dispers. Soil Amount ΔWs (gr)	Total Comp. Soil ΔWt (gr)
0.00	1.775	-	-	-	-	-
0.25	1.785	0.396	13.166	1.55×10^{-82}	4.38×10^{-78}	0.002258
0.50	1.798	0.869	8.825	1.51×10^{-40}	4.26×10^{-36}	0.011516
1.00	1.816	1.504	6.643	1.62×10^{-25}	4.58×10^{-21}	0.036065
2.00	1.843	2.403	5.179	1.11×10^{-17}	3.14×10^{-13}	0.097735
4.00	1.874	3.377	4.296	8.26×10^{-14}	2.34×10^{-9}	0.205077
2.00	1.867	3.155	4.462	1.73×10^{-14}	4.89×10^{-10}	0.177029
1.00	1.859	2.910	4.666	0.24×10^{-14}	0.68×10^{-10}	0.148110
0.50	1.849	2.619	4.943	1.38×10^{-16}	0.39×10^{-11}	0.116530
0.25	1.838	2.267	5.344	1.76×10^{-18}	4.98×10^{-14}	0.085106
0.00	1.813	1.419	6.848	9.16×10^{-27}	2.59×10^{-22}	0.031487

Note: *Since consolidation concludes at the end of each pressure increment, the value of time rate of settlement v_z was taken as v_z=0.*

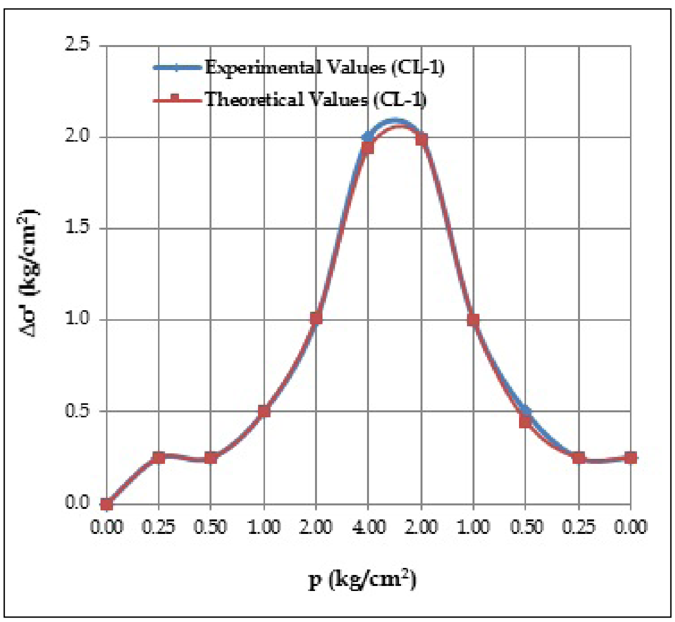

Figure 1. The Relationship between Theoretical (Δσ') and Experimental (p) Effective Stress Increments for the CL-1 Soil Sample.

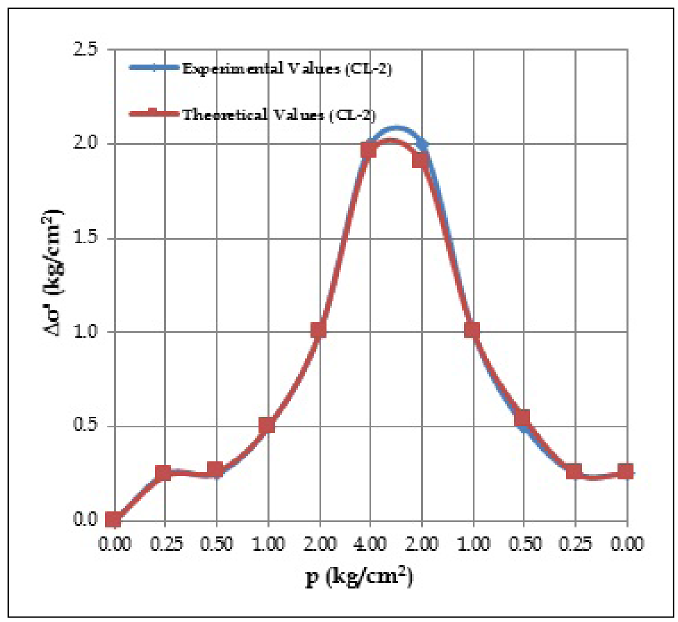

Figure 2. The Relationship between Theoretical (Δσ') and Experimental (p) Effective Stress Increments for the CL-2 Soil Sample.

variation of 5.58 % for the loading phase and 2.14 % for the entire test. Therefore it can be understood that the CL-1 sample which has a higher variation percentage of dry unit weight than the CL-2 sample should exert higher values of hydrodynamic dispersion parameters and has a higher time rate of settlement than that for the CL-2 sample.

In order to observe the aforementioned comparison between the CL-1 and CL-2 samples, if Table 3 and Table 5 are considered, it can be seen that the values for diffusivity coefficients (D_s) in column 3 for the CL-1 sample are higher than that for the CL-2 sample. These values were obtained by using the equation given below as Equation 5 (Mahmutluoğlu 2014):

$$D_s = \frac{z^2}{4t} ln \frac{z_i}{z} \tag{5}$$

where, z_i ve z are the initial and any sample heights, respectively, $z^2/4$ is the squared power of the drainage path z/2 and t is time (Mahmutluoğlu 2014).

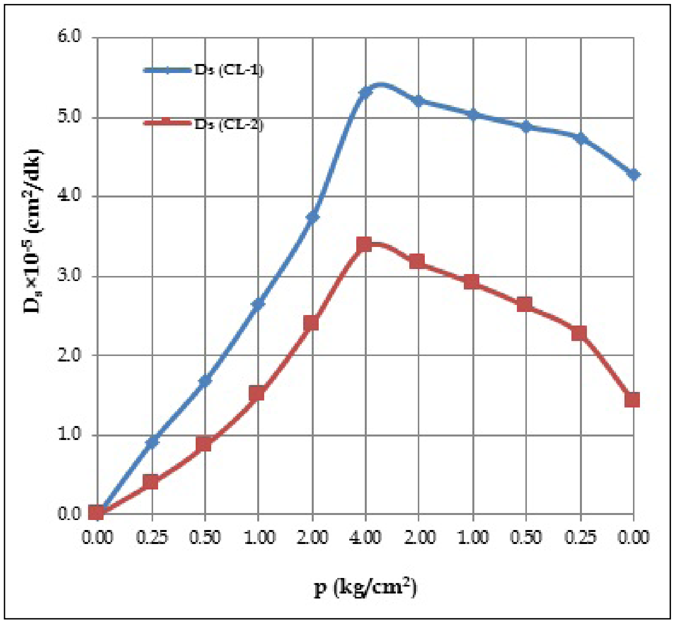

Figure 3. The Relationship of Diffusivity Coefficients of the CL-1 and CL-2 Soil Samples relative to Pressure Increments (p).

The relationship between the diffusivity coefficients (D_s) of the CL-1 and CL-2 samples can be seen from the graphic given in Figure 3 below:

It can be seen from Figure 3 that the CL-1 sample which has a higher dry unit weight variation percentage than the CL-2 sample has higher values of diffusivity coefficients (D_s) for all pressure increments or throughout the entire test.

The values for flux (J_s) and dispersive variable (x) in Table 3 and Table 5 were found by using the equations below which are given as Equation 6.a and Equation 6.b, respectively (Mahmutluoğlu 2014):

$$J_s = \sqrt{\frac{D_s}{\pi t}}.(\gamma_{kf} - \gamma_{ki})e^{-x^2} \quad (6.a)$$

$$x = \frac{z + v_z t}{2\sqrt{D_s t}} \quad (6.b)$$

where, J_s is flux, x is the dispersive variable which is used in the computation of the flux J_s, D_s is the diffusivity coefficient, γ_{ki} and γ_{kf} are the initial and final values for dry unit weights, respectively, v_z is the time rate of settlements, z is sample height at any instant and t is time (Mahmutluoğlu 2014).

If the derivative of the equation for D_s given as Equation 5 is taken with respect to z, the following equation is obtained for time rate of settlements v_z (Mahmutluoğlu 2014):

$$v_z = \frac{\partial D_s}{\partial z} = -\frac{z}{2t}\left[In\left(\frac{z}{z_i}\right) + \frac{1}{2}\right] \quad (7.a)$$

where, v_z is the time rate of settlements, z/2 repsesents the drainage path and H_d/t which can also be given as z/2t is the variation in time rate of the drainage path and t is time (Mahmutluoğlu 2014). In the expression in Equation 7.a, the term 1/2 which is added to the logarithmic term in the parenthesis remains very insignificant relatively and so it can be neglected to simplify the equation. The newly formed equation becomes equal to the time rate of settlements of the soil samples which is given below as Equation 7.b (Mahmutluoğlu 2014).

$$v_z = \frac{z}{2t} ln \frac{z}{z_i} \quad (7.b)$$

where, v_z is the time rate of settlements, z_i and z are the initial and any sample deformation heights, respectively and t is time. Eventually, it can be expressed that the time rate of dispersion, v_z, is a velocity term which includes both soil settlements and their dispersion properties and which has a higher value than the aforementioned velocity terms. In other words, the time rate of settlements given in Equation 7.a and Equation 7.b is a velocity term which includes all terms of time rates. The occurrence of $_{vz}$ in the equations includes the effects of time rates of settlement to the phenomenon.

If the time rates of pore water are to be considered (v_{z2}), the following expression can be given as Equation 8 which was derived from the solutions of the dispersion differential equation given as Equation 3 (Mahmutluoğlu 2014).

$$v_{z2} = \frac{z}{2t}\left[1 - \left(\frac{z}{z_i}\right)\right] \quad (8)$$

where, v_{z2} is the time rate of pore water, z_i and z are the initial and any sample deformation heights, respectively and t is time. By using the equation given as Equation 8, time rates of pore water in a soil can be obtained for any pressure increment or for any moment throughout a consolidation event. Since time rates of both settlements and pore water can be determined, time rate of compression of the soil skeleton, v_s, can also be obtained by using the Equation 9 given below (Mahmutluoğlu 2014):

$$v_s = v_z - v_{z2} \quad (9)$$

where, v_s is the time rate of compression of the soil skeleton, v_z is the time rate of settlements and v_{z2} is the time rate of pore water.

In order to compare the experimental and theoretical time rates of settlements for the samples CL-1 and CL-2, the following tables in Table 6 and Table 7 are given, respectively.

In the following graphical relationships in Figure 4 and Figure 5 given for the CL-1 and CL-2 samples, respectively, it can clearly be observed that the values of theoretical and experimental time rates of settlement and pore water are in a very close agreement to each other, individually.

In the following graphic given as Figure 6, experimental and theoretical time rates of settlements for the CL-1 and CL-2 soil samples are compared both individually and with each other:

As can be observed from Figure 6, experimental and theoretical time rates of settlements of both the CL-1 and CL-2 soil samples are in a very close agreement to each other individually and the values of both experimental and theoretical time rates of settlements for the CL-1 sample

Table 6. Comparison of experimental and theoretical time rates of settlements for the CL-1 soil sample.

Total Press.	Sample Height	Settlement Difference	Experimental Time Rate of Settlement	Theoretical Time Rate of Settlement	Time Rate of Pore Water	Time Rate of Compression of Soil Skeleton
p(kg/cm²)	H mm	ΔH mm	$v_z = \frac{\Delta H}{2t}$ (×10⁻⁴) mm/dk	$v_z = \frac{z}{2t} ln\left(\frac{z}{z_i}\right)$ (×10⁻⁴) mm/dk	$v_{z2} = \frac{z}{2t}\left[1 - \left(\frac{z}{z_i}\right)\right]$ (×10⁻⁴) mm/dk	$v_s = v_z = v_{z2}$ (×10⁻⁵) mm/dk
0.00	20.000	-	-	-	-	-
0.25	19.778	0.266	0.92	0.92	0.91	0.13
0.50	19.502	0.503	1.75	1.72	1.70	0.50
1.00	19.188	0.815	2.83	2.77	2.72	0.60
2.00	18.754	1.180	4.10	3.97	3.86	2.40
4.00	18.222	1.760	6.11	5.83	5.57	5.40
2.00	18.322	1.717	5.96	5.70	5.45	5.10
1.00	18.492	1.652	5.74	5.49	5.26	4.80
0.50	18.672	1.592	5.52	5.30	5.09	4.30
0.25	18.742	1.538	5.34	5.13	4.93	4.10
0.00	18.988	1.370	4.76	4.59	4.43	3.30

Table 7. Comparison of experimental and theoretical time rates of settlements for the CL-2 soil sample.

Total Press.	Sample Height	Settlement Difference	Experimental Time Rate of Settlement	Theoretical Time Rate of Settlement	Time Rate of Pore Water	Time Rate of Compression of Soil Skeleton
p(kg/cm²)	H mm	ΔH mm	$v_z = \frac{\Delta H}{2t}$ (×10⁻⁴) mm/dk	$v_z = \frac{z}{2t} ln\left(\frac{z}{z_i}\right)$ (×10⁻⁴) mm/dk	$v_{z2} = \frac{z}{2t}\left[1 - \left(\frac{z}{z_i}\right)\right]$ (×10⁻⁴) mm/dk	$v_s = v_z = v_{z2}$ (×10⁻⁵) mm/dk
0.00	20.000	-	-	-	-	-
0.25	19.885	0.115	0.40	0.40	0.40	0.03
0.50	19.745	0.255	0.89	0.88	0.87	0.11
1.00	19.552	0.448	1.56	1.54	1.52	0.40
2.00	19.268	0.732	2.54	2.50	2.45	0.90
4.00	18.945	1.055	3.66	3.57	3.47	1.90
2.00	19.020	0.980	3.40	3.32	3.24	1.60
1.00	19.102	0.898	3.12	3.05	2.98	1.40
0.50	19.198	0.802	2.78	2.73	2.67	1.10
0.25	19.312	0.688	2.39	2.35	2.31	0.80
0.00	19.578	0.422	1.47	1.45	1.43	0.40

are higher than that for the CL-2 sample throughout the consolidation of the samples. This relationship given in Figure 6 reemphasizes the aforementioned effect that since the CL-1 sample has a higher dry unit weight variation percentage than the CL-2 sample, the CL-1 samples are observed to have higher values of time rates of settlements for all pressure increments or throughout the entire consolidation test.

In the following graphical relationships which are given as Figure 7 for time rates of pore water and Figure 8 for time rates of compression of the soil skeleton, it can also

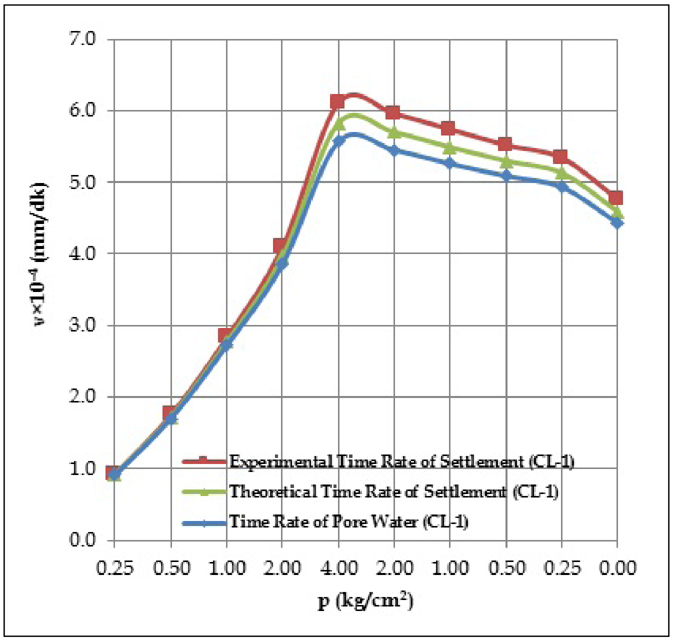

Figure 4. The Relationship of Experimental and Theoretical Time Rates of Settlement and Pore Water for the CL-1 Soil Sample.

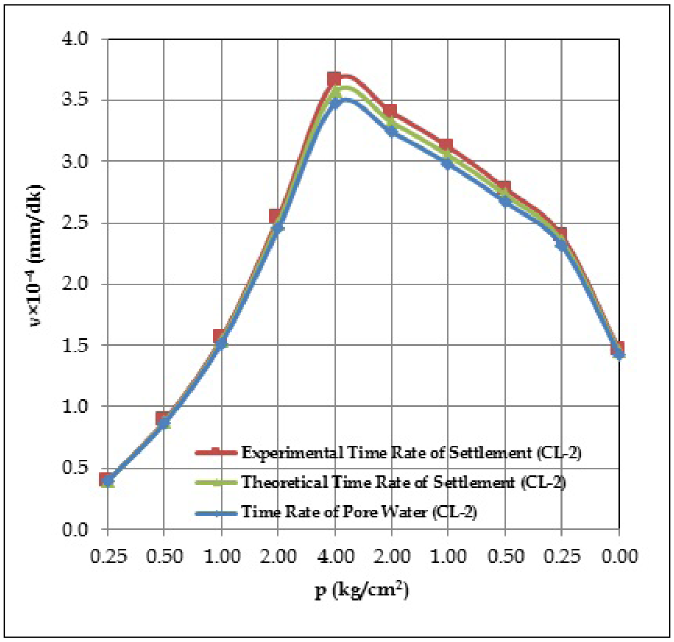

Figure 5. The Relationship of Experimental and Theoretical Time Rates of Settlement and Pore Water for the CL-2 Soil Sample.

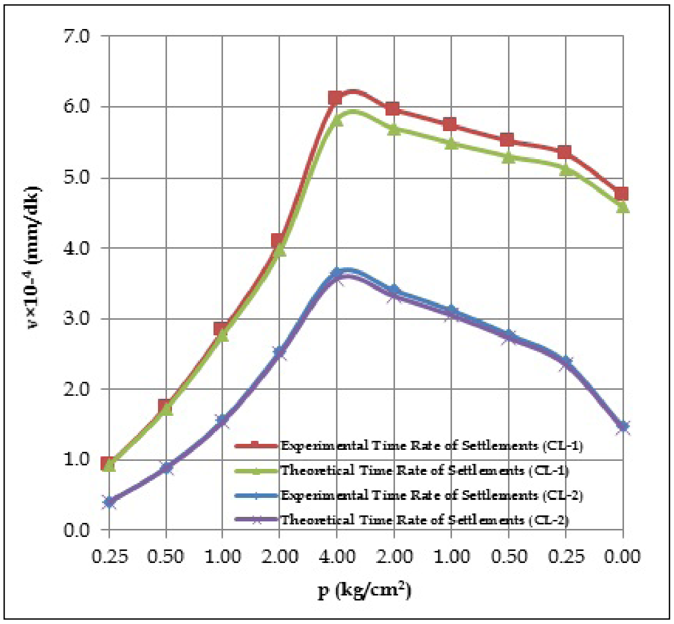

Figure 6. The Relationship of Experimental and Theoretical Time Rates of Settlements of the CL-1 and CL-2 Soil Samples relative to Pressure Increments (p).

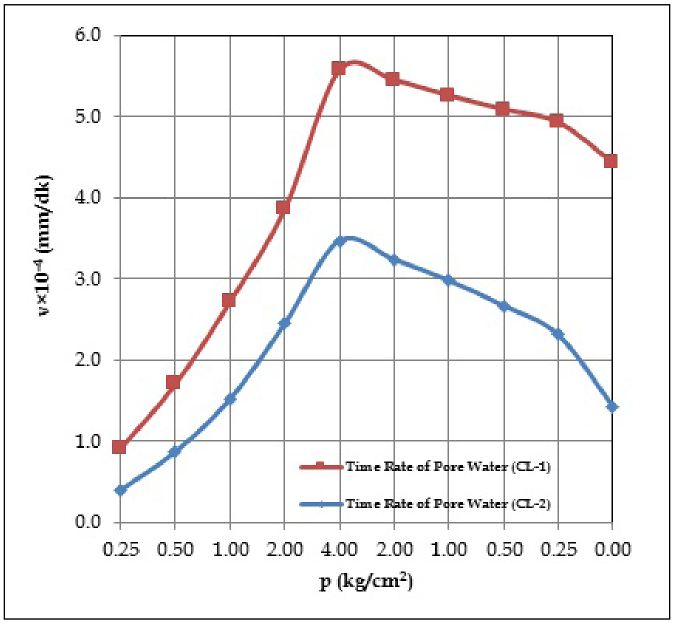

Figure 7. The Relationship of Time Rates of Pore Water for the CL-1 and CL-2 Soil Samples.

be observed that the values for the CL-1 sample are higher than those for the CL-2 sample throughout the entire consolidation process.

On the other hand, the amount of soil which fills into macro pores and diffuses inside the soil matrix (6th columns of Table 3 and Table 5 for the CL-1 and CL-2 samples, respectively) were found by using Equation 10 below (Mahmutluoğlu 2014):

$$\Delta W_s = J_s A \Delta t \tag{10}$$

where, ΔW_s is the amount of diffused soil mass, J_s is flux, A is the cross sectional area of a sample and t is time (Mahmutluoğlu 2014). Since variables of both time and location exist in Equation 10, the amount of dispersive soil can be obtained at any pressure increment or for any instant throughout a consolidation process.

In the last columns of Table 3 and Table 5, total compacted soil amounts (ΔW_t) are given for the CL-1 and CL-2 samples, respectively. These values were obtained by using Equation 11 below (Mahmutluoğlu 2014):

$$\Delta W_t = A \Delta z \Delta \gamma_k \tag{11}$$

where, ΔW_t is the total compacted soil mass, A is the cross sectional area, z is the deformational height, γ_k is dry unit weight and this equation represents total compaction (Mahmutluoğlu 2014). If the total compacted amounts of the CL-1 and CL-2 samples are compared, it can be seen that all of the values are lower for the CL-2 sample. This relationship can be observed in the graphic given in Figure 9.

As can be seen from the graphical relationship given in Figure 9, the total amount of compacted soil for the CL-2 sample is lower than that for the CL-1 sample throughout the consolidation process again as a result of lower percentage of dry unit weight variation which takes place in the consolidation of the CL-2 soil samples than that for the CL-1 samples.

Therefore, the CL-1 sample is compacted more than the CL-2 sample and more soil particles are transported, oriented and compacted in terms of grain diffusion and dispersion in the consolidation of the CL-1 soil sample.

4. Conclusion

In this study, an approach was presented in which time rates of settlements and pore water and their effects on consolidation were analyzed in terms of hydrodynamic dispersion.

A result of this study is that it enables to acknowledge information on the subject of hydrodynamic dispersion in a consolidation phenomenon. In this aspect, values for variations in effective stresses, diffusivity coefficients, dispersive flux, dispersive and total compacted soil amounts and time rates of settlements, time rates of pore water and time rates of compression of soil skeletons were obtained by solving the differential equation given as Equation

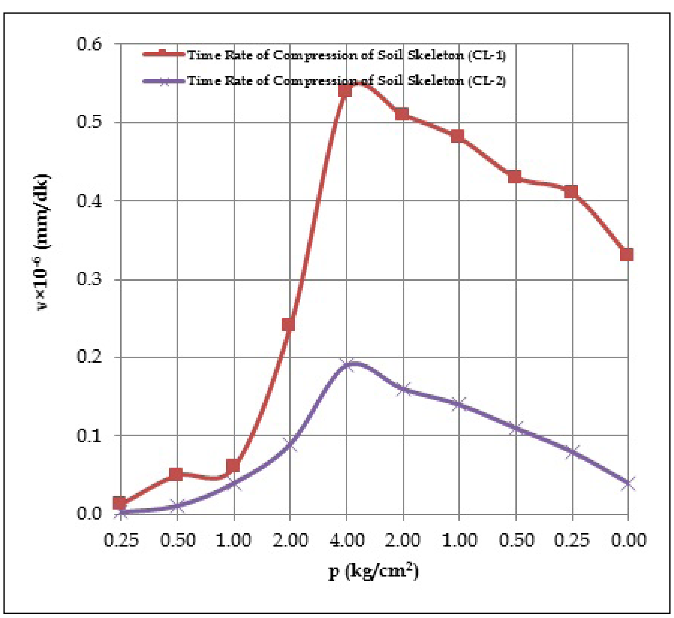

Figure 8. The Relationship of Time Rates of Compression of Soil Skeleton for the CL-1 and CL-2 Soil Samples.

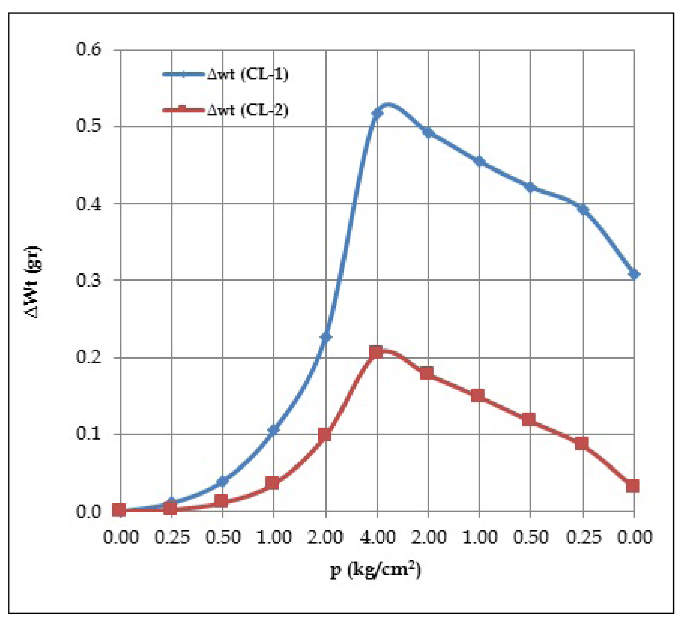

Figure 9. Total Compacted Soil Amounts (ΔWt) for the CL-1 and CL-2 Soil Samples with respect to Pressure Increments (p).

(3) being related to hydrodynamic dispersion in terms of dry unit weight variations. These equations which were obtained theoretically were shown to give results that are in a very close agreement to the experimental counterparts. On the other hand, diffusive and dispersive characteristic of a specific region in Antalya Turkey was examined in this study and the effects of dry unit weight and plasticity were discussed to shed some light to the phenomenon of hydrodynamic dispersion.

By considering the phenomenon of consolidation in an aspect of hydrodynamic dispersion, a great deal of information can be obtained about time rates of settlements and time rates of pore water. Consequently, viscous properties of a soil could be considered as a result of using the presented theory. Viscous properties of soils are of great importance in the examination and solution of slope stability problems. Eventually, it can be pointed out that the presented theory enables a more thorough understanding and perspective about the subject.

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