Estimating Wetting-induced Settlement of Compacted Soils using Oedometer Test

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ABSTRACT: Compacted soils undergo volume changes when wetted. Oedometer tests have been commonly used to estimate the settlement of unsaturated soils when wetted. Several variations of the oedometer test are available. The double-oedometer test has been more popular as it requires only two nominally identical specimens for the test to produce the unsaturated and saturated compression curves. The wetting-induced settlement of a compacted soil at any load can be estimated by the difference in ordinate between the unsaturated and saturated compression curves. In this paper, it is shown that the unsaturated and saturated compression curves are linked by the initial degree of saturation and soil type. The compression curve of an unsaturated compacted soil can be derived from the compression curve of an inundated compacted soil, making it possible to estimate the wetting-induced settlement of compacted soils using a single oedometer test on an inundated specimen.

1. INTRODUCTION

Compacted soils are used in many engineering constructions. As compacted soils are unsaturated, wetting can lead to changes in volume. The amount of volume change depends on a number of factors: soil type and structure, percentage of fines particularly clay content, initial soil density, imposed stress state and degree of saturation (Jennings and Burland 1962, Barden et al. 1973, Hodek and Lovell 1979, Houston and Houston 1997). The volume change of an unsaturated soil due to wetting can be in the form of swell or collapse. Soils containing significant amount of expansive clay minerals such as montmorillonite are expected to exhibit swelling behaviour when wetted. On the other hand, meta-stable structured soils containing inactive clay minerals where the clay particles only act as bonding agent between the grains to form an open structure are expected to collapse when water is introduced (Lawton et al 1992). Examples of meta-stable structured soils are loess and poorly compacted soils. Any compacted soils under certain conditions can collapse upon wetting (Houston and Houston 1997). However, depending on the water content of the soil, collapse may take place progressively rather than as a sudden settlement (Barksdale and Blight 1997). More generally a soil will swell when wetted under low loads and collapse when wetted at higher loads as illustrated in Figure 1. The amount of heave and the amount of collapse are indicated in Figure 1. The objectives of this paper are to examine the unsaturated and saturated compression curves from double-oedometer tests and to develop a method to estimate the settlement of compacted soils due to wetting.

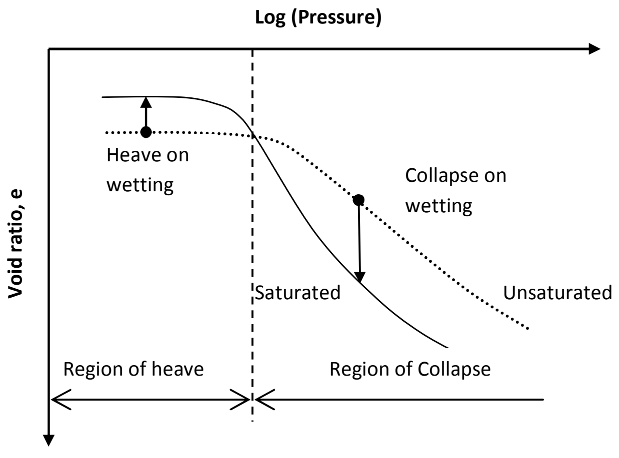


Figure 1 Volume change behavior of a soil due to wetting

2. QUANTIFICATION OF VOLUME CHANGE

It is widely recognised that any type of soil compacted dry of optimum and at a low dry density may develop a collapsible fabric of meta-stable structure (Barden et al. 1969, 1973). The microstructure of the soil is important in determining the collapse behaviour of the soil (Alonso et al. 1993). A compacted and meta-stable soil structure is supported by microforces of shear strength that are highly dependent on matric suction (Matyas and Radhakrishna 1968; Pereira and Fredlund 2000). The bonds start to lose strength upon wetting and at a critical degree of saturation the soil structure collapses (Jennings and Knight 1957, Barden at al. 1973). Above the critical degree of saturation, negligible collapse will occur (Jennings and Burland 1962, Houston et al. 1993).

The quantification of volume change that occurs when a soil undergoes wetting is usually determined in a one-dimensional oedometer test. There are several variations of the oedometer tests to determine the amount of soil collapse. These include the double-oedometer test (Jennings and Knight 1957), the single specimen collapse test (Houston et al. 1988; ASTM D5333 1998a), and a single point, multiple specimens test procedure (Noorany 1992). In the double-oedometer tests, two nominally identical specimens are tested in the oedometer. One is tested in the in situ or as compacted condition giving the unsaturated compression curve while the other being inundated with water is tested at saturation to give the saturated compression curve. The collapse strain is given by the difference in ordinate between the unsaturated and saturated compression curves. In a single specimen collapse test, a specimen is loaded to a fixed total stress of 200 kPa (ASTM D5333) and then inundated. The collapse strain given by the change in ordinate when water is introduced can be converted to a collapse index. The collapse index is an indicator for identifying collapsible soils and does not provide an estimate of the potential settlement (Houston and Houston 1997). In the single point, multiple specimen test, a specimen is loaded to a particular total stress and then inundated to obtain a single point on the saturated compression curve. The next specimen is then loaded to a higher total stress and then inundated. By repeating the procedure for a whole range of total stresses, the saturated compression curve can be traced. Double-triaxial tests equivalent to double-oedometer tests have also been used to quantify volume change (Lawton et al. 1991). Generally the double-oedometer tests are more commonly used for assessing the amount of soil settlement from the unsaturated state to the saturated state.

3. SOIL MATERIALS AND TEST PROCEDURES

Three soils were used in the study: kaolin, sedimentary residual soils JFA and JFB. The basic properties of the soils are summarized in Table 1 and their grain size distributions are given in Figure 2. The kaolin used was from a commercial source. Residual soils JFA and JFB were derived from a sedimentary rock formation. The three soils were chosen as their plasticity index and activity values are spaced apart. The standard Proctor compaction curves of the soils obtained in accordance with ASTM D698 (1998b) are shown in Figure 3. Soil samples were prepared at different water contents by compaction using the standard Proctor compaction effort as indicated in Figure 3. Each of the soil samples would have a different soil structure due to the different compaction conditions.

Table 1 Summary of soil properties

Soil Properties		Kaolin	Residual Soil	
			Soil JFA	**Soil JFB**
Liquid limit		61	29	35
Plastic limit		44	20	22
Plasticity index		17	9	13
Specific gravity		2.6	2.62	2.71
Grain size distribution				
	Sand	-	60%	50%
	Silt	92%	20%	39%
	Clay	8%	20%	11%
Activity		2.13	0.45	1.18
USCS		MH	SC	SC

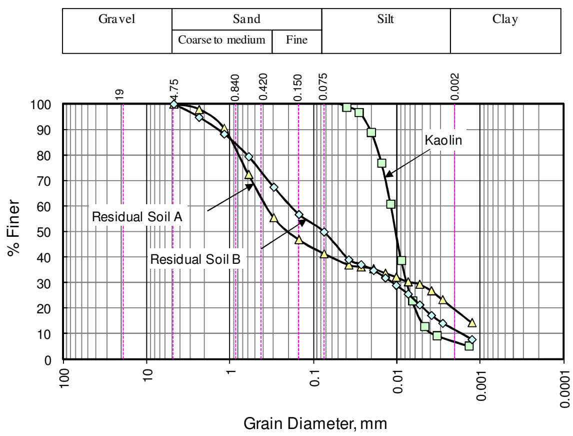


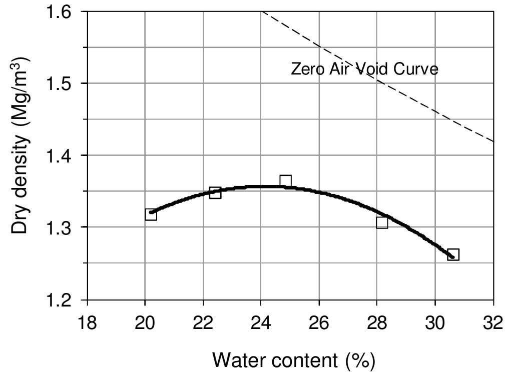
Figure 2 Grain size distribution of soil samples

Two or three soil specimens were extruded from each compacted sample using a cookie-cutter method directly into an oedometer ring. The inside surface of the oedometer ring was lightly oiled to minimize friction. The surfaces of the soil specimens were then trimmed flat to flush with the oedometer ring and carefully examined to ensure that they do not contain any cracks.

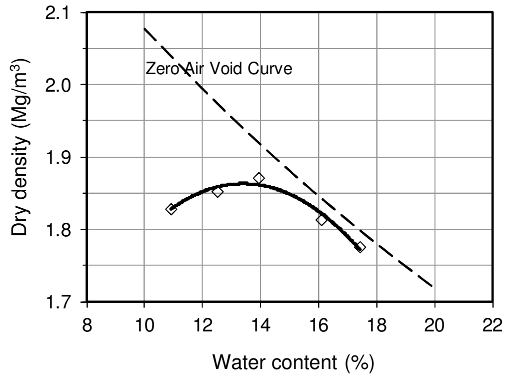
The soil specimen in the oedometer ring was then placed into the oedometer apparatus. One soil specimen was tested as compacted, i.e. unsaturated, while the other specimen was tested saturated, i.e. the specimen was inundated with water after placement of the seating load (~5 kPa). The double-oedometer test procedures followed that recommended by Jennings and Knight (1957). Selected soil specimens were also tested using the single oedometer test in accordance with ASTM D5333 (1998a) except the loading was continued after inundation with water. The soil specimen was inundated with water at 200 kPa and left for 24 hours before loading was continued to the final load increment. The single oedometer compression curve was compared with the unsaturated and saturated compression curves of the double-oedometer test.

4. RESULTS AND ANALYSES

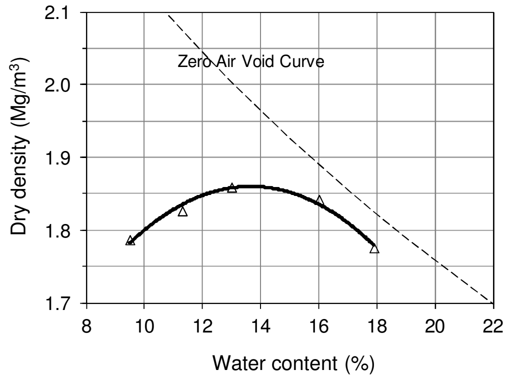
The double-oedometer test results for the kaolin, residual soils JFA and JFB at various compaction water contents are shown in Figures 4 to 6, respectively. In Figures 4 to 6, the x-axis label is in term of net normal stress ($\sigma - u_a$) instead of σ'. The net normal stress ($\sigma - u_a$) is applicable to both unsaturated and saturated soils. When the soil is saturated, $u_a = u_w$ and ($\sigma - u_a$) becomes σ'. The figures show that the unsaturated soil specimens have a lower compressibility than the saturated soil specimens. The basis of the double-oedometer test is that a soil specimen tested unsaturated will “collapse” onto the saturated compression curve when inundated at any of the vertical loads and any further loading will follow the saturated compression curve. Comparison of the single and double-oedometer test results for two residual soil specimens are shown in Figure 7. Figure 7 shows that the single oedometer compression curve traces the unsaturated compression curve of the double-oedometer test until the point of water inundation and subsequently traces the saturated compression curve of the double-oedometer test approximately. This suggests that the single and double-oedometer tests are comparable in estimating wetting-induced settlement of compacted soils.



(a) Kaolin

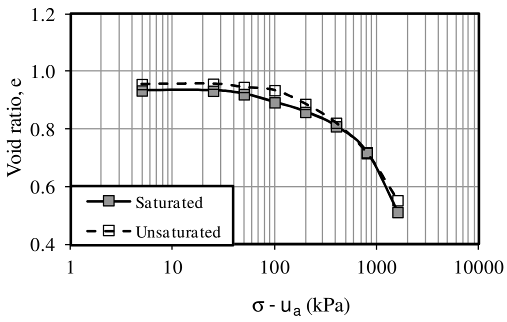


(b) Residual soil JFA

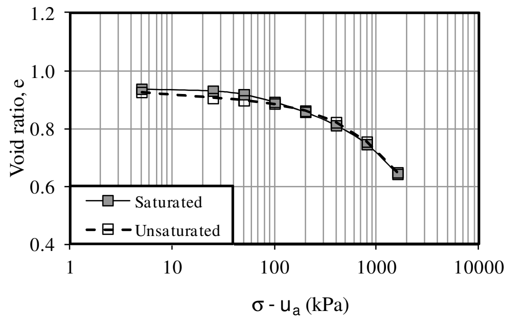


(c) Residual soil JFB

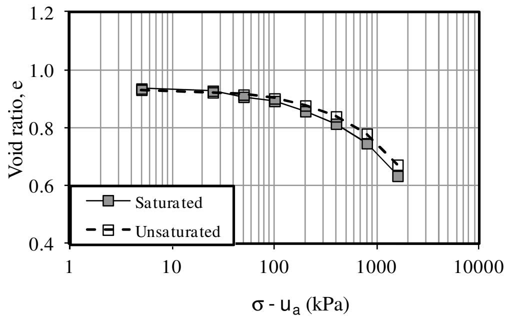
Figure 3 Compaction properties of soil samples



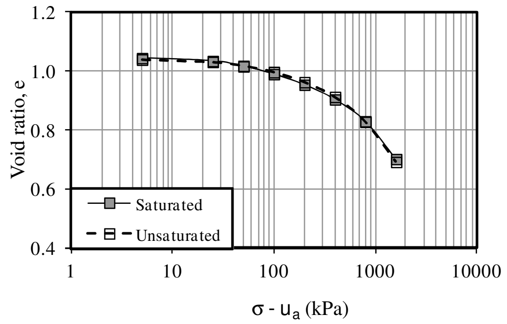
(a) w_0 = 21.5%, ρ_d = 1.342 Mg/m^3



(b) w_0 = 23.6%, ρ_d = 1.346 Mg/m^3

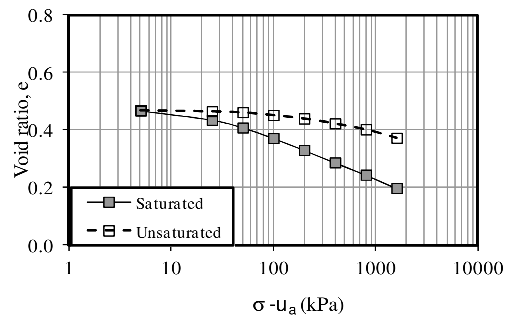


(c) w_0 = 26.5%, ρ_d = 1.346 Mg/m^3

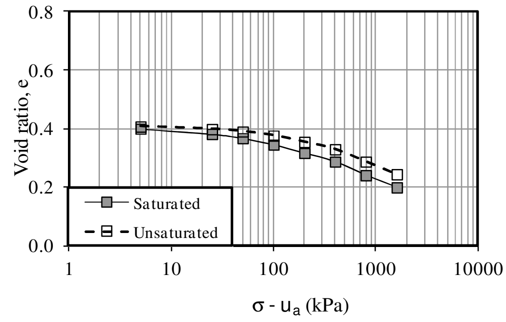


(d) w_0 = 30.1%, ρ_d = 1.275 Mg/m^3

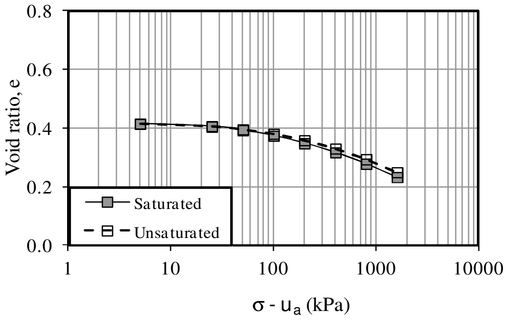
Figure 4 Double-oedometer test results for compacted kaolin



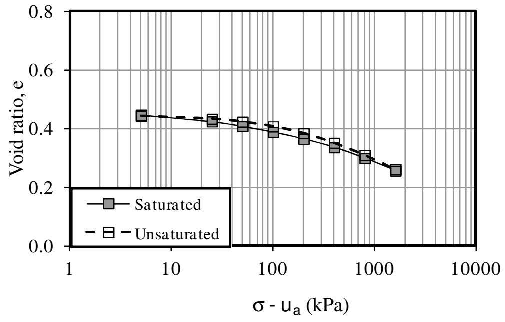
(a) w_0 = 9.4%, ρ_d = 1.776 Mg/m^3



(b) w_0 = 13.8%, ρ_d = 1.870 Mg/m^3



(c) w_0 = 15.3%, ρ_d = 1.852 Mg/m^3

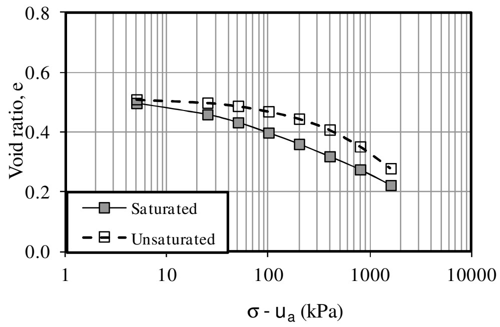


(d) w_0 = 16.7%, ρ_d = 1.813 Mg/m^3

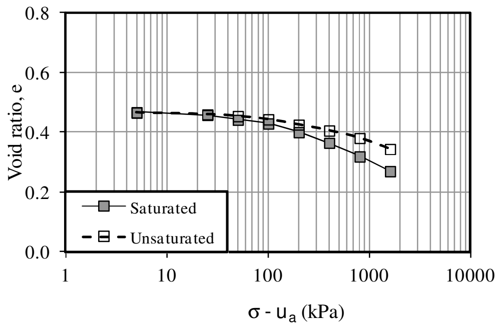
Figure 5 Double-oedometer test results for compacted JFA

Houston and Houston (1997) have suggested that collapse settlement be estimated from the saturated compression curve alone as the compression of the unsaturated soil is negligible. For compacted soils, double-oeodometer test results indicate that the unsaturated soil specimen does trace an unsaturated compression curve (e.g. Pereira and Fredlund 2000, Lim and Miller 2004).

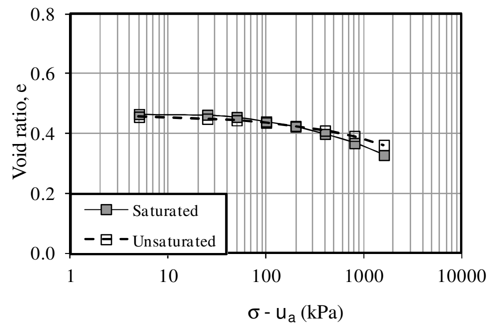
Several approaches for estimating volume change of soil under wetting conditions have been proposed. Hibibagahi and Mokhberi (1998) proposed a hyperbolic model to describe the volume change behaviour of collapsible soil using results from triaxial tests with matric suction measurements. The collapse settlement in terms of volumetric strain $(\Delta\varepsilon_v)_{collapse}$ is given as:



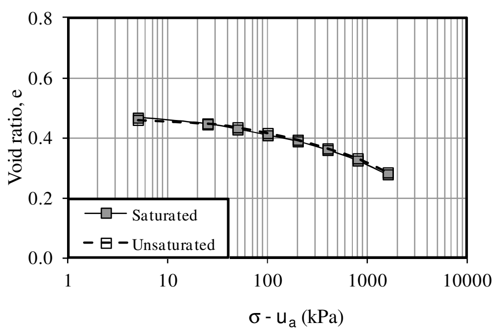
(a) $w_0 = 9.7\%$, $\rho_d = 1.789$ Mg/m^3



(b) $w_0 = 12.1\%$, $\rho_d = 1.849$ Mg/m^3



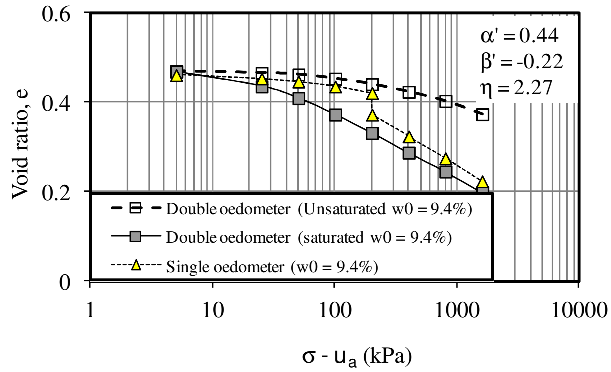
(c) $w_0 = 13.2\%$, $\rho_d = 1.859$ Mg/m^3



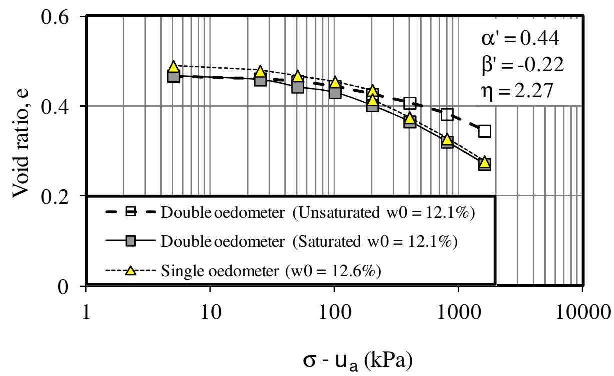
(d) $w_0 = 15.2\%$, $\rho_d = 1.849$ Mg/m^3

Figure 6 Double-oedometer test results for compacted JFB

$$(\Delta\varepsilon_v)_{collapse} = \left[\frac{K_w p_a k w^{-k-1}\sigma_m}{\left(\frac{\sigma_m}{\varepsilon_u} + K_w p_a w^{-k}\right)^2}\right]\Delta w \qquad (1)$$



(a) Residual soil JFA



(b) Residual soil JFB

Figure 7 Comparison of single and double-oedometer test results of compacted residual soils

where K_w = non-dimensional bulk modulus number; p_a = atmospheric pressure; k = initial tangent bulk modulus exponent; w = water content; σ_m = mean confining pressure; ε_u = is the asymptotic volumetric strain at large stresses. Pereira and Fredlund (2000) have suggested a three phase model to describe volume change behaviour of a collapsible soil under wetting: pre-collapse, collapse and post-collapse. The following relationship was proposed to simulate soil collapse:

$$e = e_0 + \frac{e_f - e_0}{\left\{1 + \left[\frac{(u_a - u_w)}{c}\right]^b\right\}} \qquad (2)$$

where e = void ratio; e_0 = initial void ratio; e_f = final void ratio under a given confining stress; $(u_a - u_w)$ = matric suction; u_a = pore-air pressure; u_w = pore-water pressure; b = slope parameter (i.e. slope of the collapse phase); and c = matric suction at the inflection point (i.e. middle point of collapse phase). Relationships similar to Eq. (1) were also suggested to describe the change in Poisson's ratio and degree of saturation as the soil is being wetted. Futai and Almeida (2002) extended Alonso et al. (1990) model to describe the behaviour of collapsible soil using results from suction controlled oedometer tests. In the above-mentioned models, measurement of matric suction which is still unavailable in many laboratories is necessary, and a reasonable number of tests is required to fully define the parameters.

Eqs. (1) and (2) can be easily shown to have the following forms, respectively:

$$e = e_0 + \frac{A\left(\frac{\sigma_m}{p_a}\right)}{\left[1 + B\left(\frac{\sigma_m}{p_a}\right)\right]^2} \qquad (3)$$

$$e = e_0 + \frac{A}{\left[1 + \left(\frac{u_a - u_b}{c}\right)^b\right]} \qquad (4)$$

where A and B are constants. Eqs. (3) and (4) are observed to be very similar to the general compressibility relationship proposed by Carrier and Beckham (1984) for one-dimensional compression of saturated clays:

$$e = \varepsilon + \alpha\left(\frac{\sigma'}{p_a}\right)^\beta \qquad (5)$$

where σ' is the effective vertical stress, and α, β and ε are empirical constants. For normally consolidated clays, α, β and ε are given by:

$$\alpha = 0.0208PI\left(1.192 + \frac{1}{A}\right) \qquad (6a)$$

$$\beta = -0.143 \qquad (6b)$$

$$\varepsilon = 0.027PL - 0.0133PI\left(1.192 + \frac{1}{A}\right) \qquad (6c)$$

where PI is plasticity index, A is activity (i.e. $\frac{PI}{\%clay < 2\mu m}$) and PL is plastic limit.

It is conceivable that any saturated soils under one-dimensional compression can also be modeled using Eq. (5). Equation (5) can be normalized by the initial void ratio e_o to give:

$$\frac{e}{e_o} = \frac{\varepsilon}{e_o} + \frac{\alpha}{e_o}\left(\frac{\sigma'}{p_a}\right)^\beta \qquad (7)$$

At the beginning of the compression test, σ' = 0 and e = e_o. Therefore Eq. (7) can be shown to be

$$\frac{e}{e_o} = 1 + \alpha'\left(\frac{\sigma'}{p_a}\right)^\beta \qquad (8)$$

where $\alpha' = \alpha/e_o$. Equation (8) can then be used to describe the one-dimensional compression of any saturated soils. The relationships given for the empirical constants α, β and ε in Eq. (6) are dependent on Atterberg limits, water content at the start of the compression test, the activity and the pore-water composition (Carrier and Beckman 1984). In other words, the empirical constants α, β and ε in Eq. (6) are dependent on soil type and soil structure. Using the same reasoning, the empirical constants α' and β in Eq. (8) account for soil type and soil structure as well. Therefore for each soil compacted at different water contents, α' and β will be different for each water content as the soil structure is different.

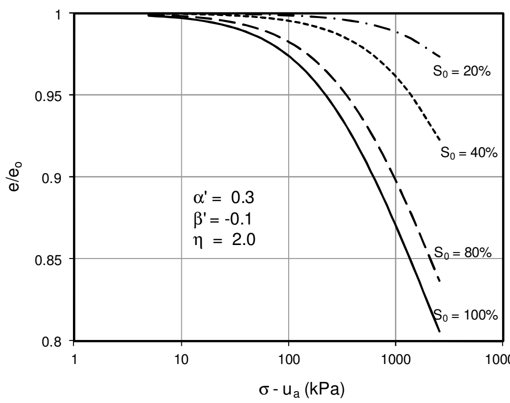
Considering Eqs. (3), (4) and (8), the following equation was found to give a better fit to the saturated compression curves of the soils in the double-oedometer tests:

$$\frac{e}{e_o} = \left[1 + \alpha'\left(\frac{\sigma'}{p_a}\right)\right]^{\beta'} \qquad (9)$$

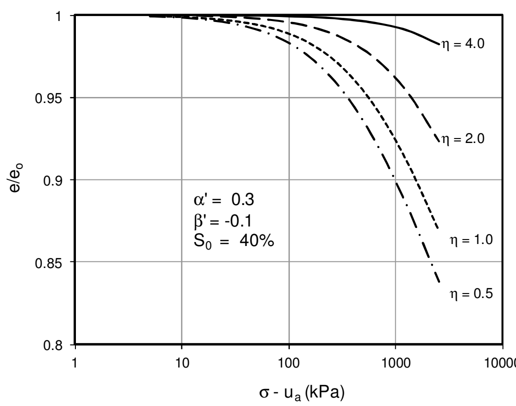
where α' and β' are empirical constants that account for soil type and soil structure. Equation (9) can be extended to describe the unsaturated compression curve by replacing σ' with the more general net normal stress ($\sigma - u_a$) and accounting for the initial degree of saturation S_0. One possible modification to Eq. (9) is:

$$\frac{e}{e_0} = \left[1 + \alpha' S_0^\eta\left(\frac{\sigma - u_a}{p_a}\right)\right]^{\beta'} \qquad (10)$$

where η is an empirical constant dependent on soil type and loading condition only. For the case of the saturated compression curve, S_0 = 1 and Eq. (10) reverts back to Eq. (9). Therefore, Eq. (10) is a general equation for describing one-dimensional compression of soils at any initial degree of saturation as illustrated in Figure 8 for α' = 0.3, β' = -0.1 and η = 2.0. The compression curve of soils at different degrees of saturation can be derived from the saturated compression curve if η is known. It is implicitly assumed in Figure 8a that e_0 for the saturated compression curve refers to the initial void ratio after water inundation and S_0 is calculated based on the larger e_0 value of the unsaturated and saturated compression curve. The effect of η on the compression curve is illustrated in Figure 8b for α' = 0.3, β' = -0.1 and S_0 = 40% . As the value of η increases, the compression curve becomes flatter. The wetting-induced settlement can be estimated from the difference in ordinates between the unsaturated compression curve and the saturated compression curves at any applied net normal stress.



(a) Effect of initial degree of saturation, S_0



(b) Effect of η

Figure 8 Compression curves from Equation 10

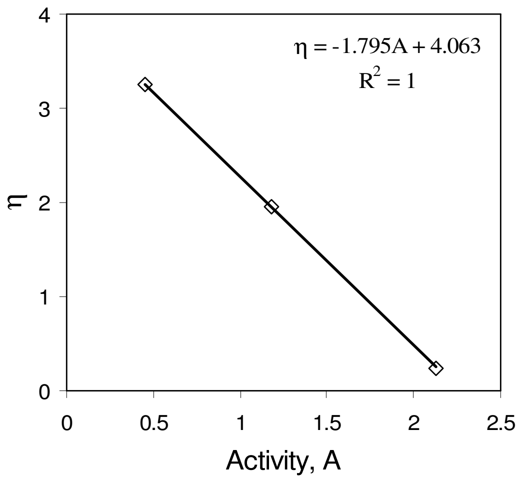


Figure 9 Relationship of η with activity A

Table 2 Summary of empirical constants

Soil type	Compaction water content, w_0 (%)	From saturated compression curve		From unsaturated compression curve
		α'	β'	η
Kaolin	21.5	0.05	-1.0	0.24
	23.6	0.16	-0.29	
	26.5	0.12	-0.36	
	28.5	0.17	-0.33	
	30.1	0.14	-0.33	
JFA	9.4	1.43	-0.27	3.25
	11.4	0.27	-0.30	
	13.8	0.65	-0.28	
	15.3	0.42	-0.28	
	16.7	1.04	-0.19	
JFB	9.7	2.23	-0.22	1.95
	12.1	0.33	-0.29	
	13.2	0.28	-0.20	
	15.2	1.06	-0.17	
	17.5	0.72	-0.23	

The empirical constants α'and β' in Eq. (10) were obtained by curve fitting the saturated compression curves of the compacted kaolin and residual soils (i.e. S_0 = 1). The values of α'and β' are summarized in Table 2. To obtain the value of η, the α'and β' of the corresponding saturated compression curve were used and η was obtained by best fit for all the unsaturated compression curves of each soil as the value of η is a function of soil type and loading condition only. The η values for kaolin, residual soils A and B are also shown in Table 2. By plotting η against basic soil properties such as Atterberg limits, fines content, clay content and activity, the following linear relationship was found between η and activity A (Figure 9):

$$\eta = -1.795A + 4.063 \qquad (11)$$

The validity of Eq. (11) can be verified using double-oedometer test data of other soils. Several data sets are available in the literature for verification. These are described in the following section.

5. VERIFICATION OF EQUATION (11)

Double-oedometer test data on compacted soils from Lawton et al. (1989), Vilar (1994), Fredlund and Gan (1995), Alawaji (1997), and Lim and Miller (2004) were used to verify the applicability of Eq. (10). The properties of the soils are summarized in Table 3. The values of α'and β' were derived from the saturated compression curve and the value of η was obtained from Eq. (11). The estimations of the unsaturated compression curves are shown in Figures 10 to 14. Good agreement was obtained between the estimated unsaturated compression curves (shown as dashed lines) and the experimental unsaturated compression curves (shown as open square symbols) for all the soils up to $(\sigma - u_a)$ = 1000 kPa. Beyond $(\sigma - u_a)$ = 1000 kPa, the unsaturated soil specimen showed a tendency to approach the saturated compression curve as the unsaturated soil specimen approach the saturated condition.

Table 3 Summary of soil properties for verification of Equation (11)

Reference	Soil type	Soil Properties	
Lawton et al. (1989)	Clayey Sand	Liquid Limit	34
		Plasticity Index	15
		% clay	15
		Activity	1.0
		USCS	SC
Vilar (1994)	Clayey Soil	Liquid Limit	71
		Plasticity Index	35
		% clay	34
		Activity	1.03
		USCS	SC
Fredlund and Gan (1995)	Indian Head Silt	Liquid Limit	22.2
		Plasticity Index	5.6
		% clay	6
		Activity	0.93
		USCS	SM
Alawaji (1997)	Al Helwah Alluvial Soil	Liquid Limit	18
		Plasticity Index	17
		% clay	11
		Activity	1.55
		USCS	SM
Lim and Miller (2004)	Minco Silt	Liquid Limit	28
		Plasticity Index	8
		% clay	15
		Activity	0.53
		USCS	CL
	Blaine Shale	Liquid Limit	28
		Plasticity Index	10
		% clay	32
		Activity	0.31
		USCS	CL
	Hennessey Shale 1	Liquid Limit	34
		Plasticity Index	13
		% clay	51
		Activity	0.25
		USCS	CL
	Boggy Shale	Liquid Limit	45
		Plasticity Index	24
		% clay	48
		Activity	0.50
		USCS	CL

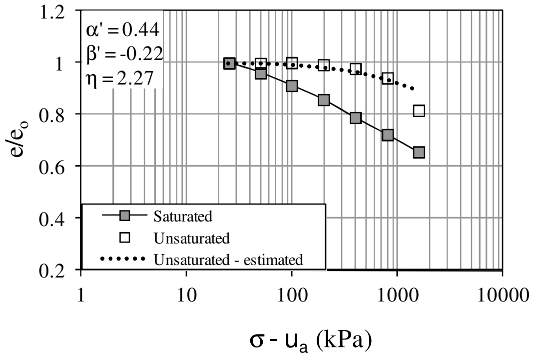
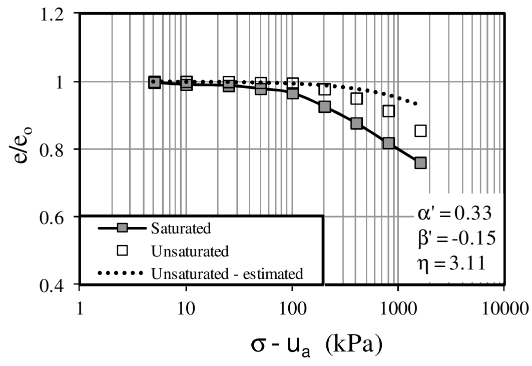


Figure 10 Estimation of unsaturated compression curve for Lawton et al. (1989) data



(a) Minco Silt

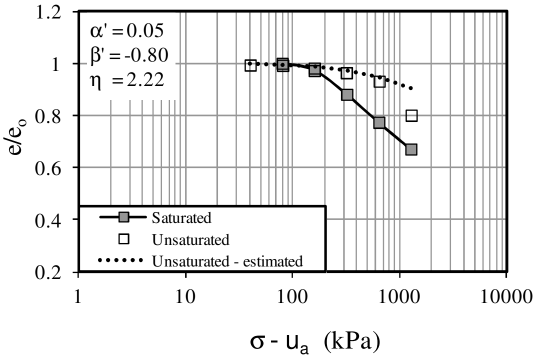
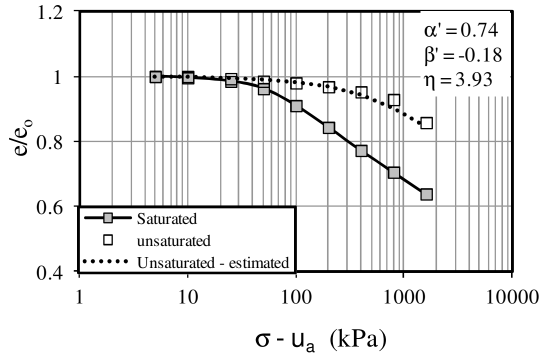


Figure 11 Estimation of unsaturated compression curve for Vilar (1994) data



(b) Blaine Shale

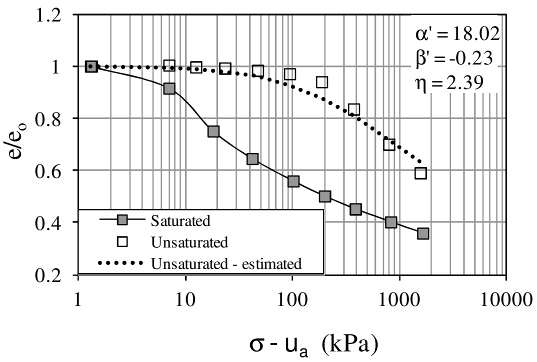
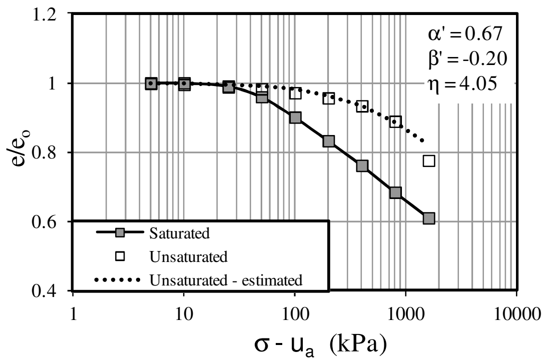


Figure 12 Estimation of unsaturated compression curve for Fredlund and Gan (1995) data



(c) Hennessey Shale 1

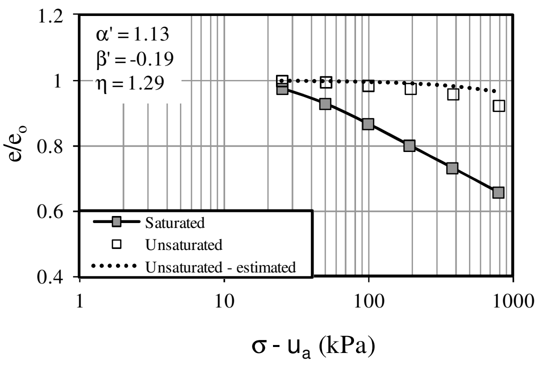
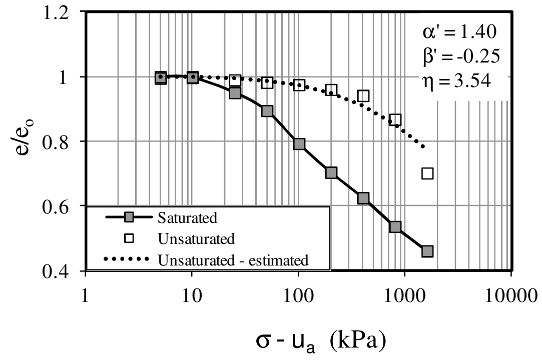


Figure 13 Estimation of unsaturated compression curve for Alawaji (1997) data



(d) Boggy Shale

Figure 14 Estimation of unsaturated compression curves for Lim and Miller (2004) data

6. CONCLUSION

Double-oedometer tests were conducted on compacted soils of three soil types. The unsaturated compression curve with normalized e lies above the corresponding saturated compression curve. A compacted soil specimen on the unsaturated compression curve collapsed onto the saturated compression curve on wetting. Therefore, the wetting-induced settlement of a compacted soil at any net normal stress can be estimated from the difference in ordinates between the unsaturated and saturated compression curves at that net normal stress. The results of the double oedometer tests were used to develop a compression equation for unsaturated compacted soils. The compression equation has three empirical constants: α', β' and η. The empirical constants α' and β' are dependent on soil type and soil structure, and can be obtained from the saturated compression curve. The empirical constant η is a linear function of activity. The compression equation when applied to double oedometer test data of compacted soils from the literature showed that it could replicate the unsaturated compression curve of compacted soils up to a net normal stress of 1000 kPa, and therefore it can be used to estimate the wetting-induced settlement of compacted soils from a single inundated oedometer test.

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