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Behaviour of Clay Subjecting to Vacuum and Surcharge Loading in an Oedometer

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ABSTRACT: The deformation of clayey soil subjected to the combination of a vacuum pressure and a surcharge load in an oedometer apparatus, with either vertical or inward radial drainage, has been investigated experimentally. A parameter K_w, defined as the horizontal effective stress exerted on the wall of the consolidation ring (σ'_{hw}) divided by the vertical effective stress (σ'_v) in the soil specimen is suggested as an indicator of the tendency for lateral deformation under field conditions. It is observed that if the value of K_w is close to the value of K_0 of the soil (the 'at-rest' earth pressure coefficient), there should be very limited lateral displacement in field situations. The laboratory test results show that during the loading process, the value of K_w is mainly influenced by the ratio of the magnitude of the surcharge pressure to the vacuum pressure (*RL*) and the rate of surcharge loading (*SLR*). At the end of consolidation the value of K_w increases with increasing *RL*, but it is almost independent of *SLR* for the conditions investigated. In the presence of a vacuum pressure, at the end of consolidation the value of K_w is usually less than the original value of K_0 of the soil. The test results also indicate that K_w has a strong correlation with the synthetic non-dimensional loading parameter, *RLS*, as defined by Chai et al. (2013), which includes the effects of both *RL* and *SLR* as well as the consolidation properties of the soil. It is suggested that the parameter *RLS* can be used to predict lateral displacements under field conditions that involve combined surcharge and vacuum pressure loading.

Keywords: Vacuum consolidation, combined loading, oedometer, earth pressure, deformation.

1. INTRODUCTION

Preloading a soft clayey deposit, either by application of a surcharge load (e.g., embankment fill) or vacuum pressure, or some combination of both, has been used to improve the strength and stiffness of soft ground (e.g., Bergado et al. 1998; Yan and Chu 2005; Chai et al. 2006; Rujikiatkamjorn et al. 2007; Kelly and Wong 2009; Hirata et al. 2010; Indraratna et al. 2011). There are several advantages in using a combination of vacuum pressure and surcharge load, including the following:

(a) The combination of these loads effectively increases the preloading pressure and ultimately results in over-consolidated ground. The maximum achievable vacuum pressure in the field is about 60 to 80 kPa (e.g., Bergado et al. 1998; Tang and Shang 2000), which is approximately equivalent to the vertical pressure that is applied by 3 to 4 m of earth fill. After releasing the vacuum pressure, the ground will be in an over-consolidated state, which will tend to reduce the settlements that might be induced by subsequent construction activities.

(b) The application of a vacuum pressure tends to reduce outward lateral displacement of the ground. A vacuum pressure is an isotropic consolidation pressure increment and it tends to induce inward lateral displacement (toward the centre of a treated area), while an embankment load will generally cause outward lateral displacement of a deposit. Conceptually at least, the combination of a vacuum pressure and a surcharge load can reduce the lateral displacement of the ground to values smaller than might be expected for a surcharge pressure of the same overall magnitude applied alone.

It is important to note that in engineering practice, particularly in urban areas, the lateral displacement of the ground may in fact control the design of the ground treatment and improvement works. The possibility that preloading a soft soil deposit with a combination of vacuum pressure and surcharge load can limit the lateral displacement of the deposit is thus highly significant.

Although a vacuum pressure and a surcharge load can induce lateral displacements of the ground potentially in opposite directions, the mechanisms and the characteristics of these displacements are different. The surcharge loading not only imposes a consolidation pressure to a soft clayey deposit but it will also induce increments of shear stress in the soil. These shear stresses will induce immediate outward lateral displacement of the ground, and normally, the maximum value of this outward movement will occur below a relatively stiff surface crust. On the other hand, inward lateral displacement induced by a vacuum will occur during the consolidation process and will generally exhibit a maximum value at the ground surface. Consequently, under the combination of a vacuum pressure and a surcharge load, the resulting lateral displacement of the ground is not only a function of the ratio of the magnitude of surcharge load to vacuum pressure (*RL*), but also a function of loading sequence (Ong and Chai 2011). In practice, normally the design value of the vacuum pressure can be applied and will become effective in a relatively short time, while the application of an embankment load generally requires a relatively longer application period. Therefore, in principle at least, it is possible by adjusting the value of *RL* as well as the rate of application of the surcharge loading (*SLR*) to control and minimize the lateral displacement of the ground. To use this technique in practice, a quantitative understanding of the effects of both *RL* and *SLR* on lateral displacement is essential, but there is limited research on this topic reported in the literature (e.g., Chai et al. 2013).

The objective of this laboratory study is therefore to investigate the effects of the parameters *RL* and *SLR* on the deformation behaviour of clayey soil samples when subjected to the combination of a vacuum pressure and a surcharge load in an oedometer device. In engineering practice, vacuum consolidation in the field is almost always conducted for ground improved by the prior installation of prefabricated vertical drains (PVDs), and in this case horizontal drainage toward the PVDs may be the dominant drainage mechanism. Considering this possibility, laboratory oedometer tests with either vertical or radial (horizontal) drainage were conducted. The tests with vertical drainage were conducted to simulate the situation near the ground surface where the vacuum pressure is applied. Test involving inward radial (horizontal) drainage were conducted to simulate the likely drainage behaviour at greater depth in the soil deposit. In the following sections a parameter with the potential to characterise the tendency for lateral displacement of the soil sample is first introduced, and then the laboratory test apparatus and procedures, the test results and discussion of those results are presented.

2. EARTH PRESSURE COEFFICIENT AND LATERAL DISPLACEMENT

If the lateral displacement of the ground resulting from a combination of a vacuum pressure and a surcharge loading (e.g., the weight of embankment fill) is outward (Figure 1(a)), then typically soil element A will tend to develop active earth pressure conditions, while element B will tend to develop passive earth pressure. If the resulting lateral displacement is inward (Figure 1(b)), the stress condition of element A will tend towards an isotropic stress state but element B will tend to be in an active condition.

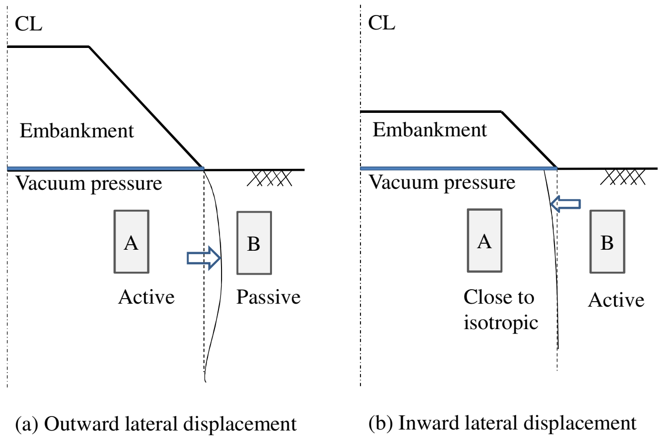


Figure 1 Lateral displacement mode and the stress conditions of typical soil elements

In the oedometer apparatus, outward lateral displacement is restricted by the specimen containment ring and an active mode, like that shown for element A in Figure 1(a), cannot be mobilized. However, inward lateral displacement can occur under vacuum pressure loading in the oedometer device and a condition close to the isotropic stress state can be developed in the soil sample, like element A in Figure 1(b). In addition, for a soil specimen deforming under oedometer conditions due to the application of a surcharge load, the earth pressure coefficient, $K = \sigma'_h/\sigma'_v$, where σ'_h and σ'_v are the effective stresses in the horizontal and the vertical directions, respectively, will vary during the consolidation process. Only after the consolidation is finished, will K attain the value of the at-rest earth pressure coefficient (K_0). Normally, during the consolidation process the value of K is larger than K_0. For example, when consolidating a clay sample from a slurry state, the initial value of K is close to 1.0 and it will gradually reduce to a constant value, K_0. Therefore, when considering the effective stress state of a soil specimen in an oedometer consolidation cell, both a surcharge load and a vacuum pressure can induce values of $K > K_0$ during the consolidation process. However, when considering the stress state of a soil element in the ground (like element A in Figure 1), the surcharge load tends to induce outward lateral displacement, while the vacuum pressure tends to induce inward lateral displacement.

Consider the stress acting on the wall of an oedometer consolidation ring, and designate the suction (vacuum) pressure as a negative stress change acting on the wall (Figure 2(b)), and the pressure change transmitted by the solid skeleton of the soil specimen as a positive component of effective stress (Figure 2(a)). If the positive stress on the wall is larger than $K_0\sigma'_v$, there will be a tendency for outward lateral displacement, and if the stress is less than $K_0\sigma'_v$, the tendency will be for inward lateral displacement. With this consideration in mind, we define a parameter known as the coefficient of earth pressure on the wall (K_w), as follows:

$$K_w = \frac{\sigma'_{hw}}{\sigma'_v} \tag{1}$$

where σ'_v = the vertical effective stress in the soil specimen, and σ'_{hw} = the horizontal effective stress on the wall of the consolidation ring. The value of σ'_{hw} is calculated as follows:

$$\sigma'_{hw} = \begin{cases} \sigma_{ep} - u & (u > 0) \\ \sigma_{ep} & (u \le 0) \end{cases} \tag{2}$$

where σ_{ep} = the measured total horizontal earth pressure, and u = the excess pore water pressure at the location of the earth pressure gauge. From Eq. (2) it can be seen that σ'_{hw} is the effective confinement of the consolidation ring acting on the soil specimen, and when a suction (vacuum pressure) alone acts on the wall, σ'_{hw} is negative. Thus, when $K_w > K_0$, there will be a tendency for outward lateral displacement, and when $K_w < K_0$, the tendency will be for inward lateral displacement. If $K_w < 0$, there is likely to be a micro-gap formed between the soil specimen and the consolidation ring. Further, under the condition of combined vacuum pressure and a surcharge load, if a loading scheme can result in a value of K_w close to K_0 during the consolidation process, then that same loading scheme will induce very limited lateral displacement. Therefore, the results of the laboratory tests conducted under odometer conditions will be analyzed using the parameter K_w to investigate the tendency for inward or outward lateral displacement of the specimen.

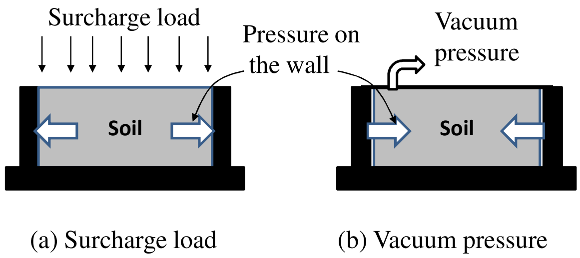


Figure 2 Illustration of the pressure change on the wall of a consolidation ring

3. OEDOMETER TESTS

3.1 Test equipment

The equipment used in the laboratory study was a Maruto Multiple Oedometer Apparatus (manufactured in Tokyo, Japan). The equipment has 5 consolidation cells, which can be used either as individual consolidation cells or connected together to form a multi-layer system. In this study only single layer tests were conducted. Each specimen was 60 mm in diameter and typically about 20 mm in thickness. The original device could only be used to conduct tests with vertical drainage, and so a new consolidation cell that allows for radial drainage was manufactured and fitted into the consolidation chamber. Figures 3(a) and (b) show the set-up of the tests with vertical and radial drainage, respectively. A picture of the device in the laboratory is shown in Figure 4. During testing, measurements were taken of the settlement, the excess pore water pressure at the bottom of the sample (vertical drainage tests) or the middle height of the consolidation ring (radial drainage tests), and the horizontal earth pressure at the middle height of the consolidation ring.

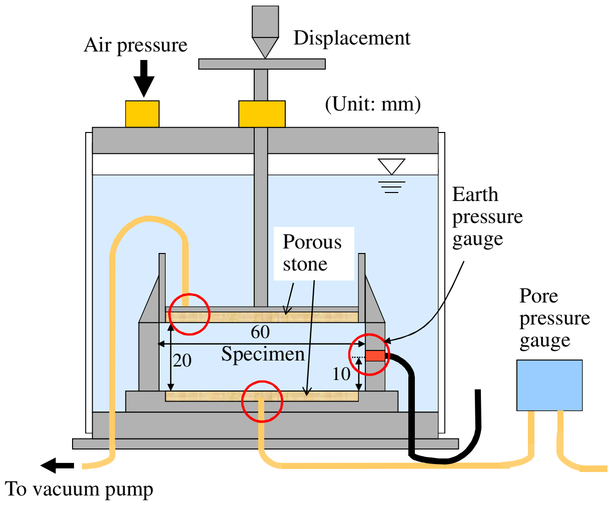
3.2 Test procedure

3.2.1 Pre-consolidation

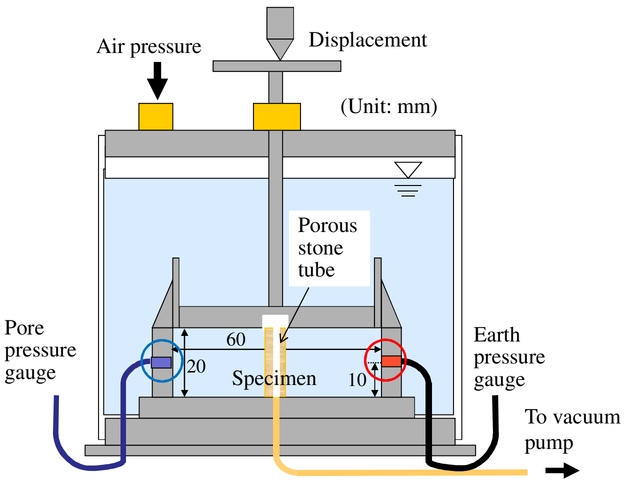
Firstly, remoulded clayey soil in a slurry state, with a water content higher than its liquid limit, was de-aired under a vacuum pressure of approximately 100 kPa. Then the de-aired slurry was put into a mould with a diameter of 60 mm and a height of 60 mm (20 mm high consolidation ring plus a 40 mm collar), and pre-consolidated under a surcharge pressure of 20 kPa.

3.2.2 Set-up of the consolidation tests

The pre-consolidated soil sample was cut to 20 mm thickness for further consolidation testing. For the tests with vertical drainage, the consolidation ring with the pre-consolidated soil specimen inside was directly set into the apparatus for the consolidation test, and all the tests were conducted under one-way drainage conditions. For the tests involving radial drainage, a hole 8 mm in diameter was formed at the centre of the specimen to allow installation of an annular porous stone, which acted as the central drain. The hole was made by a cylindrical device, which is shown in Figure 5. The inner diameter of the cylindrical device is the same as the outer diameter of the consolidation ring, and the tube at the centre of the device has an outer diameter of 8 mm and height of 12 mm. The device was set in such a way that the consolidation ring with the soil specimen inside it could be placed in the cylindrical device, and then the cylindrical device was pushed into the soil specimen. When the device was extracted soil in the central tube of the device was also extracted, forming a central hole at least half-way through the sample. The consolidation ring with the specimen inside it was then turned over in order to make the hole in the remaining half of the soil specimen, using the same procedure.



(a) Vertical drainage test



(b) Radial drainage test

Figure 3 Sketch of the set-up of the tests

Figure 4 View of the consolidation cell

Figure 5 Photograph of the cylindrical device

3.2.3 Loading procedure and test duration

To investigate the effect of the initial vertical effective stress (σ'_{v0}) in the specimen on the behaviour of the specimen during subsequent consolidation involving application of a vacuum pressure, each specimen was first consolidated under a predetermined surcharge load in the range: $\sigma'_{v0} = 0 \sim 120$ kPa. An incremental consolidation pressure was then applied by combining a surcharge load ($\Delta\sigma_s$) and a vacuum pressure ($\Delta\sigma_{vac}$). For all the tests, the vacuum pressure was applied instantaneously, but the surcharge load was applied either instantaneously or in a series of discrete steps. In the latter case, a surcharge increment of Δp (typically 10 to 20 kPa) was applied at increments of time Δt (typically every 10 to 30 min) until the target surcharge pressure was reached.

3.3 Cases tested and the soils used

Two series of tests were conducted. Series-1 (V-series) was conducted under conditions of vertical drainage only (Figure 3(a)) and Series-2 (R-series) was under conditions of radial drainage only (Figure 3(b)). All the cases tested with the combination of a vacuum pressure and a surcharge load are listed in Table 1. The test duration was 48 hours for each test.

The two series of tests were conducted at different times and different soil samples were used. Two types of remoulded Ariake clay were sampled from about 2 m depth below the ground surface at two different locations in the Saga Plain (due to the varying accessibility of the sites). Some of the physical and mechanical properties of these samples are given in Table 2. In the table, W_L and W_p are the liquid limit and the plasticity limit, respectively, C_c is compression index and ϕ' is the friction angle of the soil. The value of ϕ' for Clay-1 is from the results of consolidated undrained triaxial compression tests with pore water pressure measurement.

4. TEST RESULTS

4.1 Tests with vertical drainage

4.1.1 Measured settlements and excess pore water pressures

Before presenting the measured values of K_w, some of the measured settlement curves and excess pore water pressure variations are given in Figures 6 and 7. Figure 6 indicates finite settlement at early time. During these tests, the surcharge load was first applied under undrained conditions. When the measured excess pore water pressure at the bottom of the specimen reached more than 90% of the applied surcharge load, the vacuum pressure was applied and the drainage valve was opened. There was some time lag in the response of the excess pore water pressure gauge at the bottom of the soil specimen, and during the waiting time (about 2 hours), some settlement (mostly elastic deformation) occurred. In Figure 7, it can be seen that the measured initial excess pore water pressure response is a little less than the applied surcharge load, and the final value is less than the applied vacuum pressure. Based on the results in Figures 6 and 7, it was concluded that the test results are consistent and reliable.

Table 1 Cases tested under combined loading

Test series	Test No	σ'_{vo} (kPa)	$\Delta\sigma_s$ (kPa)	$\Delta\sigma_{vac}$ (kPa)	Loading method for $\Delta\sigma_v$: Δt (min.)	Δp (kPa)
V-series	V-a1	0	40	80	Instantaneous	
	V-a2	0	80	80		
	V-a3	0	120	80		
	V-a4	40	80	80		
	V-a5	80	80	80		
	V-b1	0	80	80	30	10
	V-b2	0	80	80	30	20
	V-b3	0	80	80	10	10
	V-b4	0	80	80	15*	10
	V-b5	0	100	80	15*	10
	V-b6	0	120	80	15*	10
R-series	R-1	0	80	80	Instantaneous	
	R-2	0	80	80	30	10
	R-3	0	80	80	10	10
	R-4	0	120	80	10	10
	R-5	0	200	80	10	10
	R-6	0	80	80	90	10

*10 min after the application of the vacuum pressure, the surcharge load increments commenced.

Table 2 Basic soil properties

Soil	Grain size distribution (%): Clay (<5μm)	Silt	Sand	W_L (%)	W_p (%)	C_c	ϕ' (°)	Remark
Clay-1	60.5	38.3	1.2	120.5	60.3	0.75	36.6	V-series
Clay-2	71.0	25.0	4.0	120.3	56.8		-	R-series

4.1.2 Effect of initial vertical effective stress ((σ'_{v0}) on K_w

Chai et al. (2009) reported that under a vacuum pressure alone, at the end of consolidation the value of K_w is a function of the initial consolidation pressure, σ'_{v0}, in the soil samples.Test V-a2, 4 and 5 in Table 1 were designed to investigate the effect of σ'_{v0} on K_w under the combination of a vacuum pressure and a surcharge load. The variation of K_w with time for different values of σ'_{v0} is shown in Figure 8. Indeed, it can be seen that K_w is strongly influenced by the magnitude of σ'_{v0}. During the early stages of consolidation, the smaller the value of σ'_{v0}, the larger is the value of K_w. However, ultimately this trend reverses with time until finally the smaller the value σ'_{v0}, the smaller is the value of K_w, and it may even become negative. This implies that with a smaller value of σ'_{v0} and fixed surcharge load and vacuum pressure increments, there is a tendency for outward lateral displacement induced by the surcharge load during the early stages of consolidation, while at later times there is a tendency for inward lateral displacement of the specimen induced by the vacuum pressure.

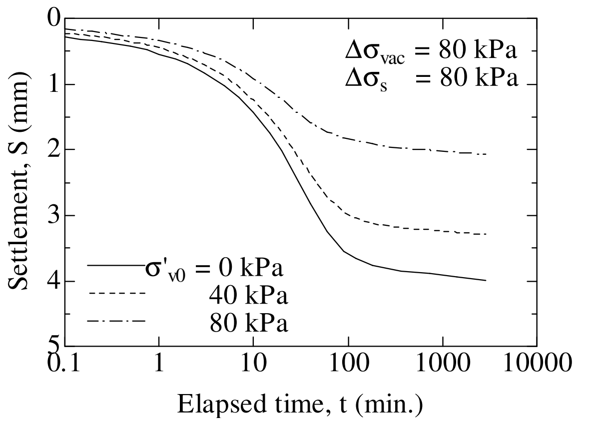


Figure 6 Measured settlement curves for different values of σ'_{v0}

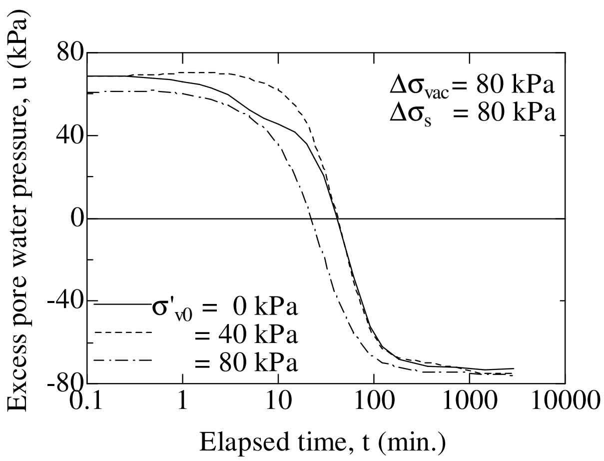


Figure 7 Measured variations of excess pore water pressure

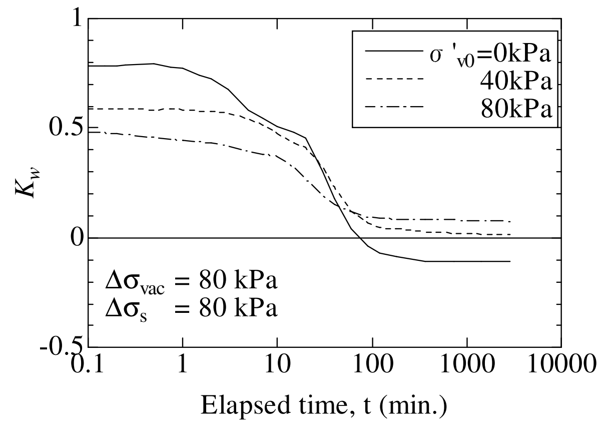


Figure 8 Variation of K_w for different values of σ'_{v0}

4.1.3 Effect of *RL* on K_w

Tests V-a1 to 3 were aimed at studying the effect of *RL* (the ratio of the magnitude of surcharge pressure to the vacuum pressure) on K_w and the results are compared in Figure 9. It can be clearly seen that the higher the value of *RL*, the large is the value of K_w at any time. Using an empirical equation for K_0, i.e., K_0 = 1-sinϕ' (Mayne and Kulhawy 1982), a value of K_0 of about 0.4 can be estimated (ϕ' =36.6° - see Table 2). When RL = 0.5, the time period for $K_w > K_0$ is about 4 min, while for RL = 1.5, the time period for $K_w > K_0$ is about 35 min. The larger the value of *RL*, the longer is the period during which there is a tendency for outward lateral displacement in the soil specimen and the higher the value of K_w at the end of consolidation.

Figure 10 depicts the values of K_w recorded in tests V-b4 to 6. For these tests, the surcharge loading rate (*SLR*) was the same in each case but the values of *RL* were different. Before $\Delta\sigma_s$ was increased to 80 kPa, the values of K_w for V-b4 to 6 were almost the same, but at the end of consolidation the values of K_w are different, and again the larger the value of *RL*, the larger the value of K_w.

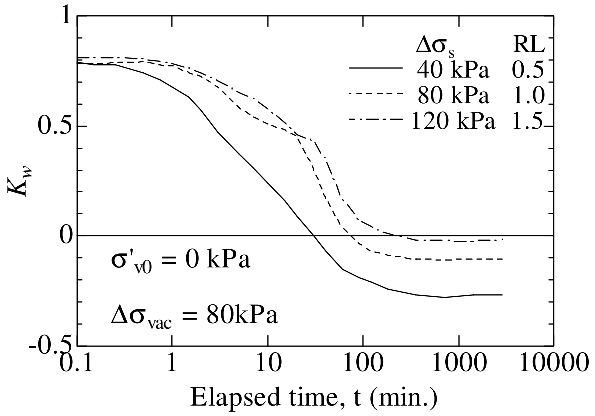


Figure 9 Variation of K_w under different *RL* value

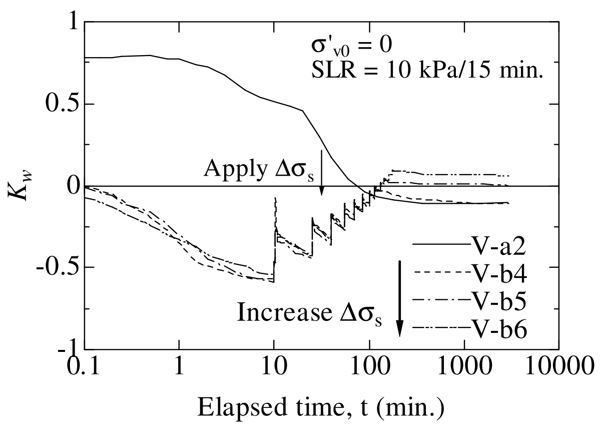


Figure 10 Variation of K_w with *RL*

4.1.4 Effect of *SLR* on K_w

Tests V-b1 to 3 were conducted with *RL* = 1.0, but the value of *SLR* was varied from 10 kPa/30 min to 10 kPa/10 min. The variation of the measured values of K_w is depicted in Figure 11. In this figure the instantaneous loading case (V-a2) is also included for comparison. Although initially the value of K_w for test V-b2 (which had a lower value of *SLR*) was larger than observed for V-b3 (which had a higher *SLR* value), after about 10 minutes the value of K_w increased in each case, with greater increase for the case with the higher value of *SLR*. This implies that in engineering practice the outward lateral displacement induced by an embankment load may be reduced by controlling the value of *SLR*.

4.1.5 Relationship between final values K_w and *RL* at the end of consolidation

The relationship between the final values of K_w and *RL* is plotted in Figure 12. It can be seen that although there is some scatter, generally K_w increases almost linearly with an increase in *RL*. As explained previously, the outward lateral displacement induced by an embankment load is due mainly to the shear deformation of the ground and this occurs during the loading process. Therefore, not only does the value of K_w at the end of consolidation have to be considered, but also its value during the consolidation process, when attempting to control the lateral displacement of the ground.

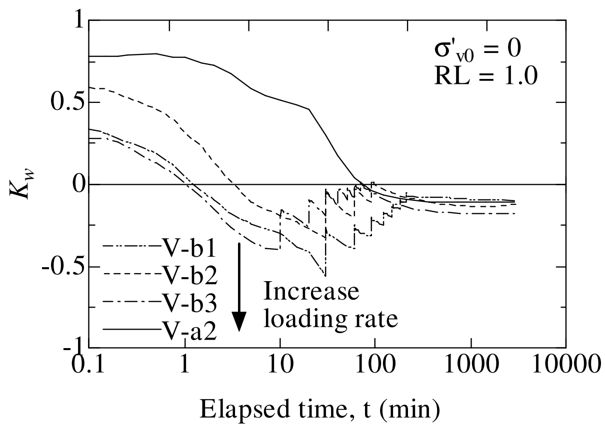


Figure 11 Variation of K_w with *SLR*

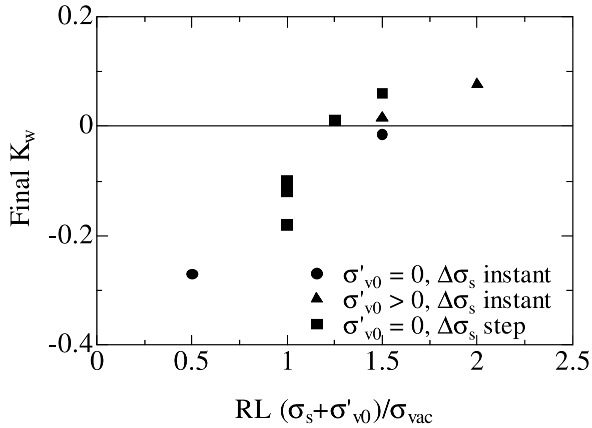


Figure 12 Relationship between values of K_w and *RL* at the end of consolidation

4.2 Tests with radial drainage

4.2.1 Behaviour in radial drainage tests

Before conducting the test with the combination of a vacuum pressure and a surcharge load, some tests with only a surcharge load or a vacuum pressure alone were conducted. In both cases the magnitude of the applied loading was 80 kPa. The measured settlement time curves are shown in Figure 13. It can be observed that the vacuum pressure resulted in a smaller final settlement but the trend is consistent with the test results for the case of vertical drainage (Chai et al. 2009).

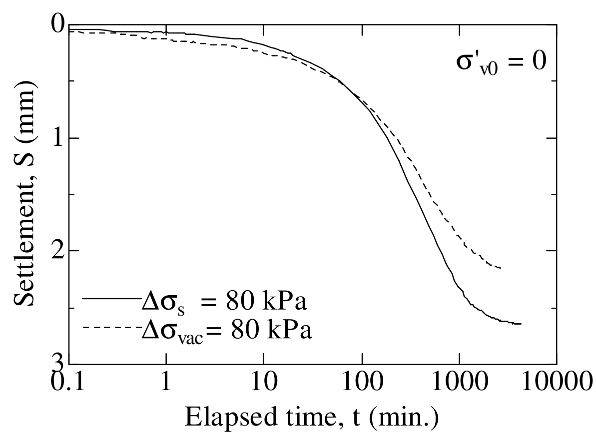


Figure 13 Settlement – time curves in radial drainage tests

The measured variations of excess pore water pressure (u) are depicted in Figure 14. Under the surcharge load, the measured initial value of u at the periphery of the specimen was about 73 kPa, which is more than 90% of the applied pressure. For the vacuum pressure case, the measured minimum value of u was about -50 kPa, which is about 63% of the applied vacuum pressure. These tests were repeated several times and the results were similar for each form of loading. It is suspected that under a vacuum pressure, the degree of saturation of the soil around the central drain might be reduced and this would have hindered the effective outward radial propagation of the vacuum pressure, toward the periphery of the specimen. The mechanism for reducing the degree of saturation may have been the expansion of trapped air bubbles in the specimen (although the soil sample was de-aired before pre-consolidation). In the outlet tube, by which the vacuum pressure was applied, small air bubbles were observed during these tests.

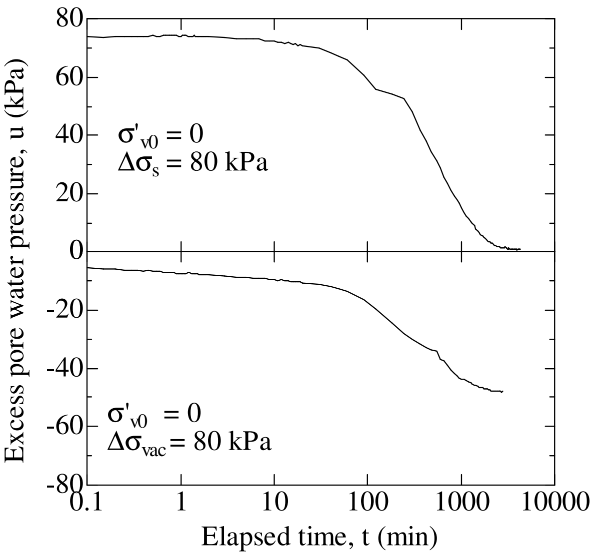


Figure 14 Variation of excess pore water pressure in radial drainage tests

In general, for a radial drainage test, the excess pore water pressure, u, and therefore the effective vertical stress will vary with radial distance. From Barron (1948)'s solution for vertical drain consolidation, ignoring any smear effect and well resistance, the value of u at a radial distance, r, and time, t, can be expressed as:

$$u(r,t) = \frac{r_e^2 \ln(r/r_w) - (r^2 - r_w^2)/2}{r_e^2 \ln(r_e/r_w) - (r_e^2 - r_w^2)/2} u_e(t) \tag{3}$$

where r_w = the radius of the central drain, r_e = the radius of the unit cell (in this case the outer radius of the specimen), and $u_e(t)$ = the value of u at the periphery of the specimen at time t. For the test conditions adopted in this study, r_w = 4 mm, and r_e = 30 mm. Using these numbers and by integration, the average excess pore water pressure in the specimen at time t will be:

$$\bar{u}(t) = 0.857 u_e(t) \tag{4}$$

In calculating the value of K_w, the average quantity $\bar{u}(t)$ was used to obtain the average vertical effective stress in the specimen. To be consistent, $\bar{u}(t)$ was also used to calculate σ'_{hw} in Eq. (2). The values of K_w calculated in this way are shown in Figure 15. At the end of consolidation the value of K_w under the surcharge load is about 0.26 and about -0.86 for the vacuum pressure case. Compared with the values reported by Chai et al. (2009) for vertical drainage tests, at the end of consolidation the values of K_w in these radial drainage tests are somewhat lower. The internal friction angle of Clay-2 (Table 2) was not measured. However, assuming it is about the same as for Clay-1, a value of K_0 of about 0.4 can be estimated. At the end of consolidation the value of K_w measured under the surcharge load is much less than 0.4. For an oedometer test with radial drainage toward the centre of the specimen, consolidation propagates from the centre to the periphery of a specimen. In order to maintain uniform vertical strain throughout the specimen, the varying degree of consolidation in the radial direction will necessarily induce a component of soil movement in the horizontal direction. At the later stages of the test, when consolidation near the centre of the specimen is almost finished, the ongoing consolidation of the soil specimen near its periphery may cause some radial contraction of the soil at that location, which may result in a reduction of the earth pressure and should therefore result in a lower measured value of K_w.

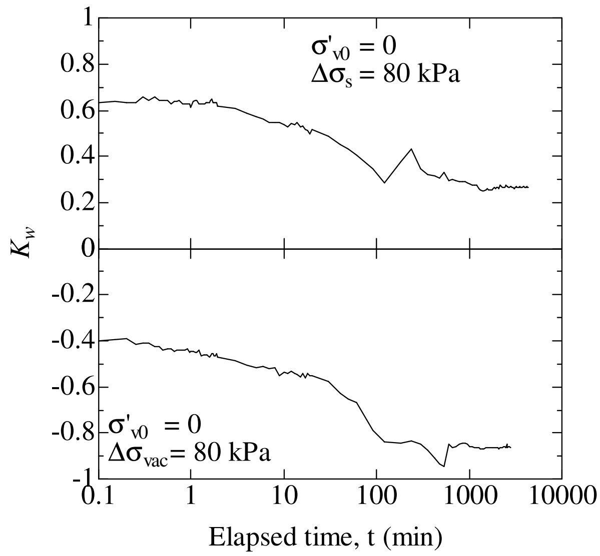


Figure 15 K_w values in radial drainage tests

4.2.1 Effect of *RL* on K_w

The results of tests with the same value of *SLR* but different values of *RL* are shown in Figure 16. Except for some divergence when the elapsed time is less than 10 min, there are no major differences in the measured values of K_w at most times. However, careful examination of the data indicates that the value of K_w was influenced slightly by *RL* during the later stages of loading, e.g., see those tests with larger values of *RL* (R-4 and R-5). As for the results of the vertical drainage tests, at the end of consolidation the value of K_w increased with an increase in *RL*. However, the absolute value of K_w observed in the radial drainage tests is generally smaller than that of the vertical drainage case. Tests R-4 and V-b6 (Table 1) have the same value of *RL*, but at the end of consolidation the value of K_w for V-b6 was positive (Figure 10), while for R-4 it was negative (Figure 16). Possible reasons for this difference have already been explained.

4.2.3 Effect of *SLR* on K_w

For the radial drainage tests, the effect of *SLR* on K_w is shown in Figure 17. As for the case of vertical drainage, increasing *SLR* caused an increase in the value of K_w observed during the loading process. However, the increment of K_w observed when a given incremental load was applied is generally larger for the radial drainage case (Figure 17) than for the vertical drainage case (Figure 11). This is possibly because the degree of consolidation of the specimen (and therefore the effective stress) at the location of the earth pressure gauge is lower for the radial drainage case compared with the vertical drainage case. The smaller the effective stress in a soil sample, the larger is the increment of horizontal earth pressure that can be induced by a given vertical load increment. As discussed previously, for a slurry with almost zero initial effective stress between the soil particles, the value of K_w will be approximately 1.0. Furthermore, when σ'_{v0} = 0, $\Delta\sigma_s = \Delta\sigma_{vac}$ = 80 kPa, during the process of load application, the most values of K_w observed in the radial drainage tests were larger than zero, and only became negative during the final stages of consolidation. This tendency is different from that shown in Figure 11 for the vertical drainage cases. This difference is probably due to the different degree of consolidation, the different drainage conditions and deformation characteristics of the specimen. The distance from the earth pressure gauge location to the drainage boundary is shorter for the case of vertical drainage and so the tendency for vacuum pressure to deform the specimen will therefore tend to occur earlier than in the radial drainage tests. At the end of consolidation the values of K_w from both types of test are comparable but they are generally smaller for the radial drainage tests.

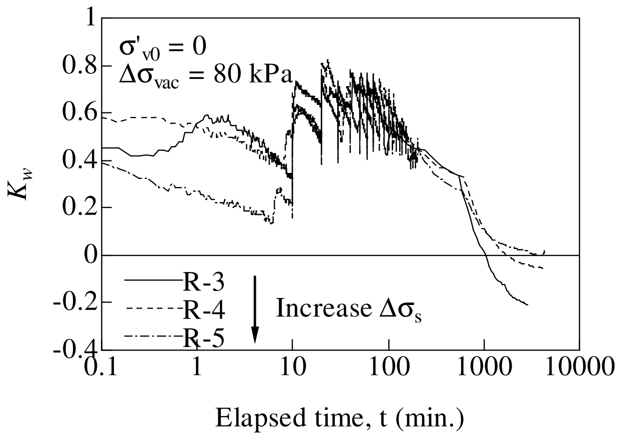


Figure 16 Effect of *RL* on K_w in radial drainage tests

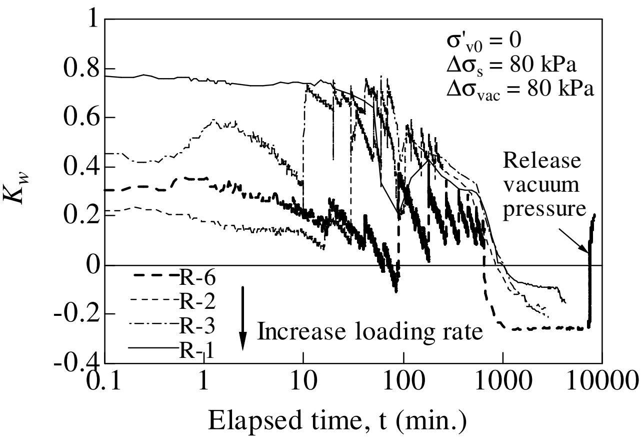


Figure 17 Effect of *SLR* on K_w in radial drainage tests

4.3 Discussion

Under the combination of a vacuum pressure and a surcharge load, the results of both the vertical and the radial drainage tests indicate that during the loading process the value of K_w increased with increases in the values of *RL* and *SLR*. It is possible to maintain the value of K_w close to K_0 by adjusting the values of *RL* and *SLR*, at least in principle if not always in practice. Since lateral deformation induced by a vacuum pressure occurs during the consolidation process, the effect of *RL* and *SLR* on the value of K_w depends on the drainage conditions of the specimen. At the location of the earth pressure gauge and at a given time, the degree of consolidation for the vertical drainage case (where the distance to the drainage boundary is about 10 mm) is generally larger than that for the radial drainage case (where the distance to the drainage boundary is about 26 mm). For given values of *RL* and *SLR*, the vertical drainage case will result in a smaller value of K_w.

Chai et al. (2013) proposed a synthetic dimensionless parameter, *RLS*, defined as the ratio of an index pressure (p_n) and a representative value of the undrained shear strength (s_u) of the soil deposit (i.e., $RLS = p_n/s_u$) in order to predict the lateral displacement of a soil deposit due to the combination of a vacuum pressure and surcharge loading. This parameter is particularly useful in cases where the soft soil is improved by the installation of prefabricated vertical drains (PVDs). In general the index pressure p_n can be expressed as follows:

$$p_n = \Delta\sigma_s - (|\Delta\sigma_{vac}| + \Delta\sigma_s) \cdot U \tag{5}$$

where $\Delta\sigma_{vac}$ = the vacuum pressure; $\Delta\sigma_s$ = surcharge pressure (usually corresponding to an embankment load for a field case); and U = the average degree of consolidation of the PVD-improved zone at the end of the embankment construction period.

It can be demonstrated that the parameter *RLS* includes both the effect of *RL* and *SLR*, as well as the consolidation properties of a soil deposit. Based on the results of 18 field cases, it has been suggested that when the value of *RLS* is in the range from about -0.6 to 0, the net maximum lateral displacement (*NLD*) of the ground at the end of embankment construction will be close to zero. The net maximum lateral displacement is defined as the maximum value of the net outward lateral displacement reduced by the maximum value of the net inward lateral displacement. Although the field data used by Chai et al. (2013) are all from PVD improved cases, the method for calculating *RLS* can be easily applied to cases without PVD installation.

Values of *RLS* were calculated for the stepwise loading cases listed in Table 1 and then plotted against the corresponding values of K_w in Figure 18. These calculations were conducted under the following assumptions.

(1) For the V-series tests, the data correspond to the end of the final step in the application of the surcharge load.
(2) For the R-series tests, the data were taken at the time when the surcharge load just reached a magnitude of 80 kPa.
(3) The results of the V-series test show that during the process of applying $\Delta\sigma_s$, generally there is a tendency for the value of K_w to increase. However, for the R-series tests, at the earlier surcharge loading steps, the value of K_w either increased or changed very little. In tests R-4 and R-5, after the surcharge load reached 80 kPa, K_w reduced with further increase in $\Delta\sigma_s$. It is considered that this behaviour might be caused by the restricted deformation permitted under laboratory conditions, as mentioned previously, and this restriction may not occur under field conditions. In addition, significant outward lateral displacement will occur when the value of K_w approaches a maximum. Considering these factors, for tests R-4 and R-5 the values of *RLS* and K_w corresponding to the situation when $\Delta\sigma_s$ just reached 80 kPa were selected for plotting in Figure 18.

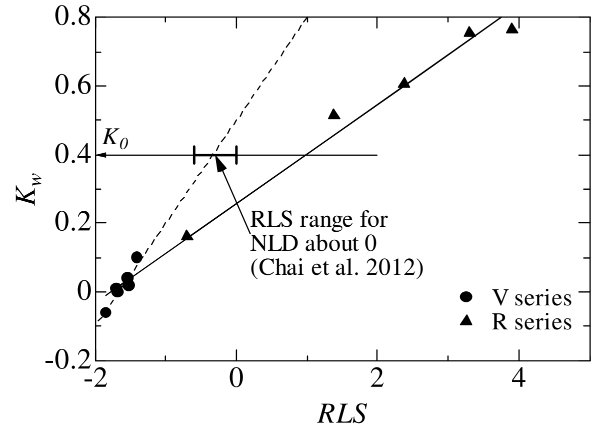


Figure 18 *RLS* versus K_w relationship

From Figure 18, it can be seen that there is a clear correlation between the values of *RLS* and K_w, although the form of this relationship may be dependent on the drainage conditions adopted in the laboratory tests. In particular, it is noted that the value of *RLS* corresponding to the condition $K_w = K_0$ (about 0.4) appears to be about 1.0, which is much larger than the value proposed by Chai et al. (2013) based on field data. However, considering the observation that the R-series tests generally resulted in lower values of K_w, and extrapolating the relationship between *RLS* and K_w based only on the results of the V-series tests, the same range proposed by Chai et al. (2013) for *RLS* is intersected when $K_w = K_0$. Therefore, the results presented in this study at least partially support the proposal of using the value of *RLS* to predict the lateral displacement of a soft soil deposit subject to a combination of vacuum and surcharge pressure.

Regarding the value of K_w at the end of consolidation under the conditions imposed in the oedometer apparatus, and in the presence of a vacuum pressure, the value of K_w is less than the value of K_0 for the soil because a vacuum pressure is an isotropic consolidation stress increment, and for the conditions investigated at the end of consolidation the value of K_w is only influenced by the value of *RL*. However, a loading condition resulting in a value of K_w less than K_0 at the end of consolidation does not ensure that the final lateral displacement will be inward. This is because the stress-strain behaviour of a clayey soil is elastoplastic and not elastic, and of course plastic deformation depends on the loading path.

5. CONCLUSIONS

The deformation of a clayey soil loaded in an oedometer apparatus by the combination of a vacuum pressure and a surcharge load has been investigated experimentally. Tests with either vertical or radial drainage were conducted, but there were no tests with both. In those tests that involved only radial drainage, the radial component of the pore water flow was toward the centre of the specimen. Those tests that involved only vertical drainage were conducted to simulate situations in the field near the ground surface, where vertical flow of pore water is likely to be the dominant form of drainage. The tests that involved only radial drainage were conducted to simulate the case of a subsoil improved by the installation of prefabricated vertical drains (PVDs) where, at depth, radial drainage is most likely to dominate the flow of pore water.

A parameter, K_w, was defined as the horizontal effective stress on the wall of a consolidation ring (σ'_{hw}, with suction defined as negative) divided by the vertical effective stress (σ'_v) in the soil specimen. When $K_w > K_0$ (where K_0 is the at-rest earth pressure coefficient of the soil), there will be a tendency for outward lateral displacement of the soil specimen, and when $K_w < K_0$ there will be a tendency for inward lateral displacement of the specimen.

The ratio of the magnitude of the surcharge pressure to the vacuum pressure was designated as the quantity *RL*, and the surcharge loading rate as *SLR*. The test results indicate that during the loading process, increasing both *RL* and *SLR* increased the measured value of K_w. The parameter, *RLS*, defined as the ratio of an index pressure (p_n) and the undrained shear strength (s_u) of the deposit (i.e., $RLS = p_n/s_u$), proposed by Chai et al. (2013), is able to include in a single parameter both the effect of *RL* and *SLR*, as well as the consolidation properties of the soil. The test results from this study indicate that *RLS* is a sensitive parameter for predicting lateral displacement of a soft soil deposit under the combined action of a vacuum pressure and a surcharge pressure.

It was also observed that the value of K_w at the end of consolidation increased with an increase in *RL*, independent of the value of *SLR* for the conditions investigated. In the presence of a vacuum pressure, at the end of consolidation the value of K_w was observed to be less than the initial value of K_0 of the soil, but this condition is not sufficient to ensure that the final lateral displacement of the ground will be inward, because there may plastic outward deformation occurring during the consolidation process. Therefore, to minimize the lateral displacement in the field, the loading scheme should ensure a value of K_w close to K_0 during the entire consolidation process not only the end of consolidation.

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