CHAPTER 5

K_O OF OLD ALLUVIUM

5.1 Introduction

The real behaviour of soil in-situ is often found to be different from the engineering properties obtained in laboratory. This is due to the fact that soil behaviour is stress state and stress path dependant. Before a soil sample is placed in test equipment, it has already been subjected to various stress and strain paths caused by sampling, transportation and laboratory handling. Thus, the properties measured in a laboratory test may not truly reflect its in-situ properties.

Since the goal of this soil characterization program is to offer guidance to real world engineering, the in-situ stress state and sampling effects must be studied. In the previous chapter, the sampling disturbance on Singapore Old Alluvium was investigated. The stress and strain changes underwent by soil samples during the sampling were discussed. This chapter provides a study on the in-situ stress state, which is the starting point of the stress change in sampling. Chapter 6 will discuss about the shearing behaviour of OA. However, even when the engineering properties of OA are known, the behaviour of the in-situ OA can not be estimated precisely without the clearly understanding of in-situ stress state.

5.2 Approach in This Study

The effective vertical stress σ'_v of a saturated soil can be calculated straightforwardly using depth, density and ground water level. Attention should be paid to the ground water level. As indicated by Figure 5-1, it is an oversimplification to assume a flat ground water level where the ground surface is rugged.

According to Vaughan (1994), the reason for a rugged ground water surface is due to capillary rise and surface flux. For granular soils, if the permeability is greater than 10^{-7}m/s, capillary rise is small but the soil is likely to be recharged by rainfall. Because the range of permeability of OA is from 10^{-6}m/s to 10^{-10}m/s (Dames & Moore, 1983) and tropical weather condition prevails in of Singapore, it is important to monitor the ground water level carefully to get an accurate in-situ condition.

Since the vertical effective stress is relatively easy to determine, this chapter focuses on K_o value of OA, which is the ratio of horizontal effective stress, σ'_h, to vertical effective stress σ'_v at rest.

Although extensive research work has been done on the horizontal stress at rest for homogeneous soils, the subject of K_o of mixed soils had not receive as much attention mainly because it is much more difficult to obtain. For Singapore OA, this problem is more complicated due to the over-consolidation stress history and heterogeneous composition.

Li & Wong (2001) compiled the K_o values measured in-situ using the pressuremeter tests and these are shown in Figure 5-2. The K_o varies from 0.5~2.7, which is a quite a big variation. It is important to understand the reasons for such variation. However, no information such as depth and composition was given in Li and

Wong (2001). Though an attempt was made to relate K_o values and SPT blow number N, the result was clearly not satisfying, considering the scatter.

In this chapter the K_o values are measured in the laboratory, and the results from in-situ pressuremeter tests will also be presented. Though limited in number, these tests were carried out in boreholes specially for this research and the borehole log was carefully documented. These results not only give K_o values, but also the soil composition and engineering properties at the depth where pressuremeter tests were done. This would facilitate better interpretation of the test results.

To understand the reasons for scattered K_o values from in-situ tests, laboratory tests on remoulded samples were conducted to evaluate the effect due to the stress path the soil has undergone. The goal was to simulate the possible stress paths OA samples might have experienced in the geological past. From the literature review, it is known that OA was transported by a braided river system and after sedimentation, the top part of OA had eroded. Thus, it went through loading first, then unloading stress path which resulted in an over-consolidation history. The soil will be reloaded if buildings are constructed on it. The laboratory approach will follow the first time loading, unloading and reloading and investigate the relationship between K_o values, clay contents and OCR. The information collected during laboratory testing will be used in interpretation of the in-situ test results.

5.3 Laboratory Test Results

5.3.1 K_o Consolidation Test Plan

Two types of apparatus were used to perform K_o consolidation in the laboratory. One is a special oedometer cell equipped with horizontal stress measurement using total stress cells. The other machine was a GDS stress path triaxial system, which is capable of performing K_o consolidation automatically. Details of test set up and procedures were given in Chapter 3.

Most of the samples used in this study were remoulded clayey sand. The host-sand is from one Kim Chuan site Mazier sample BH1A-MZ3. This host-sand was from 7m depth and had a clay content of 16.4% (see Chapter 4). The host-sand has a critical shearing resistance angle of $\phi'=32^o$. Kaolin clay with a friction angle of 25^o was added to form clayey sand samples with varying clay contents. K_o consolidation tests were also carried out on the same OA sample but remoulded. The PSD curves of the remoulded OA (natural soil which was disturbed and compacted) and host-sand are shown in Figure 5-3.

In oedometer tests, the soil is placed in the cell in slurry form with a water content of 80%. A seating load was added to ensure the contact between soil and total stress cells. Then the soil was loaded slowly using a WFi Tritech 100kN loading machine to avoid build up of pore pressure. The vertical stress was measured by the load ring and the horizontal stress was measured by the total stress cells on each side of the oedometer cell. Details of apparatus set up, calibration of total stress cells and correction can be found in chapter 3.

While doing triaxial K_o consolidation, the samples were set up in the cell and a back pressure was applied to ensure saturation. The soil sample was first consolidated to an anisotropic stress condition σ'_v=100kPa, σ'_h=50kPa and then K_o consolidation started using an automated module built into the control software.

5.3.2 K_o values in First Loading: K_{onc}

Figure 5-4 shows the relationship between K_o and σ'$_v$ of 7 triaxial samples during first time loading, unloading and reloading. It can be seen from the figure that in first loading, the K_o value is almost constant. Therefore, K_{onc} is defined as the K_o value of a loose, normally consolidated soil during the first K_o loading. The K_{onc} values measured in first K_o loading are listed in Table 5-1. Clay content up to 20% does not change the K_{onc} value of clayey sand in first loading. The curves of clean sand, 10% clay, 20% clay and remoulded OA in first loading are very close and the K_{onc} value was found to be 0.445.

Table 5-1 K_{onc} values measured in First Time Loading

Sample	K_{onc} Measured in Triaxial Cell	K_{onc} Measured in Oedometer
Clean Sand	0.445	0.430
Sand with 10% Clay	0.445	/
Sand with 20% Clay	0.445	0.430
Sand with 30% Clay	0.470	0.430
Sand with 40% Clay	0.529	0.551
100% Kaolin Clay	0.539	/
Remoulded OA (16.4% Clay)	0.447	0.430

Adding the clay content to 30% seems to result in the first sign of change with a slight increase in K_o, possibly due to a reduction in the friction angle. The 30% clay sample has a K_{onc} value of 0.470. Further increase of clay content to 40% will result in higher K_{onc} value, that is very close to 100% pure kaolin clay. K_{onc} value is 0.529 for 40% clay and 0.539 for 100% pure kaolin clay.

Five samples were tested in oedometer cell and the results are shown in Figure 5-5. The process also involved first time loading, unloading and reloading. In oedometer tests the samples were unloaded to a low stress state before reloading. In the case of remoulded OA, the test was stopped after first time loading due to failure of one total stress cell. Similar to triaxial test results, clean sand, 20% clay and remoulded OA have the same K_{onc} value at first loading, which is estimated to be 0.430. The behaviour of 30% clay in oedemeter is somewhat different with the triaxial 30% clay sample. In oedometer, the measured K_{onc} value is 0.430, the same as clean sand and 20% clay K_{onc} values and lower than the value measured in triaxial cell. For 40% clay sample, the K_{onc} value is 0.551 measured in oedometer cell.

Though there are some discrepancies in the K_{onc} values measured in triaxial and oedometer tests, the variation in K_{onc} with clay content can be clearly seen from the results of both tests. As shown in Figure 5-6, for clay content of up to around 30%, the clay content has little influence on the K_{onc} values. Therefore in first loading, clayey sands with low clay contents have nearly the same K_{onc} as that of clean sand. A threshold clay content beyond which K_{onc} changes significantly seems to exist between 30% and 40%. After this threshold value, the soil will have a much higher K_{onc} value, a value that is nearly the same as that of the pure clay.

Jaky's equation, $K_o = 1\text{-}\sin\phi'$ is used to estimate coefficient of earth pressure at rest. According to Wood (1990), the nature of the angle ϕ' used in Jaky's equation is uncertain. For sand, the value of earth pressure coefficient at rest, K_o, depends on the in-situ structure of the sand and can be expected to correlate with the peak angle of shearing resistance measured in triaxial compression, associated with the initial density and structure of the sand. It would not be appropriate to use the critical state angle in Jaky's equation to estimate the value of K_o for a dense sand.

Soil samples used in this first time loading study were all loose, normally consolidated soils for which the peak angle of shearing resistance is also the critical state angle. It was found using critical state angle in Jaky's equation yields good estimation for the K_{onc} values of clean sand and clayey sand. This result is consistent with our knowledge of K_o values and clayey sand like OA. As mentioned in Chapter 2, clay content up to a certain level will not affect the friction angle of clayey sand and that is the reason why clayey sand with low clay content has the same K_{onc} value as clean sand. However, if the clay content exceeds a threshold value, the soil sample will behave like clay and the K_{onc} value will be nearly the same as pure clay.

This first loading stage represents the sedimentation process OA experienced after transportation by the braided river system. Based on the test results, it can be concluded that K_o values will vary between 0.43 to 0.55 in this process according to the composition of the soil. Thus the much bigger K_o values seen in Li and Wong's data are not likely to be generated by the first time loading of the soil; the big K_o values may reflect its stress history experienced after sedimentation.

5.3.3 *K_o* Values in Unloading

Figure 5-4 also shows the relationship of K_o and σ'_v of seven triaxial samples in unloading. It can be seen that σ'_h does not decrease at the same speed as σ'_v, suggesting some 'lock up' in horizontal stress and as a result, K_o value increases as OCR increases. In this process, clay content plays an important role in K_o values.

For clayey sands with low clay contents (<30%), during first loading, the K_o values remain very much unchanged. However, this is not the case in unloading. The relationship of clay content and K_o values in unloading is shown in Figure 5-7. As can be seen in both triaxial and oedometer tests, during the unloading stage, at the same OCR value K_o will generally increase with increase in clay content. However, in Figure 5-7(a), at OCR value 3 and 4, there is decrease in K_o values at clay content 10% and 16.4% and the reason is not clear. It can be also seen from Figure 5-7 (a) that for clayey sands with high clay contents, like the 40% clay sample, the soil shows the same behaviour as pure clay.

During the unloading stage, Jaky's equation is no longer useful. According to Schmidt(1967), K_o values in unloading can be expressed in the following form:

$$K_o = K_{onc}\mathrm{OCR}^a \tag{5.1}$$

This equation is found to be appropriate to describe the unloading data. Values of K_o were plotted and parameter a was selected to fit the trendline using the least square method. Parameters a and the coefficient of determination R^2 are presented in Table 5-2. It can be seen that Equation 5.1 fits the test data very well with the selected a values since most of the R^2 values are greater than 0.99. Relationship between parameter a and the clay content is shown in Figure 5-8. For parameter a, at first a increases slowly with clay content from 0% to 20%, but there was a big jump when the clay content increases from 20% to 30%, then a big slump, forming a peak value at around 30% clay content. Thus, given the clay content of an OA soil sample and OCR value, the K_o during unloading can be estimated.

However, for cohesionless soils the upper bound for K_o is dictated by the passive earth pressure coefficient K_p,

$$K_p=\tan^2(45+\phi'/2) \tag{5.2}$$

Note that in the triaxial K_o tests, an extension cap was not used and the stress path cell cannot apply a horizontal stress greater than vertical stress. Thus, in the present series of triaxial tests, the upper bound for K_o values is controlled by the isotropic line due to limitation in the test set-up, and this point is clearly shown in Figure 5-4.

Table 5-2 Parameter K_{onc}, *a* with Clay Content

Sample		K_{onc}	*a*	Clay Content (%)	Coefficient of Determination R^2
Triaxial K_o Test	Clean Sand	0.445	0.500	0	0. 986
	10% Clay	0.445	0.514	10	0. 994
	20% Clay	0.445	0.545	20	0. 993
	30% Clay	0.470	0.575	30	0. 991
	40% Clay	0.529	0.510	40	0. 993
	Kaolin Clay	0.539	0.495	100	0. 996
	Remoulded OA	0.447	0.500	16.46	0. 994
Oedometer Ko test	Clean Sand	0.430	0.455	0	0. 995
	20% Clay	0.430	0.470	20	0. 943
	30% Clay	0.430	0.599	30	0. 997
	40% Clay	0.551	0.447	40	0. 997

Singapore OA has an over-consolidation stress history which is caused by erosion. At the same location, erosion will cause soils at different depths to have different OCR values. It can be clearly seen in this section that in unloading, OCR has a strong influence on K_o values. Different OCR values together with soil composition are probably the reason behind the scattering K_o values reported by Li & Wong (2001).

5.3.4 K_o Values in Reloading

Figure 5-4 shows the K_o and σ'_v values of triaxial tests. After the soil was unloaded to the isotropic stress state, reloading started. The reloading K_o line is in between unloading K_o

line and first time loading K_{onc} line. With the increase in σ'_v and the decrease of OCR, the K_o in reloading decreased and gradually approaches the K_{onc} line, merging with the K_{onc} line around OCR=1.

However, the K_o values in reloading is not only related to OCR, but also related to the point where reloading starts. Figure 5-5 presents the K_o and σ'_v values of oedometer tests. In these tests, the samples were unloaded to very low σ'_v and thus very high OCR values when reloading started. At first, the K_o values in reloading is even less than K_{onc}. When the σ'_v value approached the preconsolidation vertical stress, which is the maximum vertical stress experienced, the K_o values start to increase and reloading lines move towards the first time loading line.

It can be found that the K_o values during reloading can be even less than K_{onc} if the sample starts from a very low stress state. Theoretically, for cohesionless soils, K_o values should not be lower than the active earth pressure coefficient K_a, which is

$$K_a=\tan^2(45-\phi'/2) \tag{5.3}$$

An interesting question is whether Jaky's equation, K_o=1-sinϕ', is still appropriate for such cases when the soil is heavily over-consolidated. As stated earlier by Wood (1990), ϕ' used in this equation should be peak angle of shearing resistance. Obviously, after the first time loading and unloading, the soil is densified and the peak angle of shearing resistance is higher than the critical state angle. Using the peak angle in Jaky's equation will therefore result in K_o value lower than the K_{onc} value in first time loading. What's more, as the sample is reloaded, there is decrease in OCR, accompanied by a decrease of peak angle of shearing resistance. At the point the sample is reloaded to the preconsolidation pressure, the sample is normally consolidated and the peak angle will be equal to the critical state friction angle. Hence, if Jaky's equation is used, the calculated K_o will also show the same trend as that shown in Figure 5-5: it will start with a lower value and approach the K_o line in first time loading gradually. To prove such an idea needs systematic research using triaxial shearing to get the peak angles of shearing resistance along the reloading line, something beyond the current scope of work.

Based on the above discussion, it can be seen that K_o in reloading is difficult to estimate because it's not only related to OCR but also the point when reloading starts. Generally speaking, when soil is reloaded in K_o condition, the K_o line will be lower than the K_o line in unloading. If the soil is unloaded to a very low stress state, the K_o values will even be less than K_{onc}. However, the reloading K_o line will move towards the first time loading K_{onc} line when σ'_v approaches the maximum vertical stress, regardless of where the reloading starts.

After Singapore OA had been unloaded by erosion, OA in some places was also reloaded due to the sedimentation of a younger clay. However, based on Li & Wong's result on in-situ K_o values, it seems that few K_o value less than 0.5 exists. That may imply that OA hadn't been unloaded to very low stress levels, as in the case of Figure 5-5.

In engineering projects, it is often required to estimate the K_o value of the soil on site. It can be seen from the above discussion that K_o is related to the soil type and stress history. Therefore, site investigation should provide a clear picture of the in-situ soil composition and geological information. Construction changes the stress condition of the site: soil is unloaded during excavation and reloaded when the building starts. So the K_o values of soil are changing during the work. It is hoped the above discussion on OA K_o

values in first time loading, unloading, and reloading will help to provide a better understanding of how to arrive at better estimation of OA in-situ state.

5.3.5 Possibility of Laboratory Estimation of In-situ K_o Value Using Intact Soil Samples

It will be very useful if the in-situ K_o value can be estimated using laboratory tests on soil samples taken from the ground. Suppose that a soil sample is taken from the ground, set up in a triaxial cell and saturated. The in-situ vertical stress σ'_v of the soil sample can be determined easily using sample depth and ground water table. The question is, if the sample is K_o consolidated in the triaxial cell to the effective vertical stress σ'_v, will the effective horizontal stress σ'_h at that time be equal to the in-situ horizontal stress?

With the knowledge from the discussion on K_o values of reconstituted soil, it can be seen that more information is needed to answer the above question and to determine the in-situ K_o value. The situation is complicated because it is related to the stress and strain changes during the sampling process and laboratory handling.

For discussion purpose, first suppose K_o condition is imposed all along the sampling and laboratory working stage, so there is no lateral strain. As shown in Figure 5-9, the in-situ stress state of a normally consolidated soil sample is at point A. During the sampling process, the soil sample was unloaded in K_o consolidation. Later the soil sample is set up in triaxial cell, saturated and consolidated, usually to a low stress level. Then the sample is K_o loaded in triaxal cell. Remember since the soil has experienced unloading during sampling, this K_o loading in triaxial cell from a low stress level is reloading. The soil sample will go along the reloading line in K_o condition, and loading to σ'_{vA} will get the in-situ σ'_h because the reloading line and the first time loading line meet at point A.

If the in-situ soil sample is already over-consolidated, the problem became more complex. Suppose that the soil had been loaded to point A in first time loading and unloaded to point B due to erosion. After sampling from point B, it is set up in a triaxial cell and K_o loaded. Reloading the soil sample in K_o condition to σ'_{vB} will only get the σ'_h value at point B' on the reloading line, which is much less than the correct in-situ σ'_h at point B. The right method is to continue to load the sample to σ'_{vA}, which is the preconsolidation pressure, then unload it to σ'_{vB}. That is to say, for overconsolidated soil, not only the current vertical stress but also the preconsolidation stress (OCR) is needed.

In the above discussion, it is supposed that sampling and laboratory handling of soil samples conform to K_o condition. This hypothesis is not realistic and it's only for discussion purpose; rarely does sampling conform to K_o condition. Detailed study on the effect of sampling disturbance on shear strength of OA has been provided in Chapter 4. In this chapter, only the effect of 'perfect sampling' on K_o will be investigated. A 'perfect sampling' stands for the process of removal of the deviator stress, q, in an undrained condition. After 'perfect sampling' the deviator stress is removed so the soil is at an isotropic stress condition of $\sigma'_1=\sigma'_3$ (see Figure 4-1).

As shown in Figure 5-10, for a normally consolidated soil, the soil sample goes from its in-situ stress state point A to point A' on the isotropic line. For lightly overconsolidated and heavily overconsolidated (K_o >1) soils, the stress paths are from B to B' and C to C' respectively. In laboratory K_o test, generally the sample is first saturated using back pressure under isotropic condition, and a low effective confining pressure is applied. K_o consolidation normally starts from a low stress level, such as point P shown in Figure 5-10. Before any attempt is made to estimate in-situ K_o value using laboratory K_o consoliation test, the effect of stress paths AA'P, BB'P and CC'P should be understood.

If the soil sample is heavily overconsolidated (K_o >1), the stress path CC'P at any point, is below the K_o unloading line. That is to say, along this route the effective horizontal stress is always lower than $K_o\sigma'_v$. This will lead the sample to be actively sheared away from the K_o condition.

If the soil sample is normally consolidated or lightly overconsolidated (K_o <1), during this process of 'perfect sampling' from A to A' and B to B', σ'_h will increase and σ'_v will decrease. The soil sample will be somewhat passively sheared away from the K_o condition and elongates. However, in the following isotropic unloading from A' to P and B' to P, the route crosses the K_o unloading line and the sample will experience some active shearing. The overall combined effect of sampling and isotropic unloading is difficult to estimate: possibilities of active and passive shear both exist.

Research was carried out to determine K_o of presheared soils and it was found when a presheared soil is subjected to K_o condition (consolidation without lateral deformation), it tend to move towards the K_{onc} line in first time loading but will not return to it (Sallfors, 1975; Feda 1984; Topolnicki *et al.*, 1990). As shown in Figure 5-11, if the soil is actively sheared to point A or passively sheared to B and then K_o conditon is imposed, the K_o consolidation behaviour will move towards K_{onc} line of first time loading but will not return to it. Eventually the slope of the K_o consolidation line after shearing will be equal to K_{onc} and the K_o line after shearing will stay parallel to K_{onc} line.

According to Mersi & Hayat (1993), this is because the soil structure suffers a permanent change in shearing, which persists even during subsequent K_o loading. What's more, based on data from literature and their own data on soft clay, it was proposed there is a unique relationship between the horizontal pressure increment or decrement that is imposed to move soil away from the K_o line $[\sigma'_h]_s$ and the final offset $[\sigma'_h]_I$.

In the present study, one test was carried out on a lightly overconsolidated sample in a triaxial cell. The sample used was reconstituted OA from Kim Chuan BH1A-MZ3. The stress path of the soil is shown in Figure 5-12 using σ'_h versus σ'_v plot. The sample was first K_o consolidated along the K_{onc} line in stress path cell and then unloaded. Then at an OCR of about 2.7, 'perfect sampling' was applied to the soil sample by closing the back pressure valve and removing the load from the soil sample. Clearly, this will address only the issue of stress removal and not any other disturbance. Then the sample was unloaded to the isotropic line and K_o consolidation was restarted at low confining stress. During reloading, it can be seen that the soil sample goes on a curved reloading K_o line, and approach the K_{onc} line at the preconsolidation pressure, which indicates that 'perfect sampling' did not destroy the preconsolidation information stored in the soil sample. After that K_o unloading is applied again. This time the second unloading K_o line lies above the first unloading K_o line. This seems to indicate that the combined effect of 'perfect sampling' and isotropic unloading for this overconsolidated sample is passive shearing, which resulted in a positive offset $[\sigma'_h]_I$.. It can be further deduced that for normally consolidated soil sample, the overall effect will also be passive shearing. Thus, in laboratory K_o tests on soil samples retrieved from the ground using 'perfect sampling' techniques such as block sampling, normally consolidated and lightly overconsolidated

OA will show a higher K_o value than in-situ while heavily overconsolidated OA will show a lower K_o value. It can be seen that stress history of the site is necessary to determine the loading path of laboratory K_o test and to correct errors caused by sampling effects.

5.4 K_o Values Obtained by Pressuremeter Tests

5.4.1 Pressuremeter Test Results

Pressuremeter tests were carried out on Kim Chuan Site to explore the in-situ OA properties. Details of Kim Chuan Site has been introduced in Chapter 4. As shown in Figure 5-13 (Mair and Wood, 1987), the pressuremeter apparatus consists of a cylindrical probe and a volume-measuring unit, which is called a volumeter. Single-cell Pressuremeter was used in this project. The pressuremeter probe belongs to the Ménard type pressuremeters (Ménard, 1957) and has a single measuring cell. To perform the test, a hole with a diameter of about 65 mm was first made in the borehole. The probe, which was calibrated before each test (Details of calibration please see Clark, 1995), was then lowered into the borehole. The test began by expanding the probe by water flowing from the volumeter and the water was pressurized by compressed nitrogen gas.

According to Mair and Wood (1987), the expansion of the probe caused the soil surrounding the borehole at the testing depth to deform and the relationship between the applied pressure and deformation of the soil is obtained. The applied pressure is measured by the pressure gauge and the deformation of the soil is measured by cavity volumn, V, or cavity radius, r. Cavity strain ε_c is also commonly used. Cavity strain is defined as:

$$\varepsilon_c = (r - r_o) / r_o \tag{5.4}$$

in which r is the current cavity radius and r_o is the initial cavity radius.

As shown in Figure 5-14, usually there are 3 distinct phases in a Ménard type pressuremeter test. Phase 1 is from the starting of the test to point A, when the membrane is in full contact with the borehole wall and the pressure at point A is the initial pressure p_o. Phase 2 is approximately linear until point B, when the soil adjacent to the pressuremeter begins to creep, indicating the onset of failure of the soil. The pressure at point B is the creep pressure, p_f. In phase 3 the soil continues to fail till point C, when the soil shows completely plastic behaviour. The pressure at point C is the limit pressure p_l.

In this project, tests were carried out in one loading-unloading cycle and The applied pressure is plotted against cavity strain in Figure 5-15. First, the pressuremeter test quality in this project needs to be examined. The installation of the probe will inevitably cause disturbance in the soil. In the case of Ménard type pressuremeters, a hole is first drilled and causes unloading to the adjacent soil. The disturbance will affect the shape of the loading curve.

Clarke (1995) studied the effect of installation on the shape of a test curve and Figure 5-16 shows several cases in prebored pressuremeter tests. Curve A is a classical curve, indicating ideal installation. Curve B means that the probe is pushed in undersize pocket. Curve C means the pocket is somewhat too large and limited information can be get in the test. When the pocket is really too large, curve C' will occur and no information can be obtained.

According to Clarke (1995), the shape of ideal pressuremeter curves can also be used to identify the major ground type. Figure 5-17 from Clarke (1995) shows the type of curves obtained from self-boring pressuremeter tests in different ground conditions. After yield, the slope of a test curve for clay (AB in the figure) is significantly less than that for a sand (CD). The unloading curves for sand and clay are also different. In clays, the membrane of the pressuremeter often remains expanded as shown by H, due to suction in the clay.

To study the shape of pressuremeter tests on Kim Chuan Site, the normalized test curve is shown by Figure 5-18. The applied pressure is normalized by maximum pressure in each test. Comparing the loading curve in Figure 5-16 and Figure 5-18, it can be seen that the installation quality is good and the shape in Figure 5-18 belongs to case A, the classical curve, indicating ideal installation.

The ground type is also well represented in these tests. It is clear that in all the seven tests, there are two tests conducted in clay layers: test I, BH-1A 6m and test IV, BH-2A 4m. After yield, the slope is significantly less than other tests. During the unloading stage, the membrane remained expanded. The borehole log also confirms the result, marking the soil layer sandy Clay, while all the other tests were carried out in clayey Sand layers.

5.4.2 In-situ Horizontal Stress

As can be seen, pressuremeter test does not record modulus or horizontal stress directly. All it gives is the relationship shown in Figure 5-15. Desired soil properties need to be interpreted from these figures and the interpretation depends heavily on test quality and soil profile. In this research project, 7 pressuremeter tests were carried out. Though limited in number, the borehole profile is documented carefully and samples were taken at nearly the same depth of pressuremeter tests. Thus, the pressuremeter tests can be better interpreted using these additional information.

To decide the in-situ horizontal stress, a reference cavity pressure p_r need to be selected at a point when the pressure inside the cavity equals to the in-situ horizontal stress. It is easy to mistake p_o in Figure 5-14 as the in-situ horizontal stress. According to Cassan (1978), there is no justification for this and it should not be used. The process of forming a borehole for a Ménard type pressuremeter test leaves the soil totally or partially unsupported. Even if intact soil is being stressed from point A in Figure 5-14, it is being stressed from this unloaded condition and not from its undisturbed in-situ stress state.

For Ménard type pressuremeter tests, there are 2 graphical iteration methods to determine in-situ horizontal stress. One is developed by Marsland & Randolph (1977). First, an estimation of reference pressure p_{r1} is made and the cavity strain at that pressure (ε_o) is found from the test curve. Then the cavity strain of the test is corrected using the following equation:

$$\varepsilon = \frac{\varepsilon_c - \varepsilon_o}{1 + \varepsilon_o} \qquad (5.5)$$

After that, the undrained shear strength Cu is found as the peak slope in the pressure versus ln($\Delta V/V$) plot. One yield point is select from phase 3 of the test and the pressure is p_y. This yield point indicates the limit of the elastic zone or the onset of yield. If the estimated σ_{h1} is the in-situ horizontal stress, then the following equation should be satified:

$$p_y = p_{r1} + Cu \qquad (5.6)$$

If not satisfied, the a new estimation of σ_{h2} is made using

$$p_{r2} = p_y - Cu \tag{5.7}$$

The process is repeated until the equation is satisfied. Thus, the in-situ horizontal stress is determined.

Another graphical iteration method was developed by Denby & Hughes (1982). Several estimations of p_r are made and in each case, ε_o is found accordingly. The cavity strain of the test is corrected using equation (5.5) and volumetric strain $\Delta V/V$ is also recalculated. Finally, all these p versus corrected $\ln(\Delta V/V)$ relations are plotted on the same figure and the one gives the most linear plot are taken as the best estimation of in-situ horizontal stress.

Though various methods are developed, as admitted by Clarke(1995), determine the in-situ horizontal stress is still subjective. Though the methods in theory offer a neat mathematical solution but, in practice, the models used do not describe the soil adequately. That's why knowledge of the soil is important when interpreting pressuremeter test results. Limits should apply to the chosen values of horizontal stress, which is shown in Table 5-3

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Table 5-3 Limits applied to chosen horizontal stress (after Clarke, 1995)

$\sigma_h - \sigma_v < 2Cu$	for all clays
$\sigma_h > u_o$ (pore pressue)	for all soils
$Cu/\sigma'_h > 0.3$	(based on Skempton's relationship for normally consolidated clays)
$\sigma'_h / \sigma'_v > (1-\sin\phi')/(1+\sin\phi')$	for sands
$K_a < \sigma'_h / \sigma'_v < K_p$	for all soils

All the 7 pressuremeter tests in this project are analyzed using both Marsland & Randolph and Denby & Hughes methods. Table 5-4 presents values of p_r determined using Marsland & Randolph method. Please note in test III, V, VI, VII the yield point is not achieved by loading. Therefore in these tests, p_r values can't be decided.

Table 5-4 Reference Cavity pressures selected using Marsland & Randolph method

Test No.	Borehole	Depth (m)	Reference Cavity pressure, p_r (kPa)	Undrained Shear Strength, *Cu* (kPa)	Yield Pressure, p_y (kPa)
I	BH-1A	6.0	184	151	335
II	BH-1A	10.0	254	2000	2254
III	BH-1A	22.0	?	?	>4500
IV	BH-2A	4.0	119	443	562
V	BH-2A	8.0	?	?	>4500
VI	BH-2A	18.0	?	?	>5500
VII	BH-2A	19.0	?	?	>6000

Denby & Hughes method was also used on these pressuremeter test results and the way to select reference cavity pressure p_r is illustrated in Figure 5-19. Several estimations of p_r values were made and data of p versus corrected $\ln(\Delta V/V)$ were plotted accordingly. As shown in the figure, the one with the longest linear part is selected as the reference cavity pressure p_r.

As shown in Figure 5-13, p_r is the air supply pressure when the pressure inside the cavity equals to the in-situ horizontal total stress. That is to say,

$$p_r+\rho_w H=\sigma_h=\sigma'_h+\rho_w h \qquad (5.8)$$

After p_r values decided, both in-situ effective horizontal stress σ'_h and K_o values can be calculated. If there's discrepancy between the results of Denby & Hughes method and Marsland & Randolph method, the final p_r values selected are average values of both methods. Table 5-5 presents the calculated in-situ stress state and K_o values obtained.

Table 5-5 In-situ stress state at Kim Chuan Site Borehole BH-1A and BH-2A

Test No.	Borehole	Depth (m)	p_r (M&R) (kPa)	p_r (D&H) (kPa)	p_r (average) (kPa)	σ'_v (kPa)	σ'_h (kPa)	K_o
I	BH-1A	6.0	184	157	170.5	86	196.5	2.28
II	BH-1A	10.0	254	210	232	126	258	2.05
III	BH-1A	22.0	?	272	272	246	298	1.21
IV	BH-2A	4.0	119	119	119	58.5	137.5	2.35
V	BH-2A	8.0	?	133	133	98.5	151.5	1.53
VI	BH-2A	18.0	?	236	236	198.5	254.5	1.28
VII	BH-2A	19.0	?	239	239	208.5	257.5	1.23

As can be seen, in-situ K_o values are all above 1 and decrease with depth. From the previous research in laboratory, it is know that K_o>1 are usually due to over-consolidation. The fact that K_o decreases with depth also suggests it's over-consolidation due to erosion. In Chapter 2 it is mentioned that top of Singapore Old Alluvium had been eroded. This certainly would leave the remaining OA in over-consolidation. At one place, for a certain height of OA eroded, as the depth increase the OCR would decrease, resulting in decreasing K_o values.

An attempt was made to estimate the height of erosion using the field K_o data and the relationship of K_o with OCR obtained in laboratory testing. First, the ground water table of that time is needed. According to Biswas (1973), at the time of OA deposition the sea level drop was about 43.89 to 71.93m, so probably at that time the ground water table was also much lower than the level all these pressuremeter tests were carried out.

Therefore the preconsolidation pressure may happened during a time when the OA was unsaturated and the unit was around 20kN/m^3.

If a certain height of erosion is assumed, at a certain depth, the preconsolidation pressure can be calculated. Then OCR is obtained since the current σ'_v is known. From laboratory K_o tests, it is known K_o in over-consolidation can be expressed by K_o= K_{onc}OCR^a. For sand mixed with kaolin clay, K_{onc} and the parameter a can be obtained using Figure 5-6 and Figure 5-8 and if the clay content is known.

Table 5-6 Estimated K_o Values for 70m Erosion

Test No.	Borehole	Depth (m)	OCR	Clay Content (%)	K_{onc}	a	Estimated K_o
I	BH-1A	6.0	17.3	35.6	0.495	0.533	2.26
II	BH-1A	10.0	12.1	17.9	0.438	0.500	1.52
III	BH-1A	22.0	6.7	18.3	0.438	0.500	1.13
IV	BH-2A	4.0	24.9	39.6	0.540	0.479	2.52
V	BH-2A	8.0	15.2	15.9	0.438	0.500	1.71
VI	BH-2A	18.0	8.1	16.9	0.438	0.500	1.25
VII	BH-2A	19.0	7.7	15	0.438	0.500	1.22

Table 5-6 presents the estimated K_o values with an assumed 70m erosion. The comparison of real and estimated K_o values are shown in Figure 5-20. As can be seen, the match between real and estimated values for BH-2A is very good but for BH-1A, results do not agree very well. Several reasons may count for this deviation:

1) The above analysis assumes the in-situ OA only experienced first time K_o loading and then unloading due to erosion. The real stress history may be much more complex than this.

2) Parameter K_{onc} and a were from laboratory tests on reconstituted sand with kaolin clay. The composition of OA is more heterogeneous, with sand, silt and clay. The

clay in OA is a mixture of kaolinite, illite and smectite. So parameter K_{onc} and a from Figure 5-6 and Figure 5-8 were not accurate enough.

Despite the above deficiencies, the knowledge of K_o during first time loading, unloading and reloading obtained by laboratory tests still helps to explain the reason behind scattering K_o data by Li and Wong (2001). The pattern of K_o decrease with depth at Kim Chuan Site can be also explained by erosion caused over-consolidation. The current deviation of estimated erosion is mainly due to lack of detailed OA geology knowledge and accurate parameters (K_{onc} and a) for OA.

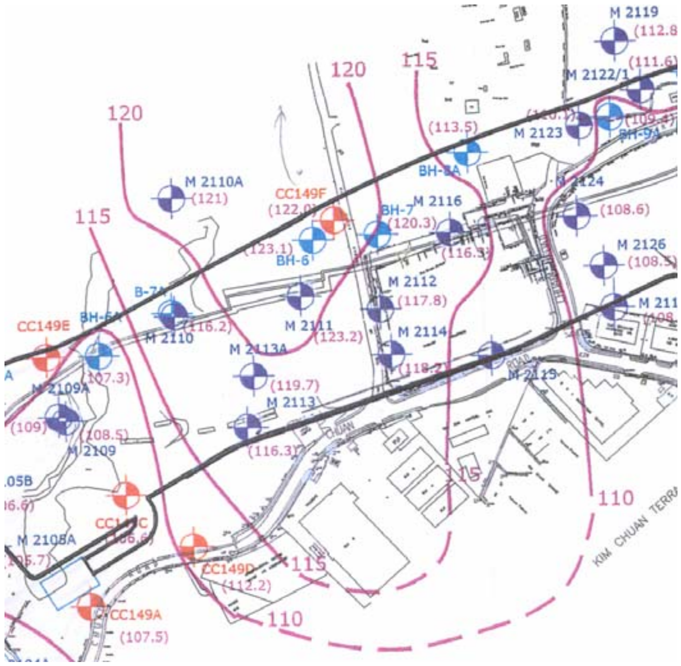
5.5 Conclusion

In this chapter, the K_o value of OA was investigated. Both laboratory and field tests were performed. Based on the above test results and analysis, it can be concluded that:

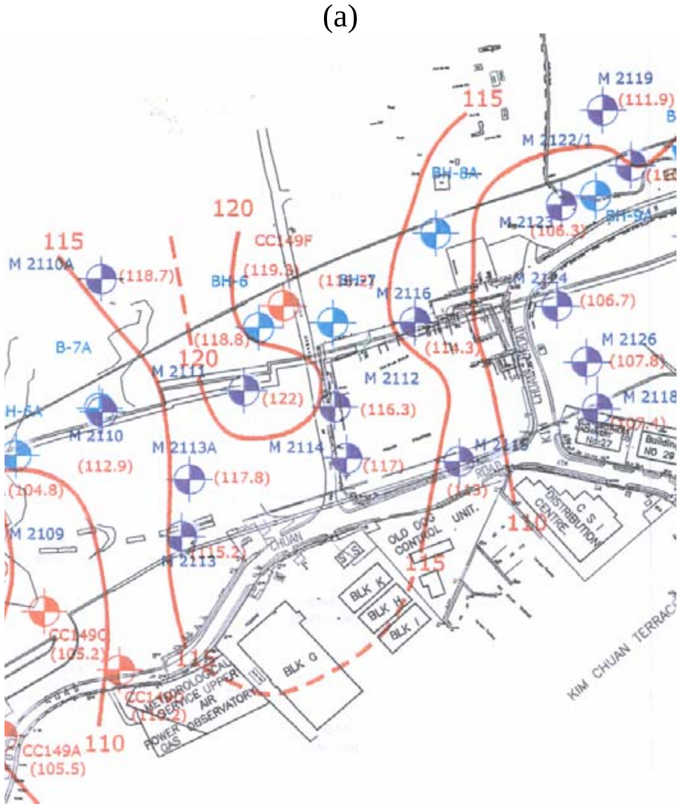
1) For loose, normally consolidated OA in first time loading, K_o value (K_{onc}) is constant and comply with Jaky's equation K_o =1-sinϕ'. ϕ'is the critical state friction angle. Clay content up to 20% does not change ϕ' at critical state, therefore does not change K_o value.
2) For over-consolidated OA, K_o values can be expressed by K_o=K_{onc}OCR^a. Parameter a are is related to clay content in OA.
3) Even 'perfect sampling' of the in-situ soil will cause shearing away from the K_o state and will cause deviations of laboratory measured K_o and in-situ K_o. For normally consolidated and slightly over-consolidated OA, the effect is passive shearing while for heavily over-consolidated OA, the effect is active shearing.

4) Measuring in-situ horizontal stress in OA is not easy, but still can be done using pressuremeter tests. Test quality should be ensured by careful handling and supervision. Interpretation of the test data is to somewhat subjective, but with proper understanding of the in-situ soil and careful analysis, it is still possible to get consistent and reliable results using graphical iteration method.

5) An attempt was made to estimate the in-situ K_o values using parameters obtained in laboratory tests and succeeded in one borehole. Estimation can be improved with better geological information and more accurate parameters.



(a)



(b)

Figure 5-1 (a) Contours of Ground Levels, Kim Chuan Site
(b) Contours of Ground Water Levels, Kim Chuan Site

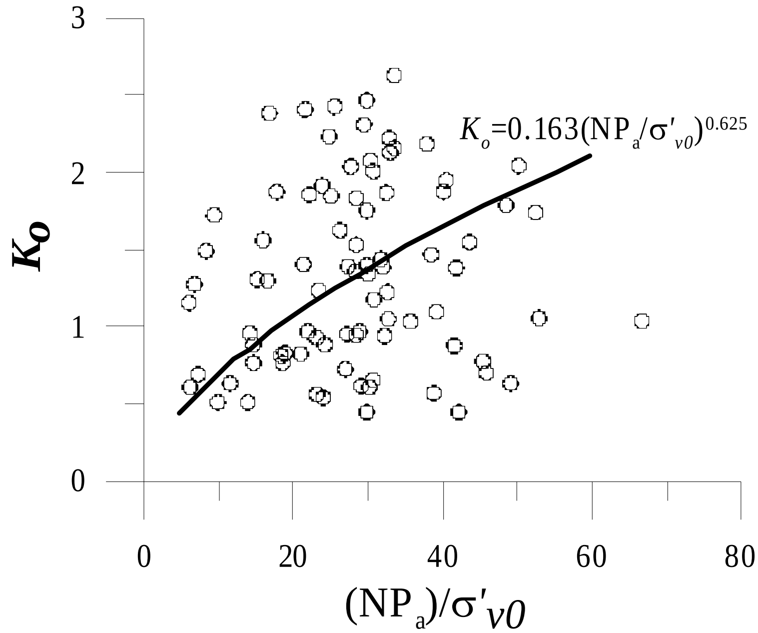


Figure 5-2 K_o Values according to Li & Wong (2001)

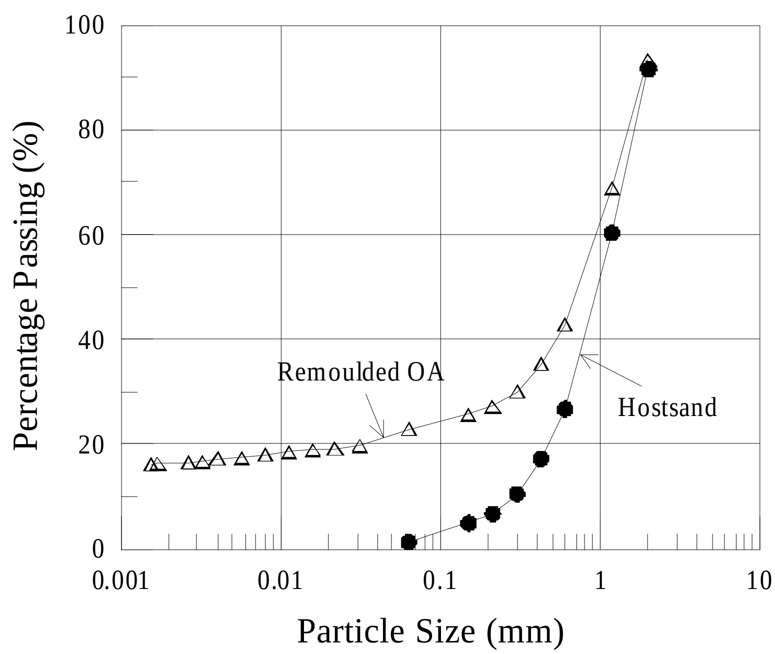


Figure 5-3 Particle Size Distribution of OA Used in Laboratory K_o Tests

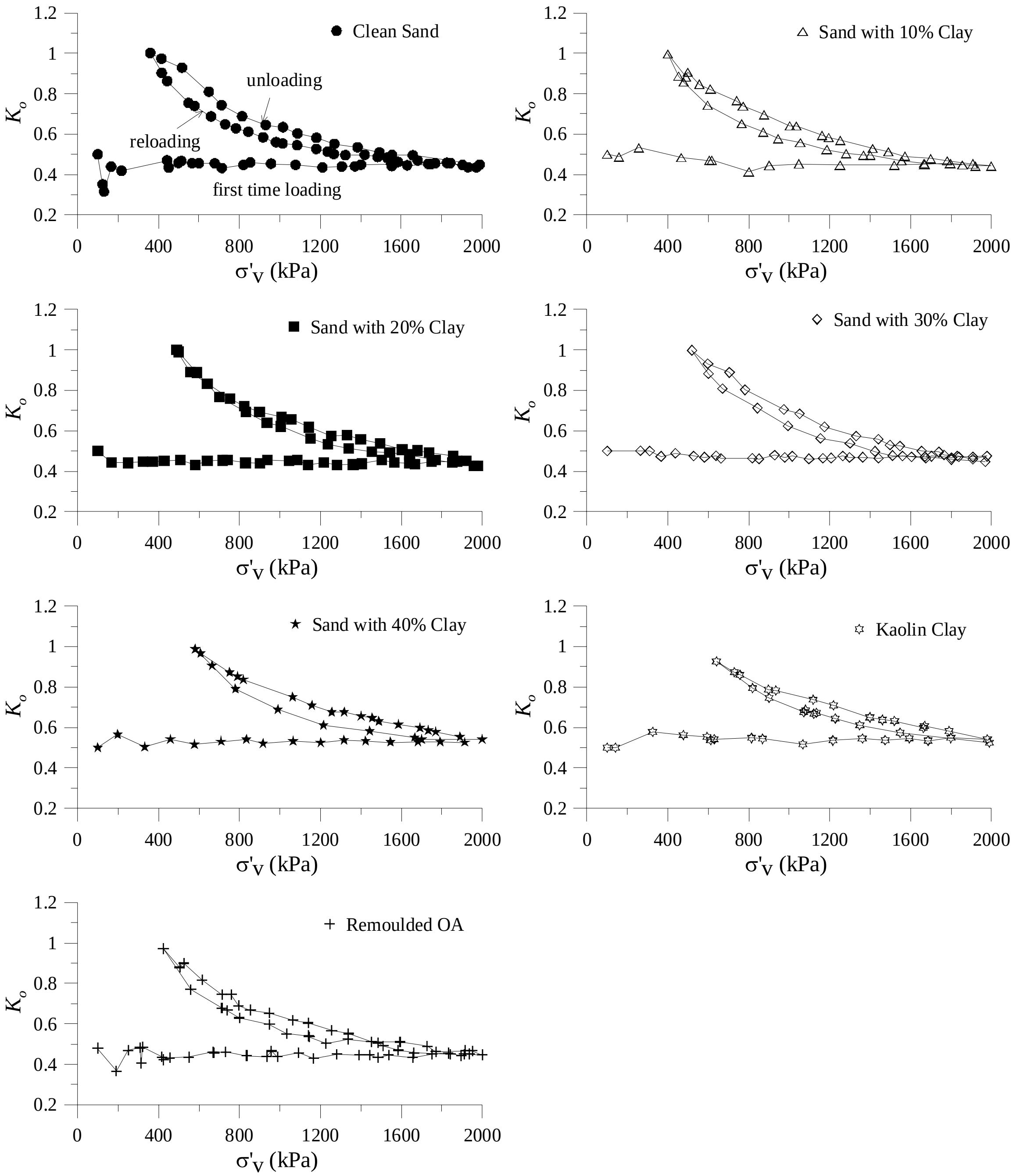


Figure 5-4 Triaxial K_o test curves

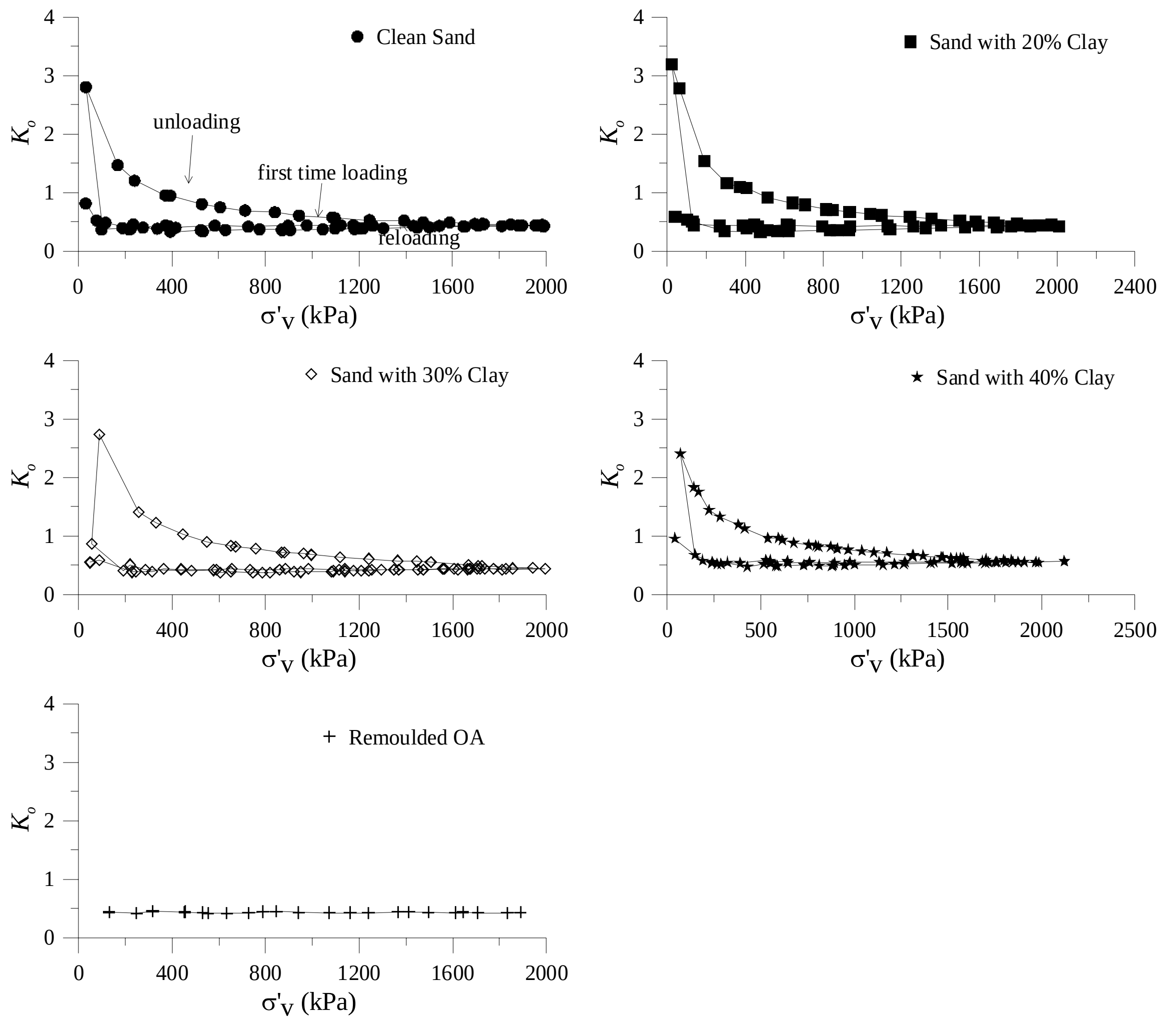


Figure 5-5 Oedometer K_o test curves

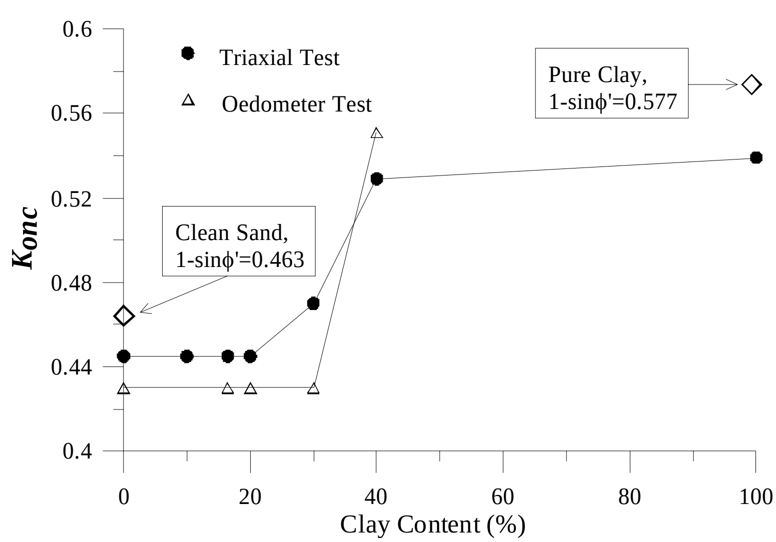


Figure 5-6 Relationship of K_{onc} and clay content

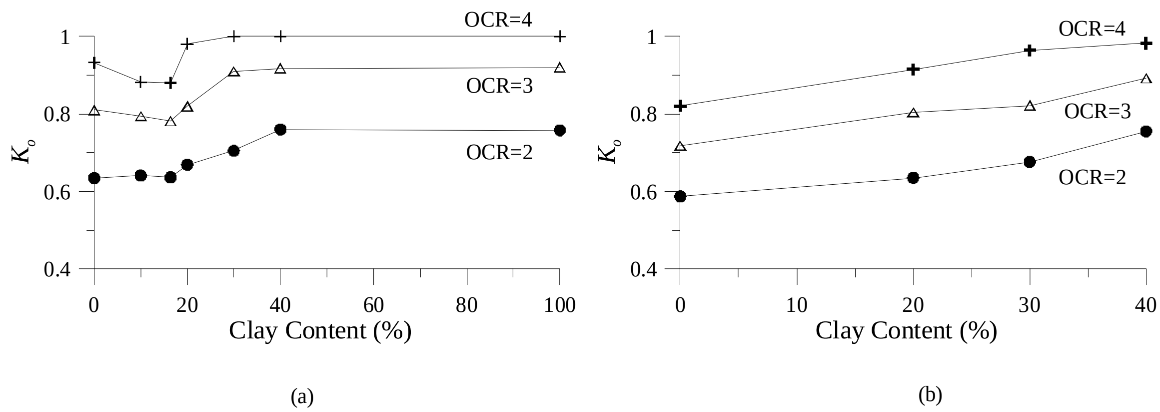


Figure 5-7 Relationship of clay content and K_o values in unloading (a) Triaxial test (b) Oedometer test

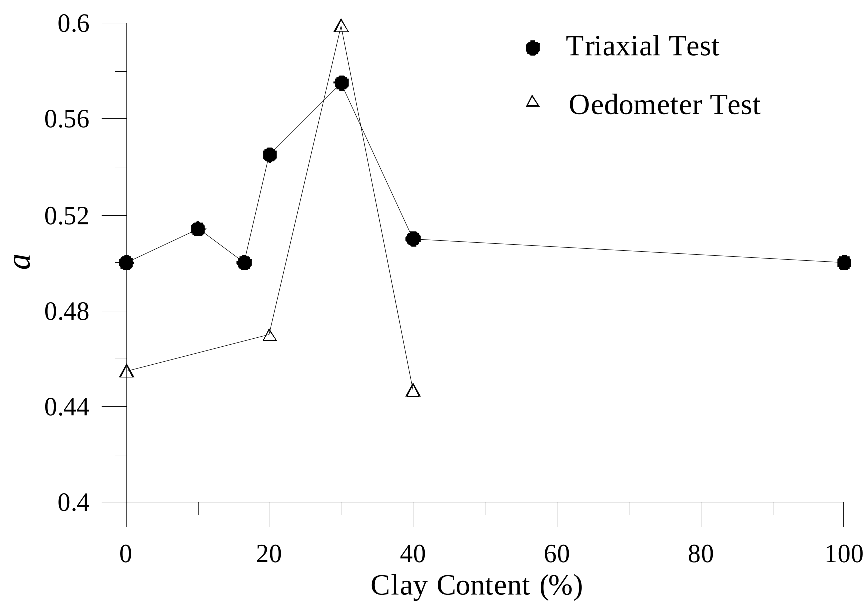


Figure 5-8 Relationship of Parameter a and Clay Content

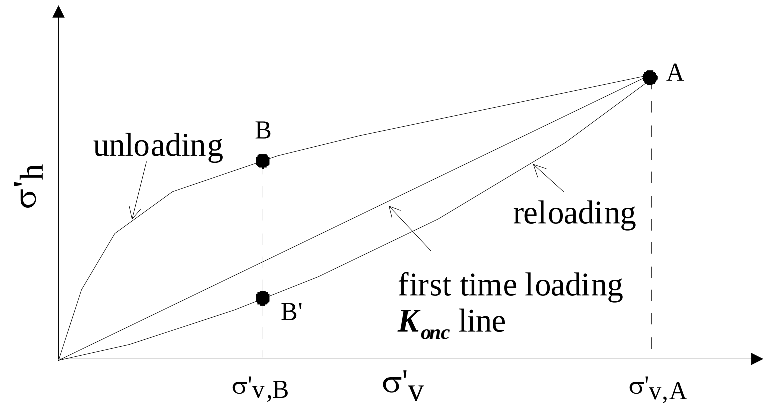


Figure 5-9 Laboratory Loading Path to Determine In-situ K_o Value

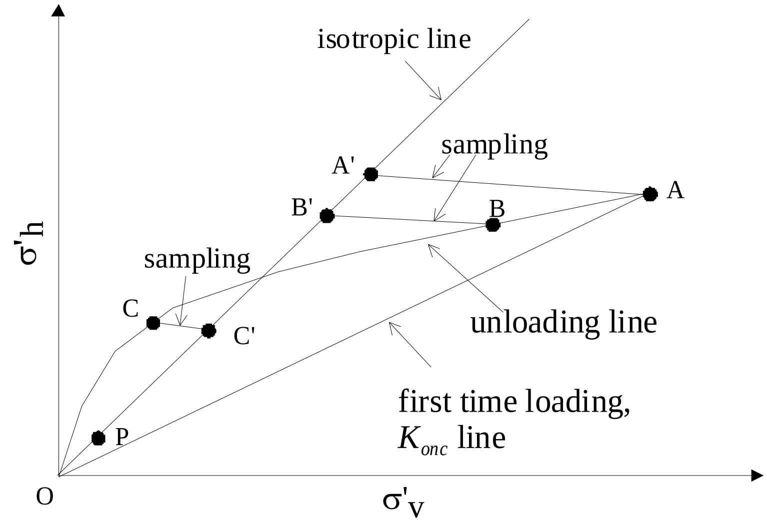


Figure 5-10 Sampling Stress Path for Normally Consolidated, Slightly and Heavily Overconsolidated Soil

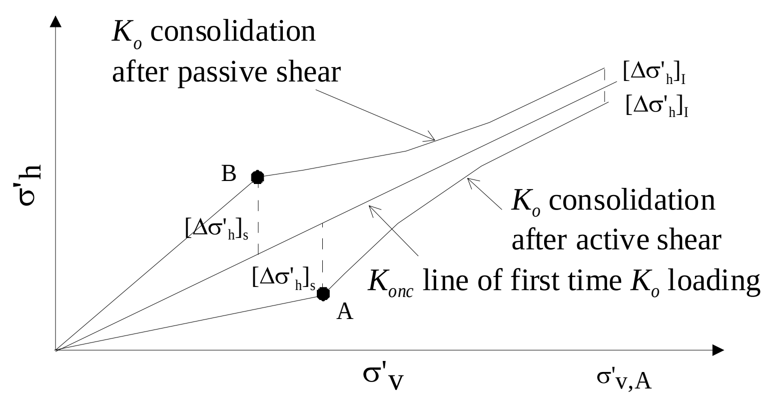


Figure 5-11 K_o of presheared soils

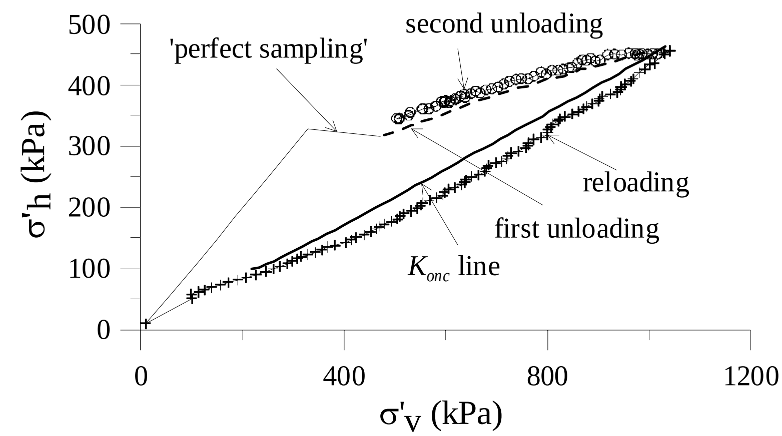


Figure 5-12 Effect of 'perfect sampling' on K_o values of overconsolidated (OCR=2.7) OA

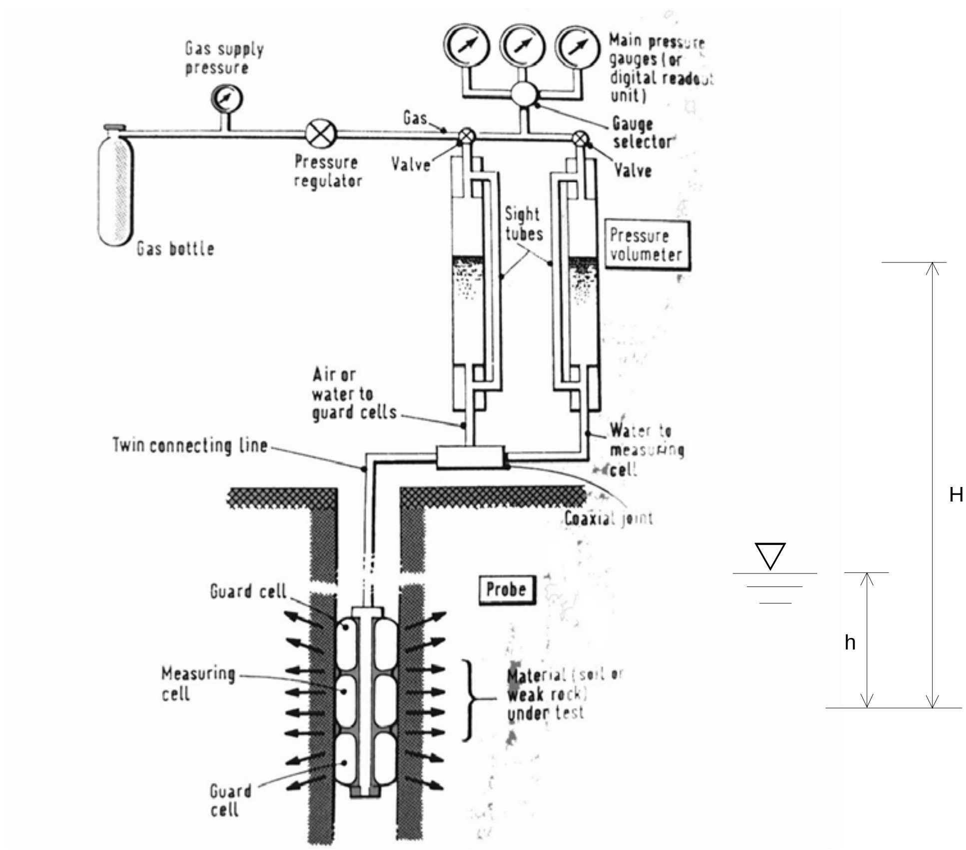


Figure 5-13 Ménard type pressuremeter test layout (Mair and Wood, 1987)

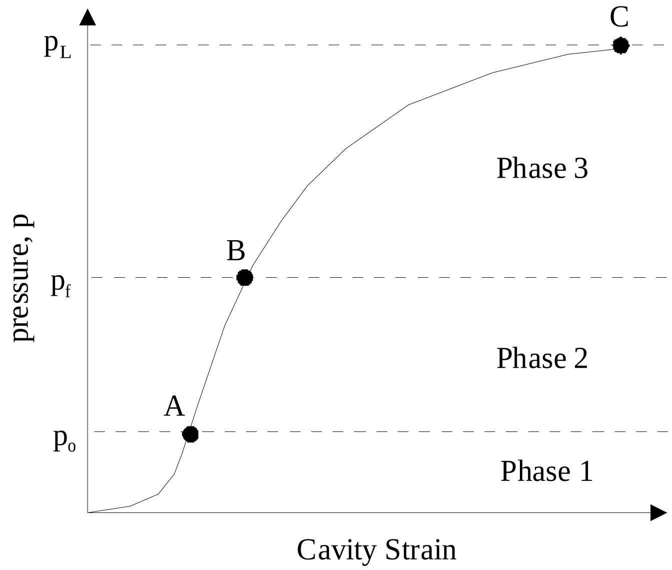


Figure 5-14 Typical Ménard type pressuremeter test Curve (after Mair and Wood, 1987)

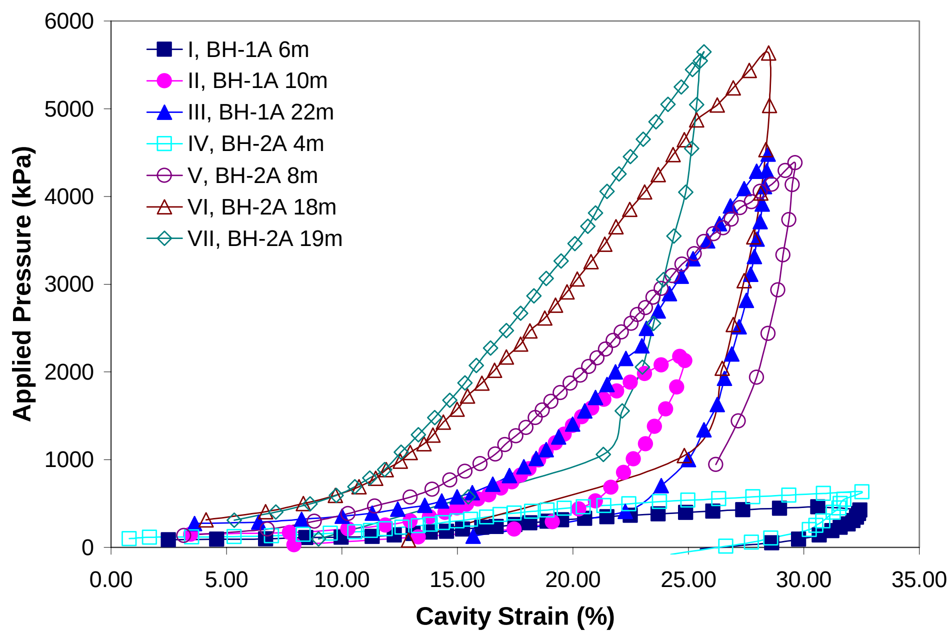


Figure 5-15 Kim Chuan Site pressuremeter test curves

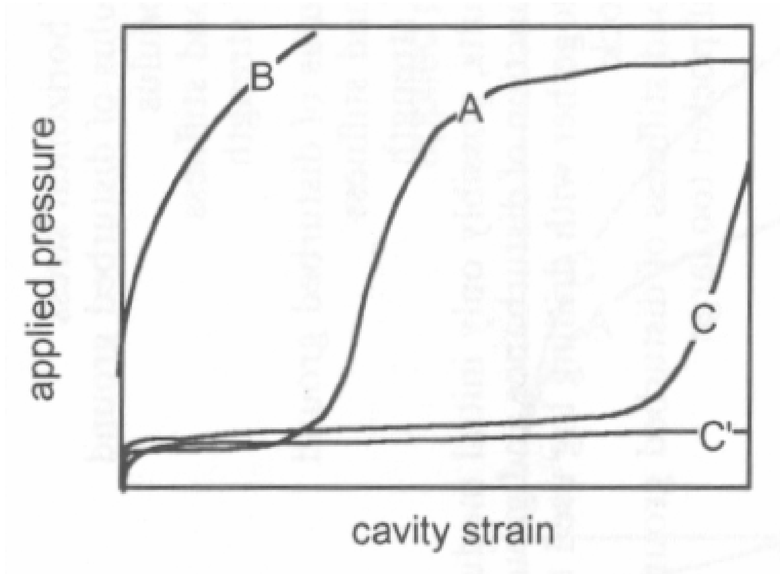


Figure 5-16 Effect of installation on the shape of a test curve (after Clarke, 1995)

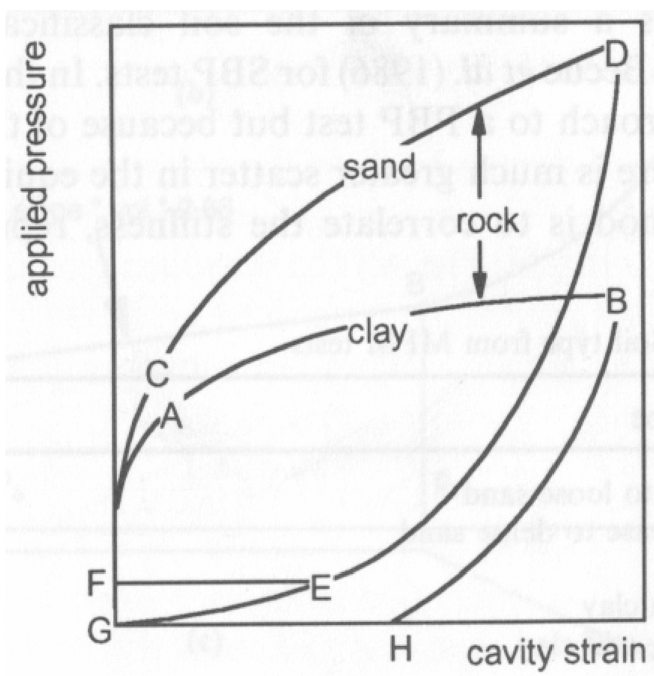


Figure 5-17 The Effect of Ground Type on the Shape of a Test Curve (after Clarke, 1995)

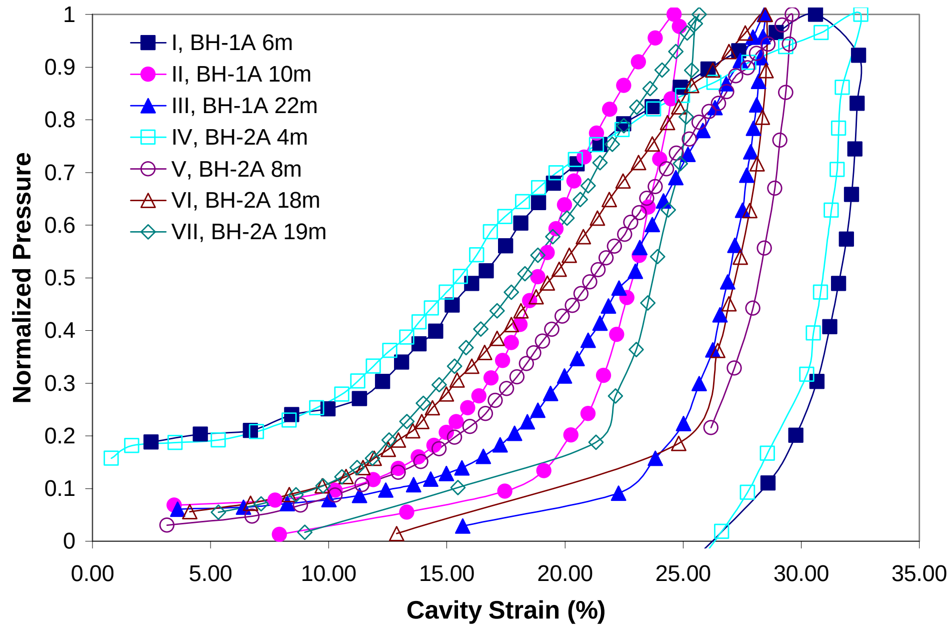


Figure 5-18 Normalized Pressuremeter Test Curves of Kim Chuan Site

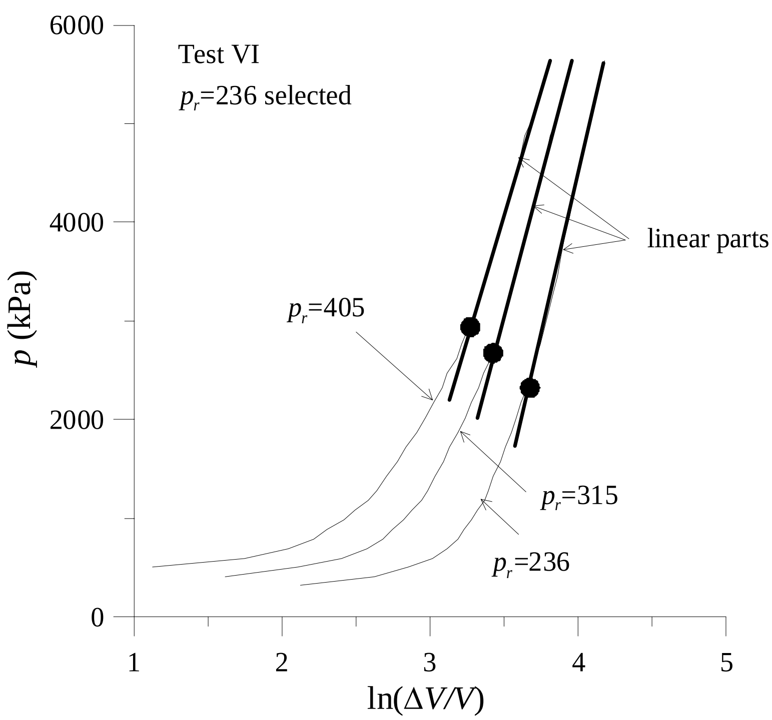


Figure 5-19 Selection of p_r using Denby & Hughes Method, Test VI

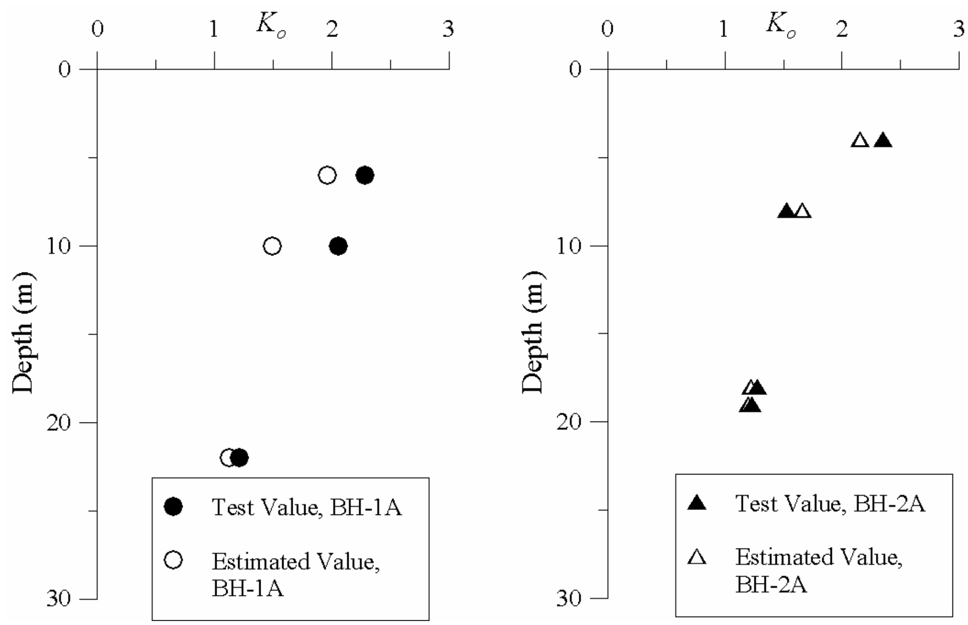


Figure 5-20 Comparison of measured and estimated K_o values versus depth